RICHMOND
HOFFMAYER INC.
STRUCTURAL ENGINEERING

STRUCTURAL ENGINEERING CALCULATIONS

CLIENT: HUGHES UMBANHOWAR ARCHITECTS

PROJECT: VILLAGE NEST

LOCATION: 5762, 5778 5786 Daybreak Ridge, Eden Utah Units 4, 13, 17

BUILDING CODE: 2015 IBC - ASCE 7-10 - WEBER COUNTY, UTAH

RHSE FILE NO.: 2017-0610

ENGINEER: J.J.H.

DATE: JULY 21, 2017

ISSUE: BLDG. DEPT. SUBMITTAL #1

No. 10416469-2202
JOSEPH HOFFMAYER

THESE CALCULATIONS ARE NOT VALID UNLESS
SIGNED AND SEALED IN THE SPACE ABOVE.

4425 CANOGA AVENUE WOODLAND HILLS CA 91364
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Info@RichmondHoffmayer.com

VERTICAL LOADS - ROOFS AND FLOORS:

LOCATION: 5754 DAYBREAK RIDGE, EDEN, UTAH

METAL ROOF	A		WEIGHTS
METAL ROOFING			4.0 psf
1/2" PLY			1.5 psf
2x10 @ 16			2.6 psf
INSULATION			1.0 psf
MISC. MECH.			1.0 psf
1/2" PLY			1.5 psf
SPRINKLERS			1.0 psf
5/8" GWB			3.1 psf
-			- psf
-			- psf
ADDITIONAL LOAD			0.0 psf
DEAD	D	=	15.7 psf
ROOF LIVE	Lo	=	20.0 psf
ROOF PITCH	F	=	0.75 : 12
PITCH REDUCTION	R2	=	1.00
ROOF LIVE	Lr	=	20.0 psf
PITCH ADJUSTED DEAD LOAD	D'	=	15.8 psf

OPENED BALCONY	B		WEIGHTS
DECKING			3.0 psf
FLOOR FRAMING			4.0 psf
-			- psf
-			- psf
-			- psf
-			- psf
-			- psf
-			- psf
-			- psf
-			- psf
ADDITIONAL LOAD			0.0 psf
DEAD	D	=	7.0 psf
LIVE BALCONY (1.5 x RESIDENCIAL LIVE LOAD)	Lo	=	60.0 psf
FRAMING PITCH			0.00 : 12
			1.00
ADJUSTED LIVE BALCONY (1.5 x RESIDENCIAL LIVE LOA	Lo	=	60.0 psf
PITCH ADJUSTED DEAD LOAD	D'	=	7.0 psf

2nd FLOOR	C		WEIGHTS
HARDWOOD			4.0 psf
1-1/8" PLY			3.4 psf
MISC. MECH. } WARMBOARD			1.0 psf
FLOOR FRAMING			4.0 psf
5/8" PLY			1.9 psf
-			- psf
-			- psf
-			- psf
-			- psf
-			- psf
ADDITIONAL LOAD			0.0 psf
DEAD	D	=	14.3 psf
LIVE RESIDENCE	Lo	=	40.0 psf
FRAMING PITCH			0.00 : 12
			1.00
ADJUSTED LIVE RESIDENCE	Lo	=	40.0 psf
PITCH ADJUSTED DEAD LOAD	D'	=	14.3 psf

DRIVEWAY LOADS	D		WEIGHTS
6" CONCRETE			75.0 psf
1-1/8" PLY			3.4 psf
FLOOR FRAMING			4.0 psf
SIDING			3.0 psf
-			- psf
-			- psf
-			- psf
-			- psf
-			- psf
-			- psf
ADDITIONAL LOAD			0.0 psf
DEAD	D	=	85.4 psf
LIVE STAIRS AND EXIT	L	=	100.0 psf
FRAMING PITCH			0.00 : 12
			1.00
ADJUSTED LIVE STAIRS AND EXIT	L	=	100.0 psf
PITCH ADJUSTED DEAD LOAD	D'	=	85.4 psf

20% SNOW LOAD FOR SEISMIC	E		WEIGHTS
-			- psf
- P_f = 264PSF			- psf
-			- psf
-			- psf
- W = 0.2(264)			- psf
-			- psf
- SEISMIC W/SNOW			- psf
-			- psf
-			- psf
-			- psf
ADDITIONAL LOAD			52.8 psf
-	-	=	52.8 psf
-	-	=	0.0 psf
FRAMING PITCH			0.00 : 12
			1.00
ADJUSTED -	-	=	0.0 psf
PITCH ADJUSTED DEAD LOAD	D'	=	52.8 psf

GARAGE LOADS	F		WEIGHTS
4" CONCRETE			50.0 psf
1-1/8" PLY			3.4 psf
FLOOR FRAMING			4.0 psf
INSULATION			1.0 psf
MISC. MECH.			1.0 psf
SPRINKLERS			1.0 psf
5/8" GWB			3.1 psf
-			- psf
-			- psf
-			- psf
ADDITIONAL LOAD			0.0 psf
DEAD	D	=	63.5 psf
LIVE RESIDENTIAL & GARAGE	L	=	40.0 psf
FRAMING PITCH			0.00 : 12
			1.00
ADJUSTED LIVE RESIDENTIAL & GARAGE	L	=	40.0 psf
PITCH ADJUSTED DEAD LOAD	D'	=	63.5 psf

DATE 6/26/2017
SHEET 1A

VERTICAL LOADS - WALLS:

LOCATION: 5754 DAYBREAK RIDGE, EDEN, UTAH

EXTERIOR	U	WEIGHTS
SIDING		3.0 psf
FURRING		2.0 psf
1/2" PLY		1.5 psf
2x8 @ 16		2.1 psf
INSULATION		1.0 psf
INSULATION		1.0 psf
1/2" PLY		1.5 psf
5/8" GWB		3.1 psf
-		- psf
-		- psf
-		- psf
- -	- =	15.2 psf

INTERIOR	V	WEIGHTS
5/8" GWB		3.1 psf
2x6 @ 16		1.6 psf
MISC. MECH.		1.0 psf
5/8" GWB		3.1 psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-	- =	8.9 psf

-	W	WEIGHTS
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-	- =	0.0 psf

-	X	WEIGHTS
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-	- =	0.0 psf

-	Y	WEIGHTS
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-	- =	0.0 psf

-	Z	WEIGHTS
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-		- psf
-	- =	0.0 psf

2

SIP INFO

PBS Panel R-Values

Type I modified EPS core

Core Thickness	R-Value at 75°	R-Value at 40°	R-Value at 25°
3-1/2"	15	16	17
5-1/2"	23	25	26
7-1/4"	30	32	33
9-1/4"	37	40	42
11-1/4"	45	49	51

PBS Panel Weights

Type I modified EPS core

WALLS

Core Thickness	OSB Skin Thickness		
	7/16"	5/8"	3/4"
3-1/2"	3.3	4.6	5.5
5-1/2"	3.5	4.8	5.7
7-1/4"	3.7	5.0	5.9
9-1/4"	3.9	5.2	6.1
11-1/4"	4.0	5.4	6.2

ROOF

Load Charts with a Built in Safety Factor (Refer to current Listing Reports for up to date load tables)

All of Premier's load charts have a built-in safety factor. We have taken our SIPs products' ultimate load at failure and divided this number by 3. The result is then used as the design load value.

Table 1: Maximum Allowable Uniform Transverse Load (psf) – Type S Panels[1,3]

Panel Core Thickness (in)	Deflection Limit[2]	Panel Span (ft)									
		4[4]	8	10	12	14	16	18	20	22	24
3.5	L/360	100	43	29	21	16	10				
	L/240	143	60	42	33	25	16				
	L/180	143*	61*	57	46	34	22				
5.5	L/360	105	52	39	30	24	18	15	11		
	L/240	162	78	58	36	32	28	22	16		
	L/180	191*	80*	60*	46*	40	34	29	21		
7.25	L/360	120	61	60	42	34	26	21	15	13	11
	L/240	179*	85*	75*	61	50	39	31	23	21	18
	L/180	179*	85*	75*	69*	60*	50*	42	31	28	24
9.25	L/360	131	80	66	52	43	33	28	22	20	18
	L/240	168*	86*	71*	57*	51*	46*	42*	34	30	26
	L/180	168*	86*	71*	57*	51*	46*	42*	39*	37*	34*
11.25	L/360	132	94*	76*	51	50	48	38	28	24	20
	L/240	163*	94*	76*	59*	55*	51*	45*	39*	36*	31
	L/180	163*	94*	76*	59*	55*	51*	45*	39*	36*	33*

[1] Table values assume a simply supported panel with 1.5 in. of continuous bearing on facing at supports. Permanent loads, such as dead load, shall not exceed 0.25 times the tabulated load. Panels shall use OSB surface splines not less than 7/16 in. thick inserted below the facing on each side of the panel.

[2] Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code.

[3] Tabulated values for 8 ft walls apply to panels constructed with the OSB strength axis oriented either parallel or perpendicular to supports. Tabulated values for other lengths are based on the strong-axis of the facing material oriented parallel to the span direction.

[4] Panels spanning 4 ft shall be a minimum of 8 ft long spanning a minimum of two 4 ft spans. No single span condition is allowed.

[5] For wall panel capacities utilizing a zero bearing configuration (Figure 2), the allowable load shall be determined using C_v=0.86.

*An asterisk (*) indicates the value shown is governed by the average peak load divided by 3.

PROJECT	VILLAGE NEST	JOB	2017-0610
CLIENT	-		
ADDRESS	EDEN, UT		
DESIGN	HUUM		
ENGINEER	J.H.	DATE	6/14/2017
	RICHMOND PE 44628 HOFFMAYER SE 3455	SHEET	3 OF

SNOW LOADS:

5754 DAYBREAK RIDGE
EDEN, UT 84310
ELEV. 8640 ft

EXPOSURE C

P_g = 264 PSF PER WEBER COUNTY

ROOF PITCH < 1:12 ⟶ CONSIDER FLAT

$P_f = 0.7\,C_e\,C_t\,I_s\,P_g$ ASCE 7-10 (7.3-1)

$C_e = 1.0$ PARTIALLY EXPOSED

$C_t = 1.2$ UNHEATED (UNUSED VACATION HOME)

$I_s = 1.0$ RISK CATEGORY II

$P_f = 0.7(1.0)1.2(1.0)264$ = 221.8 PSF ⟶ 264 PSF

USE ↑
W/O REDUCTION

S = 264 PSF

OVERHANG:

ASCE 7-10 7.4.6

$S_{OH} = 2P_f = 2(221.8)$ = 443.6 PSF

RICHMOND HOFFMAYER INC.
STRUCTURAL ENGINEERING

PROJECT	VILLAGE NEST	**JOB**	2017-0610
CLIENT	-		
ADDRESS	EDEN, UT		
DESIGN	HUUM		
ENGINEER	J.H.	**DATE**	6/14/2017
	RICHMOND PE 44628 HOFFMAYER SE 3435	**SHEET**	4 OF

LOAD COMBINATIONS:

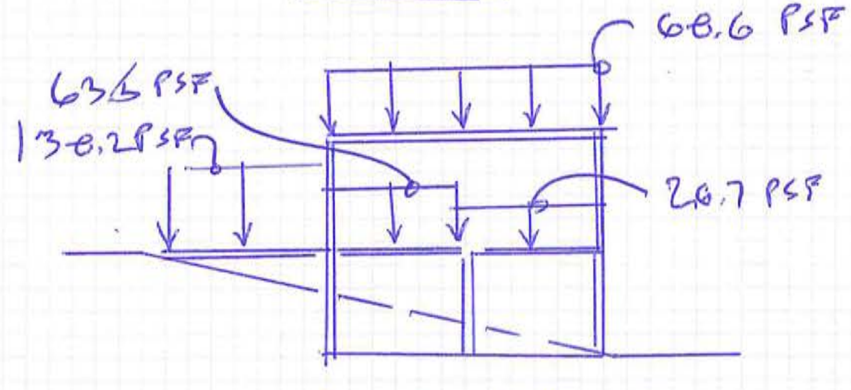
$1.2D + 1.6L + 0.5(L_r \text{ OR } S)$
$1.2D + 1.6(L_r \text{ OR } S) + L$
$1.2D + 1.0W + L + 0.5(L_r \text{ OR } S)$
$1.2D + 1.0E + L + 0.2S$
$0.9D + 1.0W$
$0.9D + 1.0E$

EFFECTIVE SEISMIC WEIGHT:

ASCE 7-10 12.7.2

P_f EXCEEDS 30 PSF ∴ 20% OF UNIFORM DESIGN SNOW LOAD...

$W_{ROOF} = 15.8 + 0.2(264) = \underline{68.6 \text{ PSF}}$
$W_{FLOOR} = \underline{26.7 \text{ PSF}}$
$W_{GARAGE} = \underline{63.5 \text{ PSF}}$
$W_{DRIVE} = 85.4 + 0.2(264) = \underline{138.2 \text{ PSF}}$

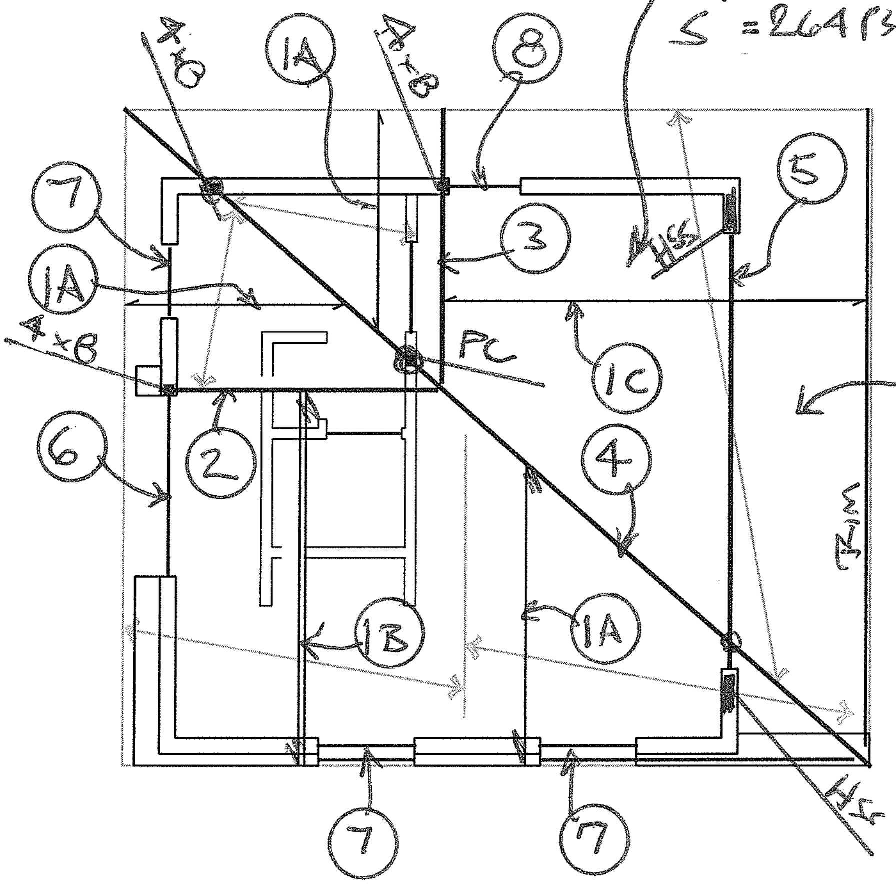


SEISMIC WEIGHTS
TYP.

RICHMOND HOFFMAYER INC.
STRUCTURAL ENGINEERING

PROJECT	FELDSPAR LOWER	JOB	2017-0610
CLIENT	VILLAGE NEST		
ADDRESS	EDEN, UTAH		
DESIGN	HUUM		
ENGINEER	J.H. RICHMOND PE 49628 HOFFMAYER SE 3935	DATE	6/16/2017
		SHEET	5 OF

ROOF
D = 16.8 PSF
Lr = 20 PSF
S = 264 PSF



OVERHANG
D = 15.8 PSF
Lr = 20 PSF
S = 444 PSF

ROOF FRAMING PLAN

FELDSPAR – LOWER
N.T.S.

○ : BEAM #

Wood Beam

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Lic. # : KW-06002886 Licensee : RICHMOND HOFFMAYER

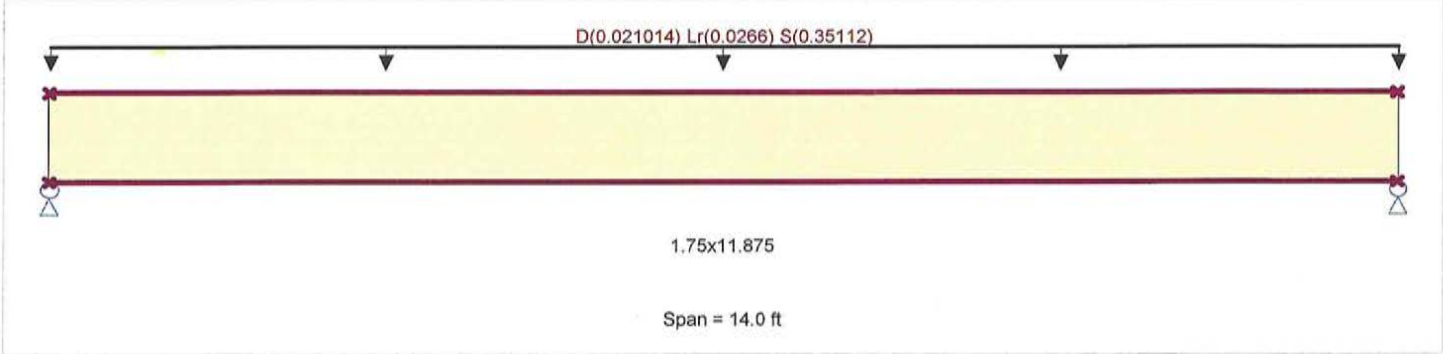
Description : B1A - RAFTERS

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb - Tension	2,600.0 psi	*E : Modulus of Elasticity*	
Load Combination	ASCE 7-10	Fb - Compr	2,600.0 psi	Ebend- xx	1,900.0 ksi
		Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Wood Species	: Trus Joist	Fc - Perp	750.0 psi		
Wood Grade	: MicroLam LVL 1.9 E	Fv	285.0 psi		
		Ft	1,555.0 psi	Density	42.0 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.

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 1.330 ft, (ROOF LOADS)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.774 1	Maximum Shear Stress Ratio	=	0.494 : 1
Section used for this span		1.75x11.875	Section used for this span		1.75x11.875
fb : Actual	=	2,660.07 psi	fv : Actual	=	161.95 psi
FB : Allowable	=	3,438.50 psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	7.000 ft	Location of maximum on span	=	13.029 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.658 in	Ratio =	255 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.697 in	Ratio =	240 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.605	2.605
Overall MINimum	0.088	0.088
D Only	0.147	0.147
+D+Lr	0.333	0.333
+D+S	2.605	2.605
+D+0.750Lr	0.287	0.287
+D+0.750S	1.990	1.990
+0.60D	0.088	0.088
Lr Only	0.186	0.186
S Only	2.458	2.458

Wood Beam

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Licensee : RICHMOND HOFFMAYER

Description : B1B - RAFTERS

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

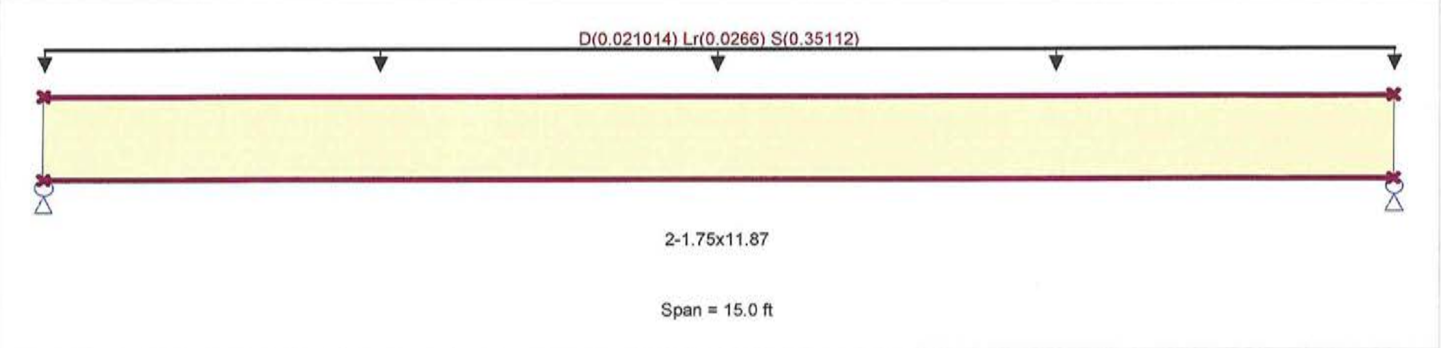
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb - Tension	2,600.0 psi
Fb - Compr	2,600.0 psi
Fc - Prll	2,510.0 psi
Fc - Perp	750.0 psi
Fv	285.0 psi
Ft	1,555.0 psi

E : Modulus of Elasticity

Ebend- xx	1,900.0 ksi
Eminbend - xx	965.71 ksi
Density	42.0 pcf

Repetitive Member Stress Increase



Applied Loads

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

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 1.330 ft, (ROOF LOADS)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.444: 1	Maximum Shear Stress Ratio	=	0.267 : 1
Section used for this span		2-1.75x11.87	Section used for this span		2-1.75x11.87
fb : Actual	=	1,526.82psi	fv : Actual	=	87.49 psi
FB : Allowable	=	3,438.50psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	7.500ft	Location of maximum on span	=	14.015 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.434 in	Ratio =	415 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.459 in	Ratio =	391 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.791	2.791
Overall MINimum	0.095	0.095
D Only	0.158	0.158
+D+Lr	0.357	0.357
+D+S	2.791	2.791
+D+0.750Lr	0.307	0.307
+D+0.750S	2.133	2.133
+0.60D	0.095	0.095
Lr Only	0.200	0.200
S Only	2.633	2.633

Wood Beam

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Description : B1C - RAFTERS w OVERHANG

CODE REFERENCES

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

Material Properties

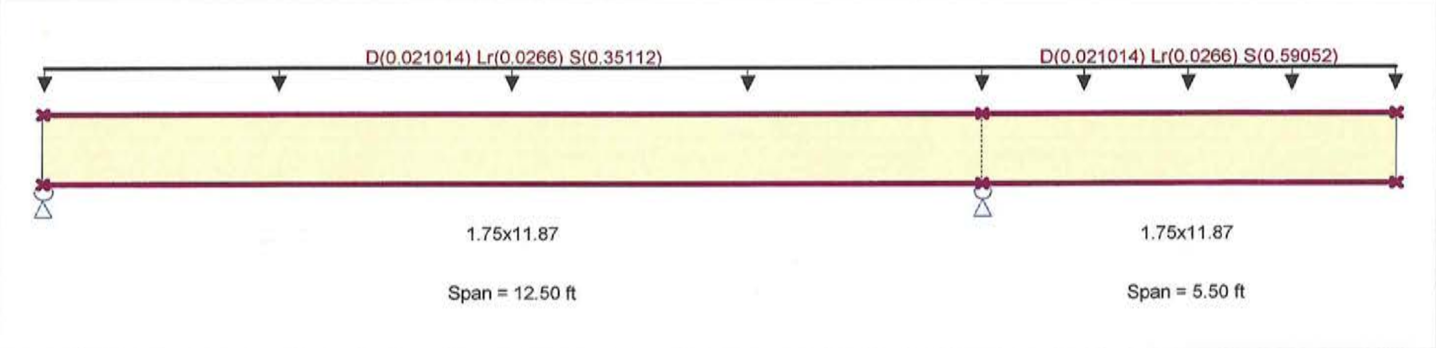
Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

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

Fb - Tension	2,600.0 psi	*E : Modulus of Elasticity*	
Fb - Compr	2,600.0 psi	Ebend- xx	1,900.0 ksi
Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Fc - Perp	750.0 psi		
Fv	285.0 psi		
Ft	1,555.0 psi	Density	42.0 pcf

Repetitive Member Stress Increase


D(0.021014) Lr(0.0266) S(0.35112)
D(0.021014) Lr(0.0266) S(0.59052)
1.75x11.87
Span = 12.50 ft
1.75x11.87
Span = 5.50 ft

Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 1.330 ft, (ROOF LOADS)
Load for Span Number 2
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.4440 ksf, Tributary Width = 1.330 ft, (ROOF LOADS)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.785 1	Maximum Shear Stress Ratio	=	0.608 : 1
Section used for this span		**1.75x11.87**	Section used for this span		**1.75x11.87**
fb : Actual	=	2,698.63psi	fv : Actual	=	199.37 psi
FB : Allowable	=	3,438.50psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	12.500ft	Location of maximum on span	=	12.500ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.428 in	Ratio =	308 >=240.
Max Upward Transient Deflection	-0.044 in	Ratio =	2976 >=240.
Max Downward Total Deflection	0.429 in	Ratio =	306 >=240.
Max Upward Total Deflection	-0.043 in	Ratio =	3046 >=240.

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	1.586	6.429	
Overall MINimum	-0.032	0.163	
D Only	0.106	0.272	
+D+Lr, LL Comb Run (*L)	0.074	0.451	
+D+Lr, LL Comb Run (L*)	0.272	0.439	
+D+Lr, LL Comb Run (LL)	0.240	0.617	
+D+S	1.586	6.429	
+D+0.750Lr, LL Comb Run (*L)	0.082	0.406	
+D+0.750Lr, LL Comb Run (L*)	0.231	0.397	
+D+0.750Lr, LL Comb Run (LL)	0.206	0.531	

Wood Beam

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Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : B1C - RAFTERS w OVERHANG

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
+D+0.750S	1.216	4.890	
+0.60D	0.064	0.163	
Lr Only, LL Comb Run (*L)	-0.032	0.178	
Lr Only, LL Comb Run (L*)	0.166	0.166	
Lr Only, LL Comb Run (LL)	0.134	0.345	
S Only	1.480	6.157	

Wood Beam

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Description : B2 - ROOF BEAM

CODE REFERENCES

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

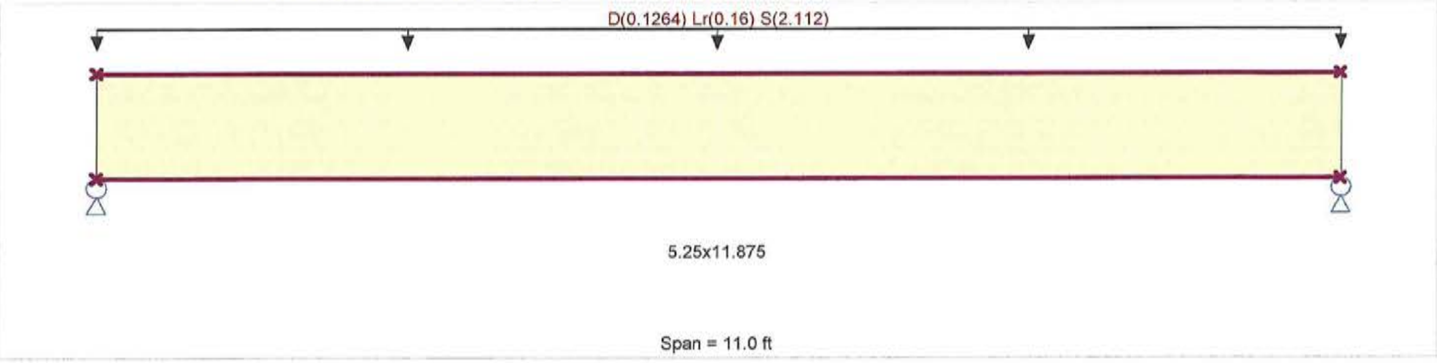
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : Parallam PSL 2.0E

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

Fb - Tension	2,900.0 psi	*E : Modulus of Elasticity*	
Fb - Compr	2,900.0 psi	Ebend- xx	2,000.0 ksi
Fc - Prll	2,900.0 psi	Eminbend - xx	1,016.54 ksi
Fc - Perp	625.0 psi		
Fv	290.0 psi		
Ft	2,025.0 psi	Density	45.050 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 8.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.996 : 1	**Maximum Shear Stress Ratio**	=	0.739 : 1
Section used for this span		5.25x11.875	Section used for this span		5.25x11.875
fb : Actual	=	3,321.29 psi	fv : Actual	=	246.45 psi
FB : Allowable	=	3,335.00 psi	Fv : Allowable	=	333.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.500 ft	Location of maximum on span	=	10.036 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.478 in	Ratio = 276 >=240.
Max Upward Transient Deflection	0.000 in	Ratio = 0 <240.0
Max Downward Total Deflection	0.511 in	Ratio = 258 >=240.
Max Upward Total Deflection	0.000 in	Ratio = 0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	12.418	12.418
Overall MINimum	0.481	0.481
D Only	0.802	0.802
+D+Lr	1.682	1.682
+D+S	12.418	12.418
+D+0.750Lr	1.462	1.462
+D+0.750S	9.514	9.514
+0.60D	0.481	0.481
Lr Only	0.880	0.880
S Only	11.616	11.616

Wood Beam

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Description : B3 - ROOF BEAM

CODE REFERENCES

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

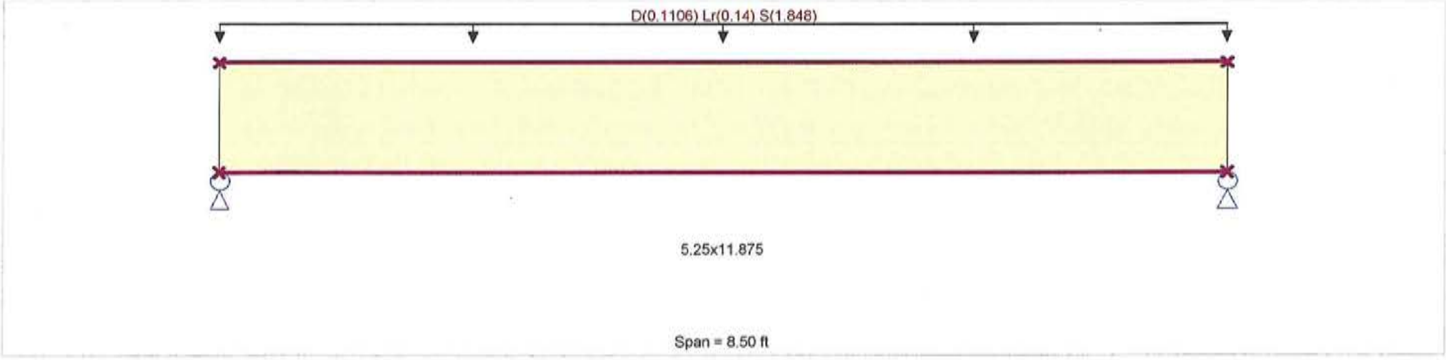
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : Parallam PSL 2.0E

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

Fb - Tension	2,900.0 psi	*E : Modulus of Elasticity*	
Fb - Compr	2,900.0 psi	Ebend- xx	2,000.0 ksi
Fc - Prll	2,900.0 psi	Eminbend - xx	1,016.54 ksi
Fc - Perp	625.0 psi		
Fv	290.0 psi		
Ft	2,025.0 psi	Density	45.050 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 7.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.521: 1	Maximum Shear Stress Ratio	=	0.469 : 1
Section used for this span		5.25x11.875	Section used for this span		5.25x11.875
fb : Actual	=	1,737.41psi	fv : Actual	=	156.50 psi
FB : Allowable	=	3,335.00psi	Fv : Allowable	=	333.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.250ft	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	0.149 in	Ratio =	684 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.159 in	Ratio =	639 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	8.407	8.407
Overall MINimum	0.332	0.332
D Only	0.553	0.553
+D+Lr	1.148	1.148
+D+S	8.407	8.407
+D+0.750Lr	0.999	0.999
+D+0.750S	6.443	6.443
+0.60D	0.332	0.332
Lr Only	0.595	0.595
S Only	7.854	7.854

Wood Beam

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Description : B4 - RIDGE

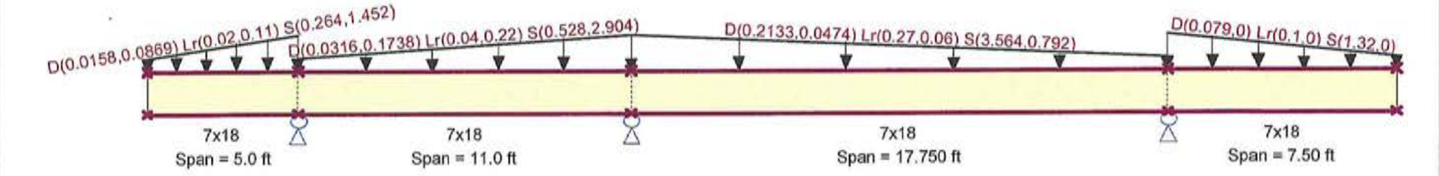
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb - Tension	2,900.0 psi	*E : Modulus of Elasticity*	
Load Combination	ASCE 7-10	Fb - Compr	2,900.0 psi	Ebend- xx	2,000.0ksi
		Fc - Prll	2,900.0 psi	Eminbend - xx	1,016.54ksi
Wood Species	: Trus Joist	Fc - Perp	625.0 psi		
Wood Grade	: Parallam PSL 2.0E	Fv	290.0 psi		
		Ft	2,025.0 psi	Density	45.050pcf

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


D(0.0158,0.0869) Lr(0.02,0.11) S(0.264,1.452)
D(0.0316,0.1738) Lr(0.04,0.22) S(0.528,2.904)
D(0.2133,0.0474) Lr(0.27,0.06) S(3.564,0.792)
D(0.079,0) Lr(0.1,0) S(1.32,0)
7x18
Span = 5.0 ft
7x18
Span = 11.0 ft
7x18
Span = 17.750 ft
7x18
Span = 7.50 ft

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
Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 0.0 -->> 5.0 ft, Trib Width = 1.0->5.50 ft, (
Load for Span Number 2
Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 0.0 -->> 11.0 ft, Trib Width = 2.0->11.0 ft,
Load for Span Number 3
Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 0.0 -->> 17.750 ft, Trib Width = 13.50->3.
Load for Span Number 4
Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 0.0 -->> 7.50 ft, Trib Width = 5.0->0.0 ft, (

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.643 : 1	**Maximum Shear Stress Ratio**	=	0.822 : 1
Section used for this span		**7x18**	Section used for this span		**7x18**
fb : Actual	=	2,049.11psi	fv : Actual	=	274.07 psi
FB : Allowable	=	3,188.23psi	Fv : Allowable	=	333.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	11.000ft	Location of maximum on span	=	11.000 ft
Span # where maximum occurs	=	Span # 2	Span # where maximum occurs	=	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.357 in	Ratio =	597 >=360
Max Upward Transient Deflection	-0.412 in	Ratio =	436 >=360
Max Downward Total Deflection	0.382 in	Ratio =	558 >=240.
Max Upward Total Deflection	-0.433 in	Ratio =	414 >=240.

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
Overall MAXimum		7.627	45.573	19.198	
Overall MINimum		0.012	-0.074	0.007	
D Only		0.799	3.111	1.712	
+D+Lr, LL Comb Run (***L)		0.825	3.016	2.156	

Wood Beam

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Description : B4 - RIDGE

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
+D+Lr, LL Comb Run (**L*)		0.419	5.502	2.630	
+D+Lr, LL Comb Run (**LL)		0.445	5.407	3.074	
+D+Lr, LL Comb Run (*L**)		1.278	4.107	1.668	
+D+Lr, LL Comb Run (*L*L)		1.304	4.012	2.112	
+D+Lr, LL Comb Run (*LL*)		0.897	6.498	2.586	
+D+Lr, LL Comb Run (*LLL)		0.924	6.402	3.030	
+D+Lr, LL Comb Run (L***)		1.192	3.037	1.719	
+D+Lr, LL Comb Run (L**L)		1.218	2.941	2.163	
+D+Lr, LL Comb Run (L*L*)		0.812	5.428	2.637	
+D+Lr, LL Comb Run (L*LL)		0.838	5.332	3.081	
+D+Lr, LL Comb Run (LL**)		1.670	4.033	1.675	
+D+Lr, LL Comb Run (LL*L)		1.697	3.937	2.119	
+D+Lr, LL Comb Run (LLL*)		1.290	6.423	2.593	
+D+Lr, LL Comb Run (LLLL)		1.316	6.328	3.037	
+D+S		7.627	45.573	19.198	
+D+0.750Lr, LL Comb Run (***L)		0.819	3.040	2.045	
+D+0.750Lr, LL Comb Run (**L*)		0.514	4.904	2.401	
+D+0.750Lr, LL Comb Run (**LL)		0.534	4.833	2.734	
+D+0.750Lr, LL Comb Run (*L**)		1.158	3.858	1.679	
+D+0.750Lr, LL Comb Run (*L*L)		1.178	3.786	2.012	
+D+0.750Lr, LL Comb Run (*LL*)		0.873	5.651	2.368	
+D+0.750Lr, LL Comb Run (*LLL)		0.893	5.580	2.701	
+D+0.750Lr, LL Comb Run (L***)		1.094	3.055	1.717	
+D+0.750Lr, LL Comb Run (L**L)		1.113	2.984	2.050	
+D+0.750Lr, LL Comb Run (L*L*)		0.809	4.848	2.406	
+D+0.750Lr, LL Comb Run (L*LL)		0.828	4.777	2.739	
+D+0.750Lr, LL Comb Run (LL**)		1.452	3.802	1.684	
+D+0.750Lr, LL Comb Run (LL*L)		1.472	3.731	2.017	
+D+0.750Lr, LL Comb Run (LLL*)		1.167	5.595	2.373	
+D+0.750Lr, LL Comb Run (LLLL)		1.187	5.524	2.706	
+D+0.750S		5.920	34.958	14.826	
+0.60D		0.479	1.867	1.027	
Lr Only, LL Comb Run (***L)		0.026	-0.095	0.444	
Lr Only, LL Comb Run (**L*)		-0.380	2.391	0.918	
Lr Only, LL Comb Run (**LL)		-0.354	2.295	1.362	
Lr Only, LL Comb Run (*L**)		0.478	0.996	-0.044	
Lr Only, LL Comb Run (*L*L)		0.505	0.900	0.400	
Lr Only, LL Comb Run (*LL*)		0.098	3.387	0.874	
Lr Only, LL Comb Run (*LLL)		0.125	3.291	1.318	
Lr Only, LL Comb Run (L***)		0.393	-0.074	0.007	
Lr Only, LL Comb Run (L**L)		0.419	-0.170	0.451	
Lr Only, LL Comb Run (L*L*)		0.012	2.316	0.925	
Lr Only, LL Comb Run (L*LL)		0.039	2.221	1.369	
Lr Only, LL Comb Run (LL**)		0.871	0.921	-0.038	
Lr Only, LL Comb Run (LL*L)		0.897	0.826	0.407	
Lr Only, LL Comb Run (LLL*)		0.491	3.312	0.881	
Lr Only, LL Comb Run (LLLL)		0.517	3.217	1.325	
S Only		6.828	42.462	17.485	

UNBALANCED
LOADING
CHECK

Wood Beam

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Description : B5 - BIG OPENING

CODE REFERENCES

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

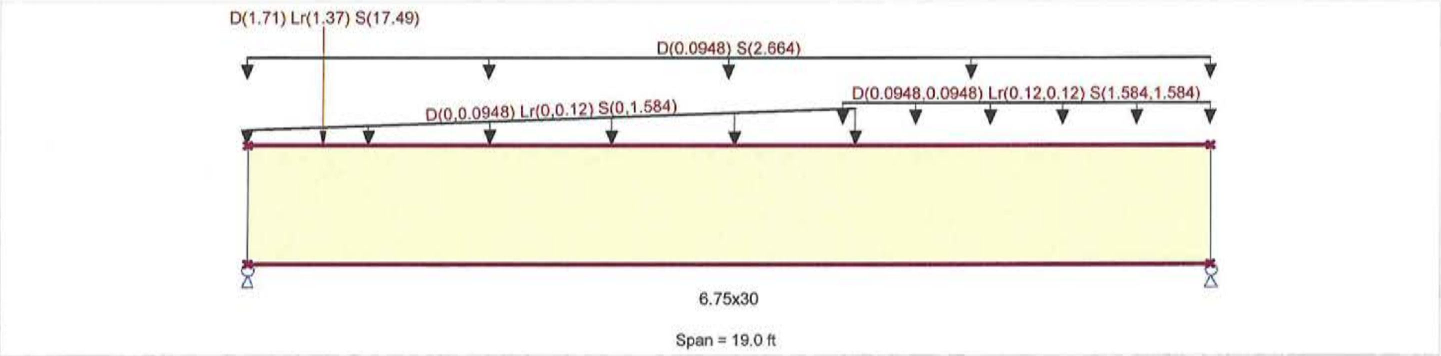
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F - V4

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

		E : Modulus of Elasticity	
Fb - Tension	2,400.0 psi	Ebend- xx	1,800.0 ksi
Fb - Compr	1,850.0 psi	Eminbend - xx	950.0 ksi
Fc - Prll	1,650.0 psi	Ebend- yy	1,600.0 ksi
Fc - Perp	650.0 psi	Eminbend - yy	850.0 ksi
Fv	265.0 psi	Density	31.20 pcf
Ft	1,100.0 psi		


D(1.71) Lr(1.37) S(17.49)
D(0.0948) S(2.664)
D(0.0948,0.0948) Lr(0.12,0.12) S(1.584,1.584)
D(0,0.0948) Lr(0,0.12) S(0,1.584)
6.75x30
Span = 19.0 ft

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

Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 0.0 -->> 12.0 ft, Trib Width = 0.0->6.0 ft, (
Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 11.750 -->> 19.0 ft, Trib Width = 6.0 ft, (R
Uniform Load : D = 0.01580, S = 0.4440 ksf, Tributary Width = 6.0 ft, (EAVE)
Point Load : D = 1.710, Lr = 1.370, S = 17.490 k @ 1.50 ft, (B4)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.950 : 1	**Maximum Shear Stress Ratio**	=	0.755 : 1
Section used for this span		6.75x30	Section used for this span		6.75x30
fb : Actual	=	2,350.85psi	fv : Actual	=	229.96 psi
FB : Allowable	=	2,474.58psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	9.569ft	Location of maximum on span	=	16.504 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.453 in	Ratio =	503 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.479 in	Ratio =	475 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	52.463	42.232
Overall MINimum	1.845	1.115
D Only	3.353	2.248
+D+Lr	5.198	3.363
+D+S	52.463	42.232
+D+0.750Lr	4.736	3.085
+D+0.750S	40.186	32.236
+0.60D	2.012	1.349

Wood Beam

Description : B5 - BIG OPENING

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Lr Only	1.845	1.115
S Only	49.111	39.983

RICHMOND
HOFFMAYER INC.
STRUCTURAL ENGINEERING

Project Title: VILLAGE NEST
Engineer: JJH
Project Descr: MULTI UNIT PROJECT

Project ID: 2017-0610

15.1

Printed: 20 JUN 2017, 10:31AM

Steel Beam

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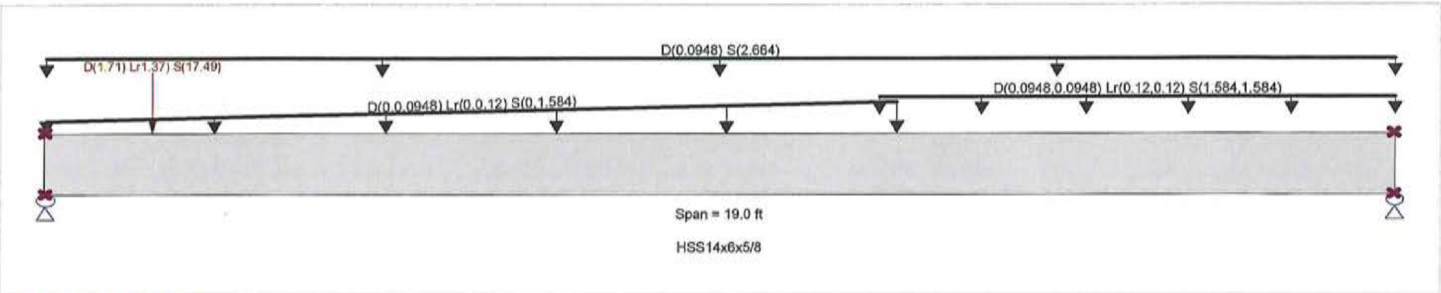
Description : B5 - BIG OPENING - STEEL OPTION

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	46.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		


D(0.0948) S(2.664)
D(1.71) Lr1.37) S(17.49)
D(0.0948,0.0948) Lr(0.12,0.12) S(1.584,1.584)
D(0,0.0948) Lr(0,0.12) S(0,1.584)
Span = 19.0 ft
HSS14x6x5/8

Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 0.0 -->> 12.0 ft, Trib Width = 0.
Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 11.750 -->> 19.0 ft, Trib Width =
Uniform Load : D = 0.01580, S = 0.4440 ksf, Tributary Width = 6.0 ft, (EAVE)
Point Load : D = 1.710, Lr = 1.370, S = 17.490 k @ 1.50 ft, (B4)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.981 : 1	Maximum Shear Stress Ratio =	0.224 : 1
Section used for this span	HSS14x6x5/8	Section used for this span	HSS14x6x5/8
Ma : Applied	199.816 k-ft	Va : Applied	52.772 k
Mn / Omega : Allowable	203.603 k-ft	Vn/Omega : Allowable	235.387 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	9.554ft	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	0.891 in	Ratio =	255 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.951 in	Ratio =	240 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 19.00 ft	1	0.060	0.016	12.25		12.25	340.02	203.60	1.12	1.00	3.66	393.10	235.39
+D+Lr													
Dsgn. L = 19.00 ft	1	0.085	0.023	17.39		17.39	340.02	203.60	1.12	1.00	5.51	393.10	235.39
+D+S													
Dsgn. L = 19.00 ft	1	0.981	0.224	199.82		199.82	340.02	203.60	1.13	1.00	52.77	393.10	235.39
+D+0.750Lr													
Dsgn. L = 19.00 ft	1	0.079	0.021	16.10		16.10	340.02	203.60	1.12	1.00	5.04	393.10	235.39
+D+0.750S													
Dsgn. L = 19.00 ft	1	0.751	0.172	152.92		152.92	340.02	203.60	1.13	1.00	40.49	393.10	235.39
+0.60D													
Dsgn. L = 19.00 ft	1	0.036	0.009	7.35		7.35	340.02	203.60	1.12	1.00	2.20	393.10	235.39

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.9512	9.500		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	52.772	42.540
Overall MINimum	1.845	1.115

15.2

Steel Beam

Description : B5 - BIG OPENING - STEEL OPTION

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
D Only	3.661	2.556
+D+Lr	5.506	3.672
+D+S	52.772	42.540
+D+0.750Lr	5.045	3.393
+D+0.750S	40.494	32.544
+0.60D	2.197	1.534
Lr Only	1.845	1.115
S Only	49.111	39.983

Wood Beam

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : B6 - HEADER

CODE REFERENCES

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

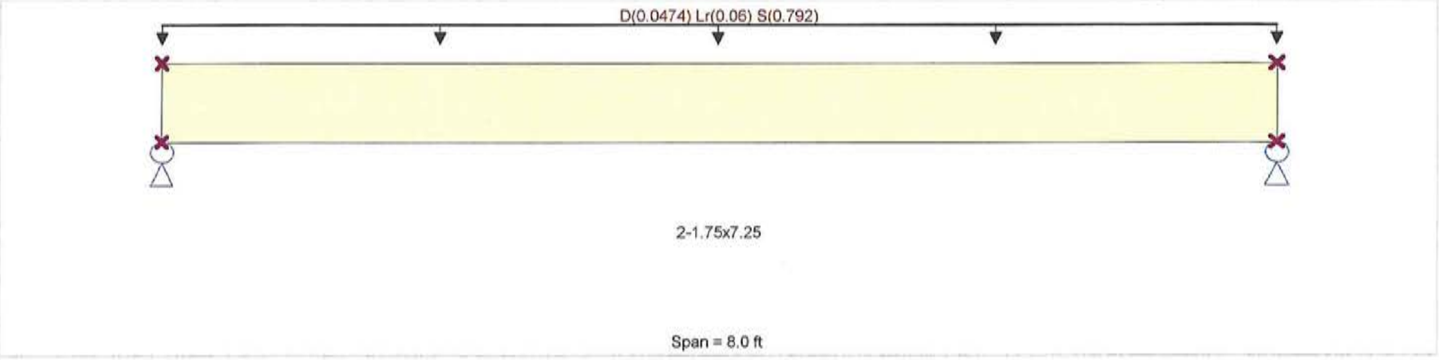
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

Beam Bracing : Completely Unbraced

Fb - Tension	2600 psi	*E : Modulus of Elasticity*	
Fb - Compr	2600 psi	Ebend- xx	1900 ksi
Fc - Prll	2510 psi	Eminbend - xx	965.71 ksi
Fc - Perp	750 psi		
Fv	285 psi		
Ft	1555 psi	Density	42 pcf



Applied Loads

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

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 3.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.895 : 1	Maximum Shear Stress Ratio	=	0.517 : 1
Section used for this span		**2-1.75x7.25**	Section used for this span		**2-1.75x7.25**
fb : Actual	=	2,628.13 psi	fv : Actual	=	169.50 psi
FB : Allowable	=	2,936.44 psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.000 ft	Location of maximum on span	=	7.416 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.348 in	Ratio =	276 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.368 in	Ratio =	260 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.358	3.358
Overall MINimum	0.114	0.114
D Only	0.190	0.190
+D+Lr	0.430	0.430
+D+S	3.358	3.358
+D+0.750Lr	0.370	0.370
+D+0.750S	2.566	2.566
+0.60D	0.114	0.114
Lr Only	0.240	0.240
S Only	3.168	3.168

Wood Beam

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Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : B7 - BEARING HEADERS

CODE REFERENCES

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

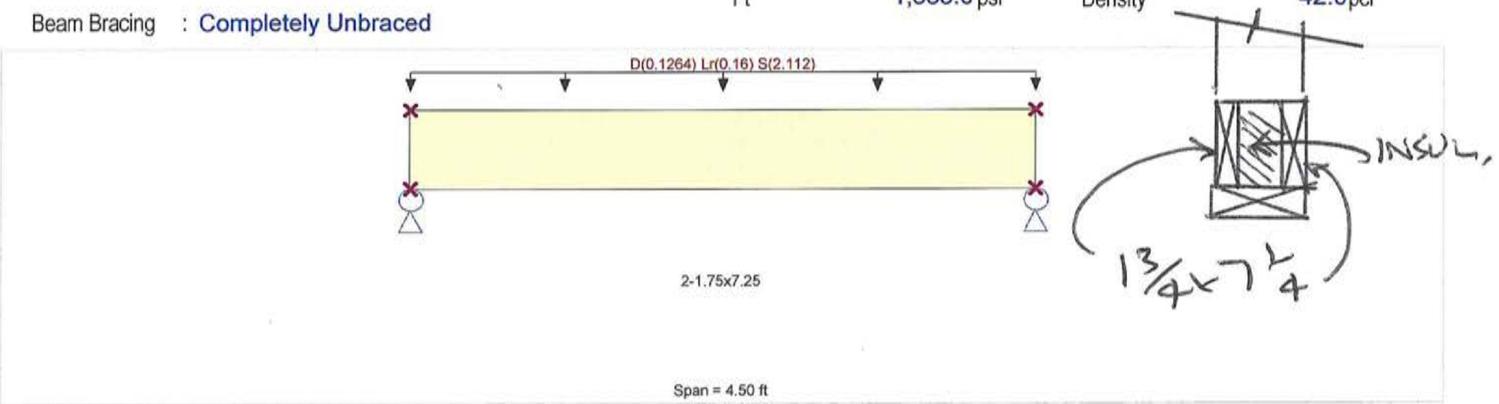
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

Beam Bracing : Completely Unbraced

Fb - Tension	2,600.0 psi	*E : Modulus of Elasticity*	
Fb - Compr	2,600.0 psi	Ebend- xx	1,900.0 ksi
Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Fc - Perp	750.0 psi		
Fv	285.0 psi		
Ft	1,555.0 psi	Density	42.0 pcf



Applied Loads

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

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 8.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.749 : 1	**Maximum Shear Stress Ratio**	=	0.670 : 1
Section used for this span		2-1.75x7.25	Section used for this span		2-1.75x7.25
fb : Actual	=	2,217.49 psi	fv : Actual	=	219.49 psi
FB : Allowable	=	2,960.54 psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	2.250 ft	Location of maximum on span	=	3.909 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.093 in	Ratio =	581 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.098 in	Ratio =	548 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	5.036	5.036
Overall MINimum	0.171	0.171
D Only	0.284	0.284
+D+Lr	0.644	0.644
+D+S	5.036	5.036
+D+0.750Lr	0.554	0.554
+D+0.750S	3.848	3.848
+0.60D	0.171	0.171
Lr Only	0.360	0.360
S Only	4.752	4.752

1e

Wood Beam

Description : B8 - BEARING HEADERS

CODE REFERENCES

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

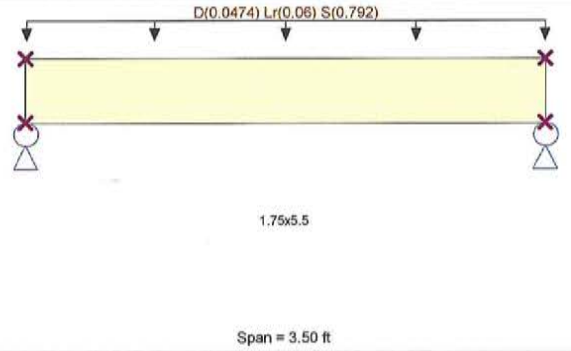
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

Beam Bracing : Completely Unbraced

Fb - Tension	2,600.0 psi	*E : Modulus of Elasticity*	
Fb - Compr	2,600.0 psi	Ebend- xx	1,900.0 ksi
Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Fc - Perp	750.0 psi		
Fv	285.0 psi		
Ft	1,555.0 psi	Density	42.0 pcf



Applied Loads

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

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 3.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.603 1	**Maximum Shear Stress Ratio**	=	0.520 : 1
Section used for this span		1.75x5.5	Section used for this span		1.75x5.5
fb : Actual	=	1,748.17psi	fv : Actual	=	170.44 psi
FB : Allowable	=	2,900.40psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	1.750ft	Location of maximum on span	=	3.053 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.058 in	Ratio =	719>=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0<240.0
Max Downward Total Deflection	0.062 in	Ratio =	679>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0<240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.469	1.469
Overall MINimum	0.050	0.050
D Only	0.083	0.083
+D+Lr	0.188	0.188
+D+S	1.469	1.469
+D+0.750Lr	0.162	0.162
+D+0.750S	1.122	1.122
+0.60D	0.050	0.050
Lr Only	0.105	0.105
S Only	1.386	1.386

WIND ON EXTERIOR WALLS:

115 MPH - EXP "C" ASCE 7-10

$W_{MIN} = 16$ PSF

$q_z = 0.00256 K_z K_{zt} K_d V^2$

$K_z = 0.9$ @ 20' TABLE 30.3-1

$K_{zt} = 1$

$K_d = 0.85$

$q_z = 0.00256(0.9 \times 1 \times 0.85)115^2 = \underline{\underline{25.9 \text{ PSF}}}$

$P = 25.9(GC_p - GC_{pi})$

$A_{STUD} = 10' \times 1.33' = 13.3 \text{ ft}^2 \longrightarrow GC_p = 1.8$

$GC_{pi} = 0.18$

$P = 25.9(1.8 + 0.18) = \underline{\underline{51.28 \text{ PSF}}} > 16 \text{ PSF}$

ALL WALL CLADDING MUST RESIST $\underline{\underline{51.3 \text{ PSF}}}$ WIND

Wood Column

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Lic. # : KW-06002886
Licensee : RICHMOND HOFFMAYER

Description : ROOF SUPPORT POSTS - EXTERIOR WALLS

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 10.50 ft
(Used for non-slender calculations)
Wood Species Spruce - Pine - Fir
Wood Grade No.1

Fb +	850.0 psi	Fv	125.0 psi
Fb -	850.0 psi	Ft	550.0 psi
Fc - Prll	700.0 psi	Density	26.210 pcf
Fc - Perp	425.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,300.0	1,300.0	1,300.0 ksi
Minimum	470.0	470.0	

Wood Section Name 4x8
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	3.50 in
Exact Depth	7.250 in
Area	25.375 in^2
Ix	111.148 in^4
Iy	25.904 in^4

Allow Stress Modification Factors	
Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.050
Cf or Cv for Tension	1.20
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 10 ft, K = 1.0

Applied Loads

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

Column self weight included : 48.495 lbs * Dead Load Factor
AXIAL LOADS . . .
ROOF SUPPORT: Axial Load at 10.50 ft, D = 1.50, Lr = 1.50, S = 15.0 k
BENDING LOADS . . .
WIND: Lat. Uniform Load creating Mx-x, W = 0.2050 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.9256 : 1
	Load Combination	+D+S
	Governing NDS Forumla	Comp Only, fc/Fc'
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Applied Axial	16.549 k
	Applied Mx	0.0 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	704.56 psi
PASS	Maximum Shear Stress Ratio =	0.1272 : 1
	Load Combination	+D+0.60W
	Location of max.above base	0.0 ft
	Applied Design Shear	38.172 psi
	Allowable Shear	200.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	1.076 k	Bottom along Y-Y	1.076 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.3922 in at 5.285 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

			Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
Load Combination	C_D	C_P	Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.877	0.1052	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+Lr	1.250	0.815	0.1604	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+S	1.150	0.834	0.9256	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.750Lr	1.250	0.815	0.1407	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.750S	1.150	0.834	0.7159	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.60W	1.600	0.750	0.3970	PASS	5.285 ft	0.1272	PASS	0.0 ft
+D+0.750Lr+0.450W	1.600	0.750	0.3184	PASS	5.285 ft	0.09543	PASS	0.0 ft
+D+0.750S+0.450W	1.600	0.750	0.7653	PASS	5.215 ft	0.09543	PASS	0.0 ft
+0.60D+0.60W	1.600	0.750	0.3869	PASS	5.285 ft	0.1272	PASS	0.0 ft

U

Wood Column

Description : ROOF SUPPORT POSTS - EXTERIOR WALLS

Load Combination Results

			Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
Load Combination	C_D	C_P	Stress Ratio	Status	Location	Stress Ratio	Status	Location
+0.60D	1.600	0.750	0.04153	PASS	0.0 ft	0.0	PASS	10.50 ft

Maximum Reactions

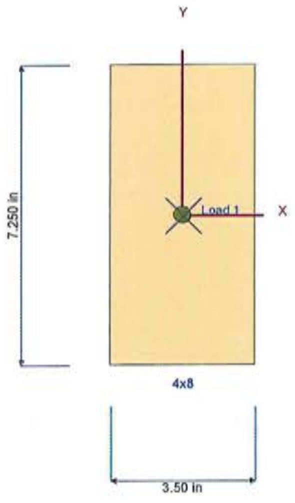
Note: Only non-zero reactions are listed.

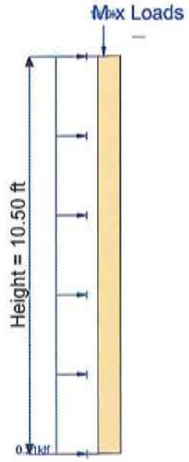
	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
Load Combination	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
D Only						1.548					
+D+Lr						3.048					
+D+S						16.548					
+D+0.750Lr						2.673					
+D+0.750S						12.798					
+D+0.60W				0.646	0.646	1.548					
+D+0.750Lr+0.450W				0.484	0.484	2.673					
+D+0.750S+0.450W				0.484	0.484	12.798					
+0.60D+0.60W				0.646	0.646	0.929					
+0.60D						0.929					
Lr Only						1.500					
S Only						15.000					
W Only				1.076	1.076						

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	0.235 in	5.285 ft
+D+0.750Lr+0.450W	0.0000 in	0.000 ft	0.176 in	5.285 ft
+D+0.750S+0.450W	0.0000 in	0.000 ft	0.176 in	5.285 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.235 in	5.285 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.392 in	5.285 ft

Sketches





Loads are total entered value. Arrows do not reflect absolute direction.

Wood Column

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Lic. # : KW-06002886 **Licensee : RICHMOND HOFFMAYER**

Description : 2ND FLOOR BEARING STUDS @ 48" o.c. - SIP SPLINES

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 10.50 ft
(Used for non-slender calculations)
Wood Species Spruce - Pine - Fir
Wood Grade No.1

Fb +	850.0 psi	Fv	125.0 psi
Fb -	850.0 psi	Ft	550.0 psi
Fc - Prll	700.0 psi	Density	26.210 pcf
Fc - Perp	425.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,300.0	1,300.0	1,300.0 ksi
Minimum	470.0	470.0	

Wood Section Name 2-2x8
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	3.0 in
Exact Depth	7.250 in
Area	21.750 in^2
Ix	95.270 in^4
Iy	16.313 in^4

Allow Stress Modification Factors	
Cf or Cv for Bending	1.20
Cf or Cv for Compression	1.050
Cf or Cv for Tension	1.20
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 10.50 ft, K = 1.0

Applied Loads

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

Column self weight included : 41.567 lbs * Dead Load Factor
AXIAL LOADS . . .
BEARING: Axial Load at 10.50 ft, D = 0.5040, Lr = 0.6390, S = 8.430 k
BENDING LOADS . . .
51.3 PSF WIND: Lat. Uniform Load creating Mx-x, W = 0.1020 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.6013 : 1
	Load Combination	+D+S
	Governing NDS Forumla	Comp Only, fc/Fc'
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Applied Axial	8.976 k
	Applied Mx	0.0 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	686.32 psi
PASS	Maximum Shear Stress Ratio =	0.07386 : 1
	Load Combination	+D+0.60W
	Location of max.above base	0.0 ft
	Applied Design Shear	22.159 psi
	Allowable Shear	200.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.5355 k	Bottom along Y-Y	0.5355 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.2277 in at 5.285 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios Stress Ratio	Status	Location	Maximum Shear Ratios Stress Ratio	Status	Location
D Only	0.900	0.862	0.04401	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+Lr	1.250	0.791	0.07490	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+S	1.150	0.812	0.6013	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.750Lr	1.250	0.791	0.06480	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.750S	1.150	0.812	0.4601	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.60W	1.600	0.719	0.2416	PASS	5.285 ft	0.07386	PASS	0.0 ft
+D+0.750Lr+0.450W	1.600	0.719	0.1868	PASS	5.215 ft	0.05540	PASS	0.0 ft
+D+0.750S+0.450W	1.600	0.719	0.3744	PASS	5.215 ft	0.05540	PASS	0.0 ft
+0.60D+0.60W	1.600	0.719	0.2391	PASS	5.285 ft	0.07386	PASS	0.0 ft

Wood Column

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Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : 2ND FLOOR BEARING STUDS @ 48" o.c. - SIP SPLINES

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios Stress Ratio	Status	Location	Maximum Shear Ratios Stress Ratio	Status	Location
+0.60D	1.600	0.719	0.01779	PASS	0.0 ft	0.0	PASS	10.50 ft

Maximum Reactions

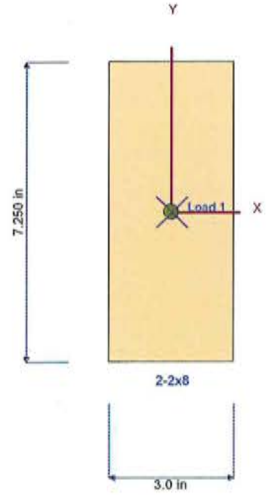
Note: Only non-zero reactions are listed.

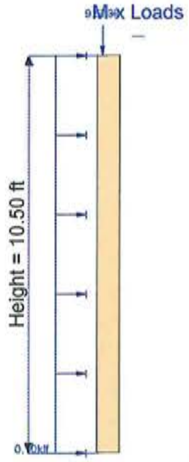
Load Combination	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only					0.546				
+D+Lr					1.185				
+D+S					8.976				
+D+0.750Lr					1.025				
+D+0.750S					6.868				
+D+0.60W			0.321	0.321	0.546				
+D+0.750Lr+0.450W			0.241	0.241	1.025				
+D+0.750S+0.450W			0.241	0.241	6.868				
+0.60D+0.60W			0.321	0.321	0.327				
+0.60D					0.327				
Lr Only					0.639				
S Only					8.430				
W Only			0.536	0.535					

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	0.137 in	5.285 ft
+D+0.750Lr+0.450W	0.0000 in	0.000 ft	0.102 in	5.285 ft
+D+0.750S+0.450W	0.0000 in	0.000 ft	0.102 in	5.285 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.137 in	5.285 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.228 in	5.285 ft

Sketches





Loads are total entered value. Arrows do not reflect absolute direction.

Steel Column

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ENERCALC, INC. 1983-2017, Build:6.17.3.17, Ver:6.17.3.17

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : RIDGE SUPPORT

Code References

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Steel Section Name :	**Pipe4 Std**
Analysis Method :	Allowable Strength
Steel Stress Grade	A-36, Carbon Steel, Fy = 36 ksi
Fy : Steel Yield	36.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height 10.0 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Fully braced against buckling along X-X Axis
Y-Y (depth) axis :
Fully braced against buckling along Y-Y Axis

Applied Loads

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

Column self weight included : 108.581 lbs * Dead Load Factor
AXIAL LOADS . . .
MAX ROOF: Axial Load at 10.0 ft, D = 3.110, LR = 3.390, S = 42.460 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.7135 : 1
	Load Combination	+D+S
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Pa : Axial	45.679 k
	Pn / Omega : Allowable	64.024 k
	Ma-x : Applied	0.0 k-ft
	Mn-x / Omega : Allowable	7.275 k-ft
	Ma-y : Applied	0.0 k-ft
	Mn-y / Omega : Allowable	7.275 k-ft
PASS	Maximum Shear Stress Ratio =	0.0 : 1
	Load Combination	
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Va : Applied	0.0 k
	Vn / Omega : Allowable	0.0 k

Maximum SERVICE Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum SERVICE Load Deflections . . .

Along Y-Y 0.0 in at 0.0ft above base
for load combination :

Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
Load Combination	Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.050	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+Lr	0.103	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+S	0.713	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750Lr	0.090	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750S	0.548	PASS	0.00 ft	0.000	PASS	0.00 ft
+0.60D	0.030	PASS	0.00 ft	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

	Axial Reaction	X-X Axis Reaction k		Y-Y Axis Reaction		Mx - End Moments k-ft		My - End Moments	
Load Combination	@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
D Only	3.219								
+D+Lr	6.609								
+D+S	45.679								
+D+0.750Lr	5.761								
+D+0.750S	35.064								
+0.60D	1.931								
Lr Only	3.390								
S Only	42.460								

Steel Column

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Description : RIDGE SUPPORT

Extreme Reactions

Item	Extreme Value	Axial Reaction	X-X Axis Reaction		k Y-Y Axis Reaction		Mx - End Moments	k-ft	My - End Moments	
		@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
Axial @ Base	Maximum	45.679								
"	Minimum	1.931								
Reaction, X-X Axis	Maximum	3.219								
"	Minimum	3.219								
Reaction, Y-Y Axis	Maximum	3.219								
"	Minimum	3.219								
Reaction, X-X Axis	Maximum	3.219								
"	Minimum	3.219								
Reaction, Y-Y Axis	Maximum	3.219								
"	Minimum	3.219								
Moment, X-X Axis Ba	Maximum	3.219								
"	Minimum	3.219								
Moment, Y-Y Axis Ba	Maximum	3.219								
"	Minimum	3.219								
Moment, X-X Axis To	Maximum	3.219								
"	Minimum	3.219								
Moment, Y-Y Axis To	Maximum	3.219								
"	Minimum	3.219								

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
Lr Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
S Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Steel Section Properties : Pipe4 Std

Depth	=	4.500 in	I xx	=	6.82 in^4	J	=	13.600 in^4
			S xx	=	3.03 in^3			
Diameter	=	4.500 in	R xx	=	1.510 in			
Wall Thick	=	0.237 in	Zx	=	4.050 in^3			
Area	=	2.970 in^2	I yy	=	6.820 in^4			
Weight	=	10.858 plf	S yy	=	3.030 in^3			
			R yy	=	1.510 in			

Ycg = 0.000 in

Steel Column

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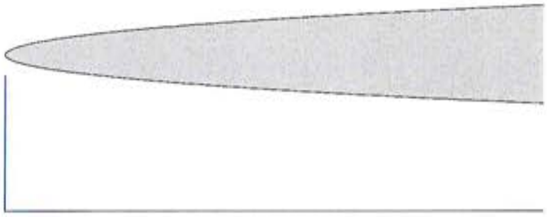
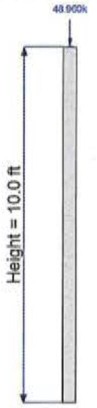
Description : RIDGE SUPPORT





Loads are total entered value. Arrows do not reflect absolute direction.

Steel Column

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Description : B5 SUPPORT

Code References

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Steel Section Name :	**HSS12x6x5/8**
Analysis Method :	Allowable Strength
Steel Stress Grade	
Fy : Steel Yield	46.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height 10.0 ft
Top & Bottom Fixity **Top & Bottom Pinned**
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 10.0 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for Y-Y Axis buckling = 10.0 ft, K = 2.1

Applied Loads

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

Column self weight included : 676.18 lbs * Dead Load Factor
AXIAL LOADS . . .
B5: Axial Load at 10.0 ft, D = 3.350, LR = 1.840, S = 49.110 k

< 0.15 ✓ USE FOR LATERAL AS OPTION

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.1324 : 1
	Load Combination	+D+S
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Pa : Axial	53.136 k
	Pn / Omega : Allowable	401.460 k
	Ma-x : Applied	0.0 k-ft
	Mn-x / Omega : Allowable	157.924 k-ft
	Ma-y : Applied	0.0 k-ft
	Mn-y / Omega : Allowable	96.637 k-ft
PASS	Maximum Shear Stress Ratio =	0.0 : 1
	Load Combination	
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Va : Applied	0.0 k
	Vn / Omega : Allowable	0.0 k

Maximum SERVICE Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum SERVICE Load Deflections . . .

Along Y-Y 0.0 in at 0.0ft above base
for load combination :

Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
Load Combination	Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.010	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+Lr	0.015	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+S	0.132	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750Lr	0.013	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750S	0.102	PASS	0.00 ft	0.000	PASS	0.00 ft
+0.60D	0.006	PASS	0.00 ft	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

	Axial Reaction	X-X Axis Reaction	k	Y-Y Axis Reaction		Mx - End Moments	k-ft	My - End Moments	
Load Combination	@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
D Only	4.026								
+D+Lr	5.866								
+D+S	53.136								
+D+0.750Lr	5.406								
+D+0.750S	40.859								
+0.60D	2.416								
Lr Only	1.840								
S Only	49.110								

Steel Column

Description : B5 SUPPORT

Extreme Reactions

Item	Extreme Value	Axial Reaction	X-X Axis Reaction		k	Y-Y Axis Reaction		Mx - End Moments		k-ft	My - End Moments	
		@ Base	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top		@ Base	@ Top
Axial @ Base	Maximum	53.136										
"	Minimum	1.840										
Reaction, X-X Axis	Maximum	4.026										
"	Minimum	4.026										
Reaction, Y-Y Axis	Maximum	4.026										
"	Minimum	4.026										
Reaction, X-X Axis	Maximum	4.026										
"	Minimum	4.026										
Reaction, Y-Y Axis	Maximum	4.026										
"	Minimum	4.026										
Moment, X-X Axis Ba	Maximum	4.026										
"	Minimum	4.026										
Moment, Y-Y Axis Ba	Maximum	4.026										
"	Minimum	4.026										
Moment, X-X Axis To	Maximum	4.026										
"	Minimum	4.026										
Moment, Y-Y Axis To	Maximum	4.026										
"	Minimum	4.026										

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
Lr Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
S Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Steel Section Properties : HSS12x6x5/8

Depth	=	12.000 in	I xx	=	321.00 in^4	J	=	271.000 in^4
Design Thick	=	0.581 in	S xx	=	53.40 in^3	Cw	=	71.10 in^6
Width	=	6.000 in	R xx	=	4.140 in			
Wall Thick	=	0.624 in	Zx	=	68.800 in^3			
Area	=	18.700 in^2	I yy	=	107.000 in^4	C	=	71.100 in^3
Weight	=	67.618 plf	S yy	=	35.500 in^3			
			R yy	=	2.390 in			
			Zy	=	42.100 in^3			

Ycg = 0.000 in

Steel Column

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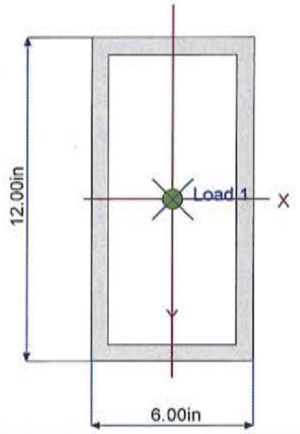
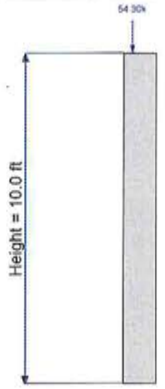
Description : B5 SUPPORT

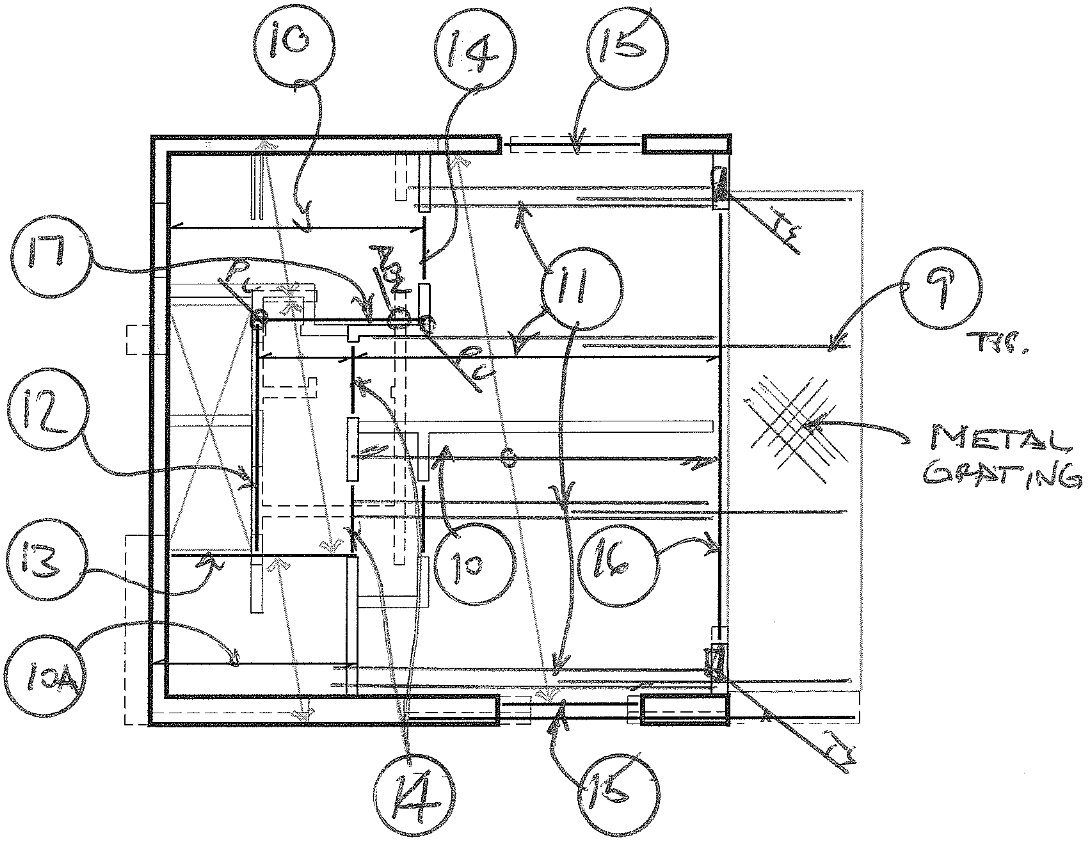




Loads are total entered value. Arrows do not reflect absolute direction.

RICHMOND HOFFMAYER INC.
STRUCTURAL ENGINEERING

PROJECT	FELDSPAR LOWER	JOB	2017-0610
CLIENT	VILLAGE NEST		
ADDRESS	EDEN, UTAH		
DESIGN	HUUM		
ENGINEER	J.H.	DATE	6/16/2017
	RICHMOND PE 49628 HOFFMAYER SE 3935	SHEET	30 OF


10
14
15
17
PL
DBL
PL
11
9
TYP.
METAL GRATING
12
10
16
13
10A
14
15

2ND FLOOR FRAMING PLAN

FELDSPAR - LOWER

N.T.S.

○ : BEAM #

PROJECT	VILLAGE NEST	**JOB**	2017-0610
CLIENT	-		
ADDRESS	EDEN, UT		
DESIGN	HUUM		
ENGINEER	J.H.	**DATE**	6/14/2017
	RICHMOND PE 44628 HOFFMAYER SE 3495	**SHEET**	31 **OF**

METAL GRATING:

D = 12 PSF
L = 60 PSF
S = 264 PSF – (OPENED GRATING – DON'T DBL SNOW)

LOAD COMBINATIONS:

D + L = 72 PSF
D + S = 276 PSF ← GOVERNS
D + 0.75L + 0.75S = 266 PSF

SPAN	$M = wl^2/8$ (1 - SPAN OR 2 - SPANS)
32"	2944 IN-LB
48"	6624 IN-LB
64"	11776 IN-LB

MAT.	ALUMINUM	STEEL
E =	10000 KSI	29000 KSI
F =	12000 PSI	18000 PSI MIN

SPAN IN INCHES

$FSK > M$ & $EIK > 5wl^3/4608 \times 240$

↑ BARS/FT ↑ 276 PSF

$K = 12/(7/16) = 27.4$

$12000(S)27.4 > 2944 \rightarrow S = 0.009\ IN^3$
$12000(S)27.4 > 6624 \rightarrow S = 0.020\ IN^3$
$12000(S)27.4 > 11776 \rightarrow S = 0.035\ IN^3$
} AL

PROJECT VILLAGE NEST JOB 2017-0610
CLIENT -
ADDRESS EDEN, UT
DESIGN HUUM
ENGINEER J.H. DATE 6/14/2017
RICHMOND PE 44628 HOFFMAYER SE 3455 SHEET 31 OF

$12/18(0.009) = 0.006\text{ IN}^3$
$12/18(0.020) = 0.0133\text{ IN}^3$
$12/18(0.035) = 0.0233\text{ IN}^3$
} STEEL.

$IK > 5(276)\ell^3/4608E$

$I > 0.0109\ell^3/E$

$0.0109(32^3)/10E6 = 0.00004\text{ IN}^4$
$0.0109(48^3)/10E6 = 0.0001\text{ IN}^4$
$0.0109(64^3)/10E6 = 0.0003\text{ IN}^4$
} AL

$10/29(0.00004) = 0.000013\text{ IN}^4$
$10/29(0.0001) = 0.000035\text{ IN}^4$
$10/29(0.0003) = 0.0001\text{ IN}^4$
} STEEL

1" DEEP × 3/16" AL → $S = 0.0313\text{ IN}^3$
$I = 0.0156\text{ IN}^4$
* USE UP TO 4-8" SPANS W/ BARS @ 7/16"

1" DEEP × 3/16" STEEL → $S = 0.0313\text{ IN}^3$
$I = 0.0156\text{ IN}^4$
* USE UP TO 64" SPANS W/ BARS @ 7/16"

1" × 3/16" AL @ 7/16" ± GRATING
W/ SPANS @ 48" ±

Steel Beam

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Description : B9 - DECK SUPPORT

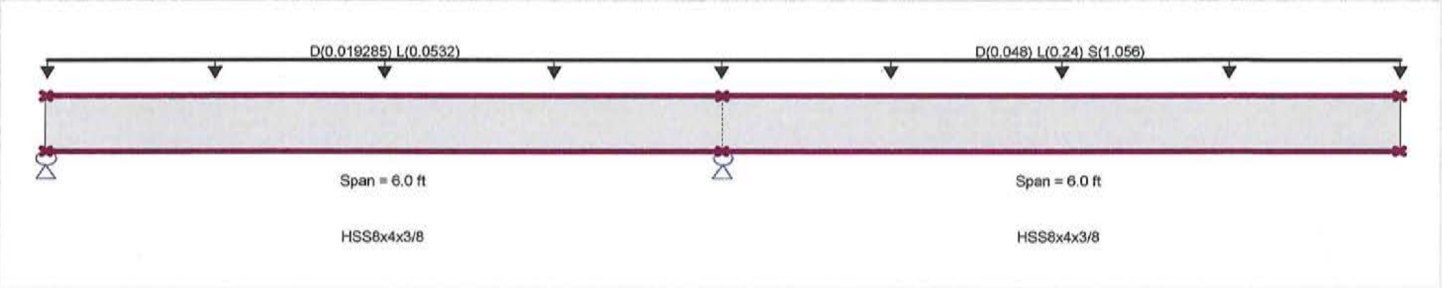
CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.01450, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)
Load for Span Number 2
Uniform Load : D = 0.0120, L = 0.060, S = 0.2640 ksf, Tributary Width = 4.0 ft, (DECK)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.434 : 1	**Maximum Shear Stress Ratio =**	0.078 : 1
Section used for this span	**HSS8x4x3/8**	Section used for this span	**HSS8x4x3/8**
Ma : Applied	20.367 k-ft	Va : Applied	6.789 k
Mn / Omega : Allowable	46.906 k-ft	Vn/Omega : Allowable	87.183 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	6.000ft	Location of maximum on span	6.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.089 in	Ratio =	1,616 >=360.
Max Upward Transient Deflection	-0.009 in	Ratio =	7,748 >=360.
Max Downward Total Deflection	0.397 in	Ratio =	363 >=240.
Max Upward Total Deflection	-0.043 in	Ratio =	1675 >=240.

Maximum Forces & Stresses for Load Combinations

Load Combination Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 6.00 ft	1	0.029	0.005		-1.36	1.36	78.33	46.91	1.00	1.00	0.45	145.60	87.18
Dsgn. L = 6.00 ft	2	0.029	0.005		-1.36	1.36	78.33	46.91	1.00	1.00	0.45	145.60	87.18
+D+L													
Dsgn. L = 6.00 ft	1	0.121	0.022		-5.68	5.68	78.33	46.91	1.00	1.00	1.89	145.60	87.18
Dsgn. L = 6.00 ft	2	0.121	0.022		-5.68	5.68	78.33	46.91	1.00	1.00	1.89	145.60	87.18
+D+S													
Dsgn. L = 6.00 ft	1	0.434	0.078		-20.37	20.37	78.33	46.91	1.00	1.00	6.79	145.60	87.18
Dsgn. L = 6.00 ft	2	0.434	0.078		-20.37	20.37	78.33	46.91	1.00	1.00	6.79	145.60	87.18
+D+0.750L													
Dsgn. L = 6.00 ft	1	0.098	0.018		-4.60	4.60	78.33	46.91	1.00	1.00	1.53	145.60	87.18
Dsgn. L = 6.00 ft	2	0.098	0.018		-4.60	4.60	78.33	46.91	1.00	1.00	1.53	145.60	87.18
+D+0.750L+0.750S													
Dsgn. L = 6.00 ft	1	0.402	0.072		-18.85	18.85	78.33	46.91	1.00	1.00	6.28	145.60	87.18
Dsgn. L = 6.00 ft	2	0.402	0.072		-18.85	18.85	78.33	46.91	1.00	1.00	6.28	145.60	87.18
+0.60D													
Dsgn. L = 6.00 ft	1	0.017	0.003		-0.82	0.82	78.33	46.91	1.00	1.00	0.27	145.60	87.18
Dsgn. L = 6.00 ft	2	0.017	0.003		-0.82	0.82	78.33	46.91	1.00	1.00	0.27	145.60	87.18

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+0.750L+0.750S	-0.0430	3.504

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Steel Beam

Description : B9 - DECK SUPPORT

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S	2	0.3967	6.000		0.0000	3.504

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	-3.254	10.324	
Overall MINimum	-0.052	0.492	
D Only	-0.086	0.820	
+D+L	-0.647	3.139	
+D+S	-3.254	10.324	
+D+0.750L	-0.506	2.559	
+D+0.750L+0.750S	-2.882	9.687	
+0.60D	-0.052	0.492	
L Only	-0.560	2.320	
S Only	-3.168	9.504	

Wood Beam

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Licensee : RICHMOND HOFFMAYER

Description : B10 - FLOOR JOISTS w/ PARTITION

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Spruce - Pine - Fir
Wood Grade : No. 1/No. 2

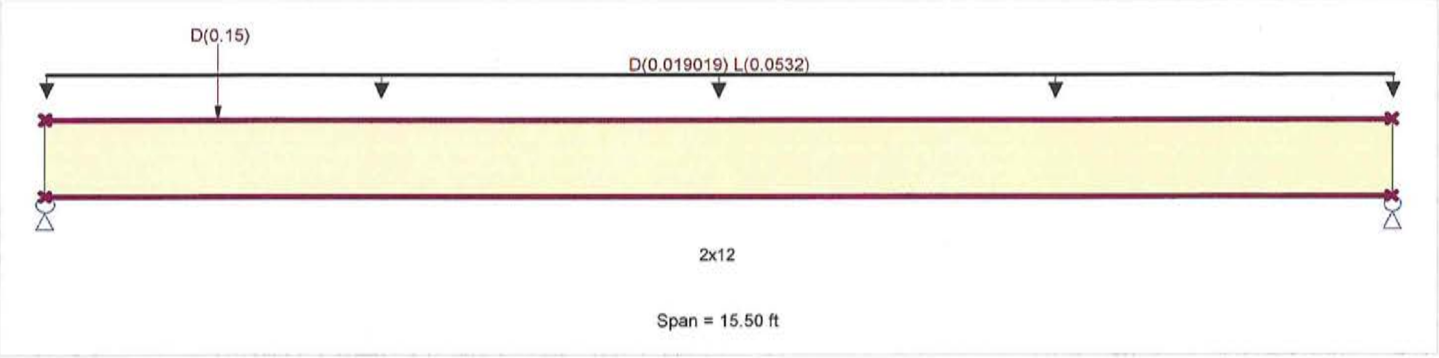
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb - Tension	875.0 psi
Fb - Compr	875.0 psi
Fc - Prll	1,150.0 psi
Fc - Perp	425.0 psi
Fv	135.0 psi
Ft	450.0 psi

E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	26.210 pcf

Repetitive Member Stress Increase



Applied Loads

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

Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)
Point Load : D = 0.150 k @ 2.0 ft, (WALL)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.875: 1	**Maximum Shear Stress Ratio**	=	0.412 : 1
Section used for this span		2x12	Section used for this span		2x12
fb : Actual	=	880.42psi	fv : Actual	=	55.55 psi
FB : Allowable	=	1,006.25psi	Fv : Allowable	=	135.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	7.467ft	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	0.279 in	Ratio =	666 >=360.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360.0
Max Downward Total Deflection	0.409 in	Ratio =	454 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.690	0.579
Overall MINimum	0.167	0.100
D Only	0.278	0.167
+D+L	0.690	0.579
+D+0.750L	0.587	0.476
+0.60D	0.167	0.100
L Only	0.412	0.412

35A

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Wood Beam

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Description : B10A - FLOOR JOISTS ABV. LANDING

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Spruce - Pine - Fir
Wood Grade : No. 1/No. 2

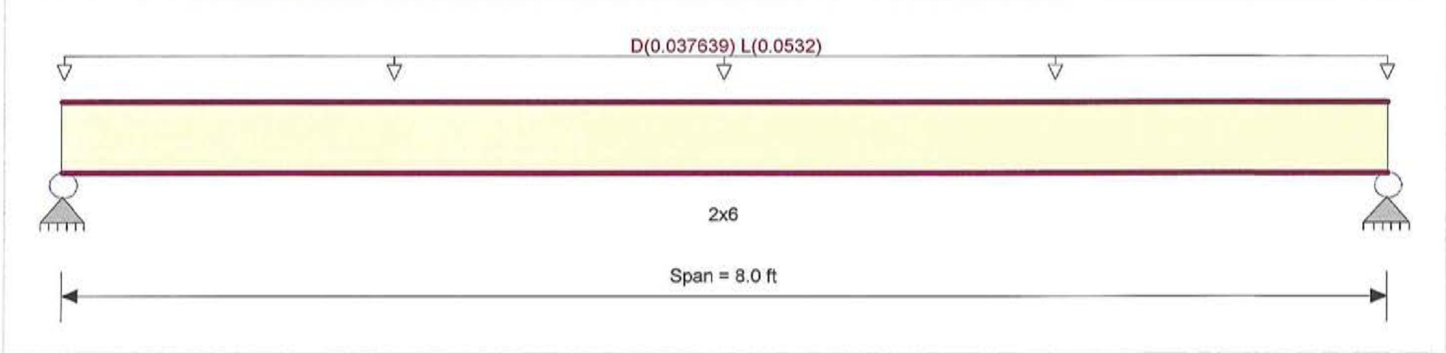
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	875.0 psi
Fb -	875.0 psi
Fc - Prll	1,150.0 psi
Fc - Perp	425.0 psi
Fv	135.0 psi
Ft	450.0 psi

E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	26.210 pcf

Repetitive Member Stress Increase



Applied Loads

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

Uniform Load : D = 0.02830, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.882 1	**Maximum Shear Stress Ratio**	=	0.436 : 1
Section used for this span		2x6	Section used for this span		2x6
fb : Actual	=	1,153.13psi	fv : Actual	=	58.83 psi
FB : Allowable	=	1,308.13psi	Fv : Allowable	=	135.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	4.000ft	Location of maximum on span	=	7.562ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.169 in	Ratio =	566 >=360.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360.0
Max Downward Total Deflection	0.289 in	Ratio =	331 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.363	0.363
Overall MINimum	0.090	0.090
D Only	0.151	0.151
+D+L	0.363	0.363
+D+0.750L	0.310	0.310
+0.60D	0.090	0.090
L Only	0.213	0.213

Wood Beam

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Description : B11 - FLOOR JOISTS w/ DECK SUPPORT

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

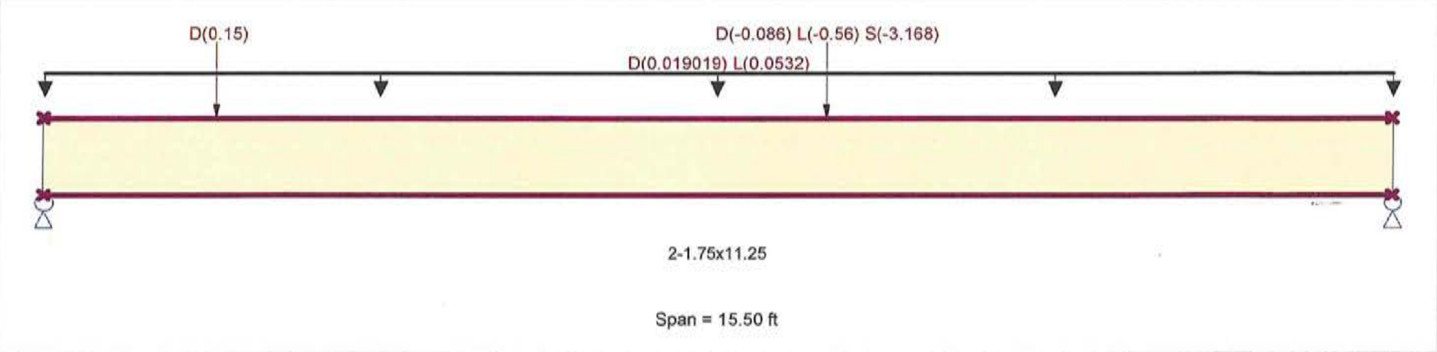
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb - Tension	2600 psi
Fb - Compr	2600 psi
Fc - Prll	2510 psi
Fc - Perp	750 psi
Fv	285 psi
Ft	1555 psi

E : Modulus of Elasticity

Ebend- xx	1900 ksi
Eminbend - xx	965.71 ksi
Density	42 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 = 0.01430, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)
Point Load : D = 0.150 k @ 2.0 ft, (WALL)
Point Load : D = -0.0860, L = -0.560, S = -3.168 k @ 9.0 ft, (B9)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.532 1	**Maximum Shear Stress Ratio**	=	0.213 : 1
Section used for this span		2-1.75x11.25	Section used for this span		2-1.75x11.25
fb : Actual	=	1,829.45psi	fv : Actual	=	69.73 psi
FB : Allowable	=	3,438.50psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.995ft	Location of maximum on span	=	9.051 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.002 in	Ratio =	92810 >=360.
Max Upward Transient Deflection	-0.005 in	Ratio =	33909 >=360.
Max Downward Total Deflection	0.046 in	Ratio =	4026 >=240.
Max Upward Total Deflection	-0.350 in	Ratio =	531 >=240.

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	-1.329	-1.839
Overall MINimum	0.177	0.087
D Only	0.331	0.206
+D+L	0.508	0.293
+D+S	-0.998	-1.634
+D+0.750L	0.464	0.271
+D+0.750L+0.750S	-0.532	-1.108
+0.60D	0.199	0.123
L Only	0.177	0.087
S Only	-1.329	-1.839

BOLT:

$R = 3254\,lb$

↑ B9

TRY NDS TABLE 11B

SINGLE SHEAR ×2

$t_m = 1\text{-}3/4''$

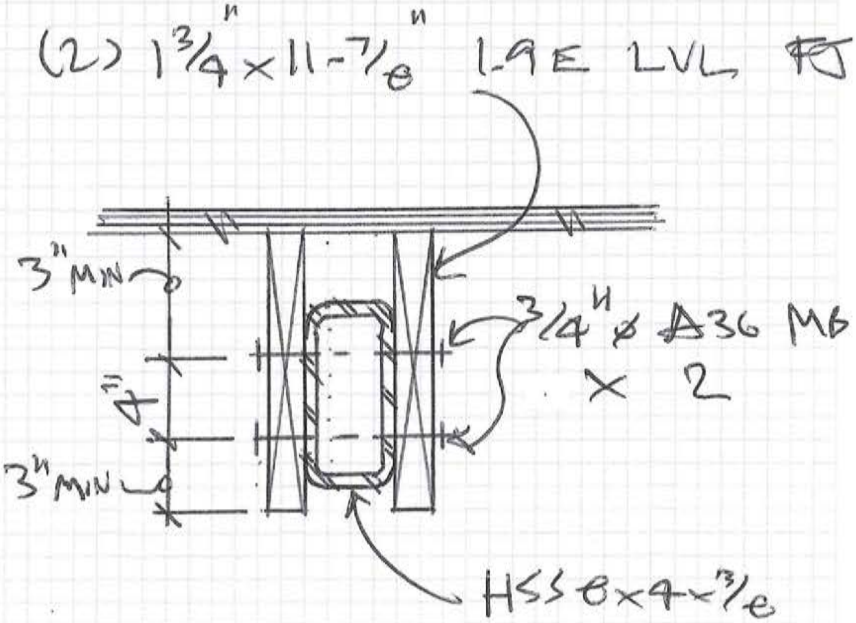
$t_s \geq 1/4''$

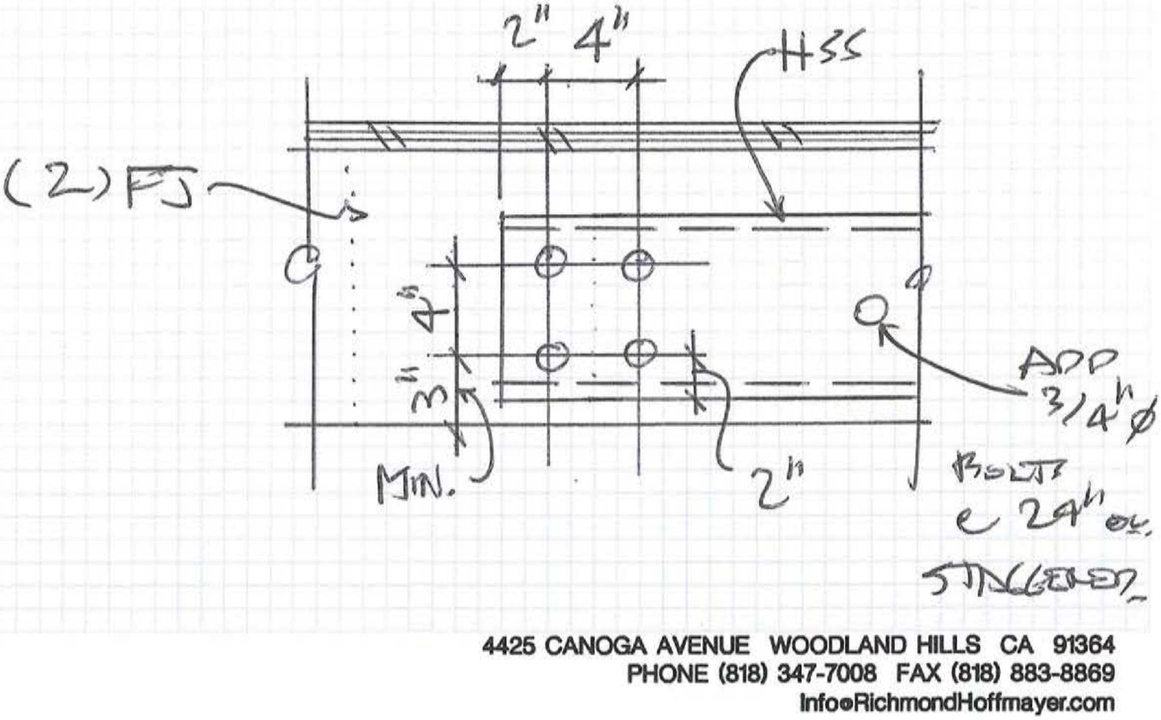
$G = 0.49$

$Z_\perp = 430\,lb$ FOR 3/4" ø

$V_{CAP} = 2(430) = \underline{\underline{860\,lb}}$

$\# BOLTS = 3254/860 = 3.78 \rightarrow$ (4) BOLTS MIN.


(2) 1 3/4" × 11-7/8" 1.9E LVL FJ
3" MIN
4"
3" MIN
3/4" ø A36 MB × 2
HSS 8×4×3/8


2" 4"
HSS
(2) FJ
4"
3"
MIN.
2"
ADD 3/4" ø BOLTS @ 24" o.c. STAGGERED

3e

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Wood Beam

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Description : B11 - FLOOR JOISTS w/ DECK SUPPORT

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

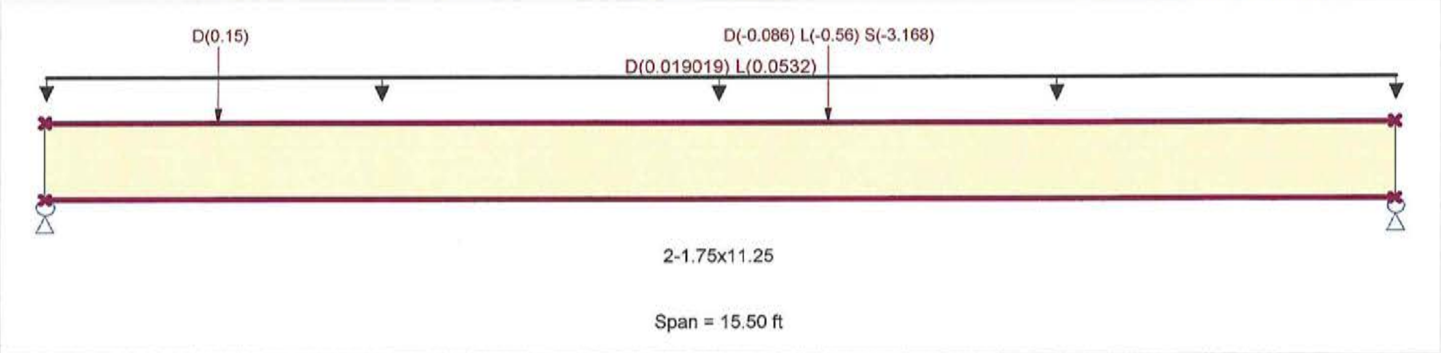
Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

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

Fb - Tension	2600 psi	
Fb - Compr	2600 psi	
Fc - Prll	2510 psi	
Fc - Perp	750 psi	
Fv	285 psi	
Ft	1555 psi	

E : Modulus of Elasticity
Ebend- xx 1900 ksi
Eminbend - xx 965.71 ksi

Density 42 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 = 0.01430, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)
Point Load : D = 0.150 k @ 2.0 ft, (WALL)
Point Load : D = -0.0860, L = -0.560, S = -3.168 k @ 9.0 ft, (B9)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.532 1	Maximum Shear Stress Ratio	=	0.213 : 1
Section used for this span		2-1.75x11.25	Section used for this span		2-1.75x11.25
fb : Actual	=	1,829.45psi	fv : Actual	=	69.73 psi
FB : Allowable	=	3,438.50psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.995ft	Location of maximum on span	=	9.051 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.002 in	Ratio =	92810 >=360.
Max Upward Transient Deflection	-0.005 in	Ratio =	33909 >=360.
Max Downward Total Deflection	0.046 in	Ratio =	4026 >=240.
Max Upward Total Deflection	-0.350 in	Ratio =	531 >=240.

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	-1.329	-1.839
Overall MINimum	0.177	0.087
D Only	0.331	0.206
+D+L	0.508	0.293
+D+S	-0.998	-1.634
+D+0.750L	0.464	0.271
+D+0.750L+0.750S	-0.532	-1.108
+0.60D	0.199	0.123
L Only	0.177	0.087
S Only	-1.329	-1.839

Wood Beam

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Description : B12 - FLOOR SUPPORT

CODE REFERENCES

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

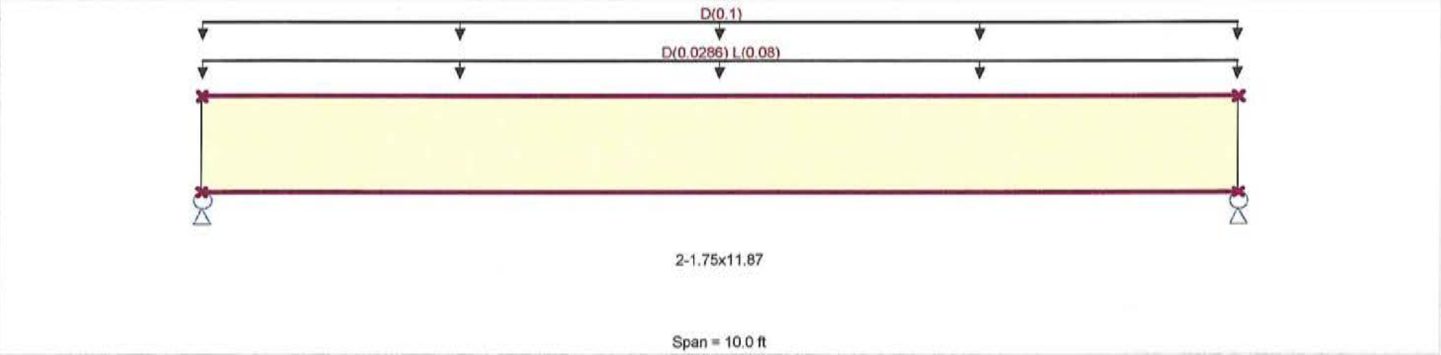
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

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

Fb - Tension	2600 psi	*E : Modulus of Elasticity*	
Fb - Compr	2600 psi	Ebend- xx	1900 ksi
Fc - Prll	2510 psi	Eminbend - xx	965.71 ksi
Fc - Perp	750 psi		
Fv	285 psi		
Ft	1555 psi	Density	42 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 2.0 ft, (FLOOR)
Uniform Load : D = 0.10 , Tributary Width = 1.0 ft, (WALL)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.155 1	Maximum Shear Stress Ratio	=	0.112 : 1
Section used for this span		2-1.75x11.87	Section used for this span		2-1.75x11.87
fb : Actual	=	402.49psi	fv : Actual	=	31.98 psi
FB : Allowable	=	2,600.00psi	Fv : Allowable	=	285.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	5.000ft	Location of maximum on span	=	0.000ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.020 in	Ratio =	6150>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0<360
Max Downward Total Deflection	0.054 in	Ratio =	2229>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0<240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.104	1.104
Overall MINimum	0.400	0.400
D Only	0.704	0.704
+D+L	1.104	1.104
+D+0.750L	1.004	1.004
+0.60D	0.422	0.422
L Only	0.400	0.400

Wood Beam

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Description : B13 - FLOOR SUPPORT ABV. STAIR

CODE REFERENCES

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

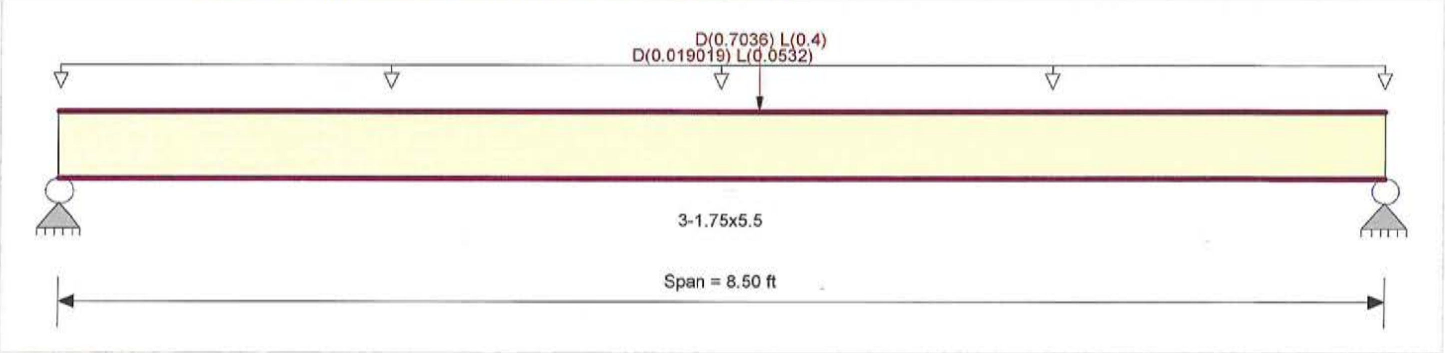
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

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

Fb +	2,600.0 psi	*E : Modulus of Elasticity*	
Fb -	2,600.0 psi	Ebend- xx	1,900.0 ksi
Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Fc - Perp	750.0 psi		
Fv	285.0 psi		
Ft	1,555.0 psi	Density	42.0 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)
Point Load : D = 0.7036, L = 0.40 k @ 4.50 ft, (B12)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.534 1	**Maximum Shear Stress Ratio**	=	0.163 : 1
Section used for this span		3-1.75x5.5	Section used for this span		3-1.75x5.5
fb : Actual	=	1,388.15psi	fv : Actual	=	46.34 psi
FB : Allowable	=	2,600.00psi	Fv : Allowable	=	285.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	4.498ft	Location of maximum on span	=	8.066 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.109 in	Ratio =	932>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0<360
Max Downward Total Deflection	0.245 in	Ratio =	415>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0<240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.862	0.927
Overall MINimum	0.269	0.293
D Only	0.448	0.489
+D+L	0.862	0.927
+D+0.750L	0.758	0.818
+0.60D	0.269	0.293
L Only	0.414	0.438

41

Wood Beam

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Description : B14 - HEADERS

CODE REFERENCES

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

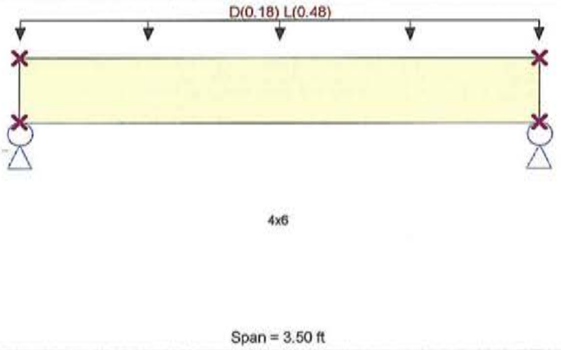
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Spruce - Pine - Fir
Wood Grade : No. 1/No. 2

Beam Bracing : Completely Unbraced

Fb - Tension	875 psi	*E : Modulus of Elasticity*	
Fb - Compr	875 psi	Ebend- xx	1400 ksi
Fc - Prll	1150 psi	Eminbend - xx	510 ksi
Fc - Perp	425 psi		
Fv	135 psi		
Ft	450 psi	Density	26.21 pcf



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, L = 0.040 ksf, Tributary Width = 12.0 ft, (FLOOR)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.610 : 1**	**Maximum Shear Stress Ratio**	=	**0.499 : 1**
Section used for this span		**4x6**	Section used for this span		**4x6**
fb : Actual	=	690.92psi	fv : Actual	=	67.36 psi
FB : Allowable	=	1,133.19psi	Fv : Allowable	=	135.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	1.750ft	Location of maximum on span	=	3.053 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.024 in	Ratio =	1750 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.033 in	Ratio =	1266 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.161	1.161
Overall MINimum	0.193	0.193
D Only	0.321	0.321
+D+L	1.161	1.161
+D+0.750L	0.951	0.951
+0.60D	0.193	0.193
L Only	0.840	0.840

42

Wood Beam

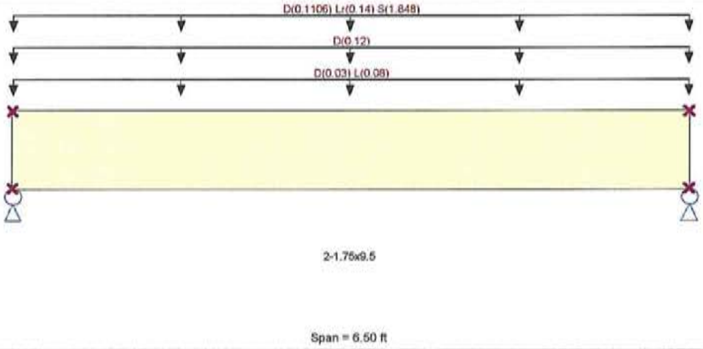
Description : B15 - HEADERS

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb - Tension	2600 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-10	Fb - Compr	2600 psi	Ebend- xx	1900 ksi
		Fc - Prll	2510 psi	Eminbend - xx	965.71 ksi
Wood Species	: Trus Joist	Fc - Perp	750 psi		
Wood Grade	: MicroLam LVL 1.9 E	Fv	285 psi		
		Ft	1555 psi	Density	42 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, L = 0.040 ksf, Tributary Width = 2.0 ft, (FLOOR)
Uniform Load : D = 0.120 , Tributary Width = 1.0 ft, (EXTERIOR WALL)
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 7.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.872 1	**Maximum Shear Stress Ratio**	=	0.719 : 1
Section used for this span		2-1.75x9.5	Section used for this span		2-1.75x9.5
fb : Actual	=	2,550.00psi	fv : Actual	=	235.77 psi
FB : Allowable	=	2,925.88psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.250ft	Location of maximum on span	=	5.717 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.157 in	Ratio = 496 >=360
Max Upward Transient Deflection	0.000 in	Ratio = 0 <360
Max Downward Total Deflection	0.180 in	Ratio = 433 >=240.
Max Upward Total Deflection	0.000 in	Ratio = 0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	6.884	6.884
Overall MINimum	0.260	0.260
D Only	0.878	0.878
+D+L	1.138	1.138
+D+Lr	1.333	1.333
+D+S	6.884	6.884
+D+0.750Lr+0.750L	1.415	1.415
+D+0.750L+0.750S	5.578	5.578
+0.60D	0.527	0.527
Lr Only	0.455	0.455

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Wood Beam

Description : B15 - HEADERS

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
L Only	0.260	0.260
S Only	6.006	6.006

Steel Beam

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ENERCALC, INC. 1983-2017, Build:6.17.3.17, Ver:6.17.3.17

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Description : B16 - BALCONY SUPPORT

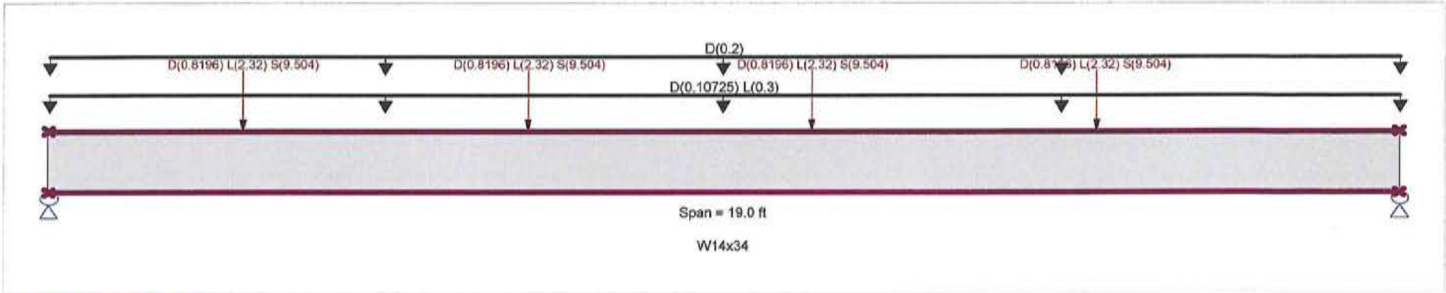
CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi


D(0.8196) L(2.32) S(9.504)
D(0.8196) L(2.32) S(9.504)
D(0.2)
D(0.8196) L(2.32) S(9.504)
D(0.8196) L(2.32) S(9.504)
D(0.10725) L(0.3)
Span = 19.0 ft
W14x34

Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 7.50 ft, (FLOOR)
Uniform Load : D = 0.20 k/ft, Tributary Width = 1.0 ft, (GLASS)
Point Load : D = 0.8196, L = 2.320, S = 9.504 k @ 2.750 ft, (BALCONY - B9)
Point Load : D = 0.8196, L = 2.320, S = 9.504 k @ 6.750 ft, (BALCONY - B9)
Point Load : D = 0.8196, L = 2.320, S = 9.504 k @ 10.750 ft, (BALCONY - B9)
Point Load : D = 0.8196, L = 2.320, S = 9.504 k @ 14.750 ft, (BALCONY - B9)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.981 : 1	Maximum Shear Stress Ratio =	0.329 : 1
Section used for this span	W14x34	Section used for this span	W14x34
Ma : Applied	133.584 k-ft	Va : Applied	26.284 k
Mn / Omega : Allowable	136.228 k-ft	Vn/Omega : Allowable	79.80 k
Load Combination	+D+0.750L+0.750S	Load Combination	+D+0.750L+0.750S
Location of maximum on span	10.749ft	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	0.259 in	Ratio = 878 >=360
Max Upward Transient Deflection	0.000 in	Ratio = 0 <360
Max Downward Total Deflection	0.421 in	Ratio = 541 >=240.
Max Upward Total Deflection	0.000 in	Ratio = 0 <240.0

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 19.00 ft	1	0.179	0.063	24.44		24.44	227.50	136.23	1.00	1.00	5.01	119.70	79.80
+D+L													
Dsgn. L = 19.00 ft	1	0.467	0.161	63.66		63.66	227.50	136.23	1.00	1.00	12.87	119.70	79.80
+D+S													
Dsgn. L = 19.00 ft	1	0.960	0.320	130.73		130.73	227.50	136.23	1.00	1.00	25.52	119.70	79.80
+D+0.750L													
Dsgn. L = 19.00 ft	1	0.395	0.137	53.85		53.85	227.50	136.23	1.00	1.00	10.90	119.70	79.80
+D+0.750L+0.750S													
Dsgn. L = 19.00 ft	1	0.981	0.329	133.58		133.58	227.50	136.23	1.00	1.00	26.28	119.70	79.80
+0.60D													
Dsgn. L = 19.00 ft	1	0.108	0.038	14.66		14.66	227.50	136.23	1.00	1.00	3.01	119.70	79.80

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.4214	9.500		0.0000	0.000

Vertical Reactions

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

Load Combination	Support 1	Support 2
Overall MAXimum	26.284	23.225

Steel Beam

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Description : B16 - BALCONY SUPPORT

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MINimum	3.006	2.851
D Only	5.010	4.752
+D+L	12.867	11.875
+D+S	25.519	22.259
+D+0.750L	10.903	10.094
+D+0.750L+0.750S	26.284	23.225
+0.60D	3.006	2.851
L Only	7.856	7.124
S Only	20.509	17.507

Steel Beam

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Licensee : RICHMOND HOFFMAYER

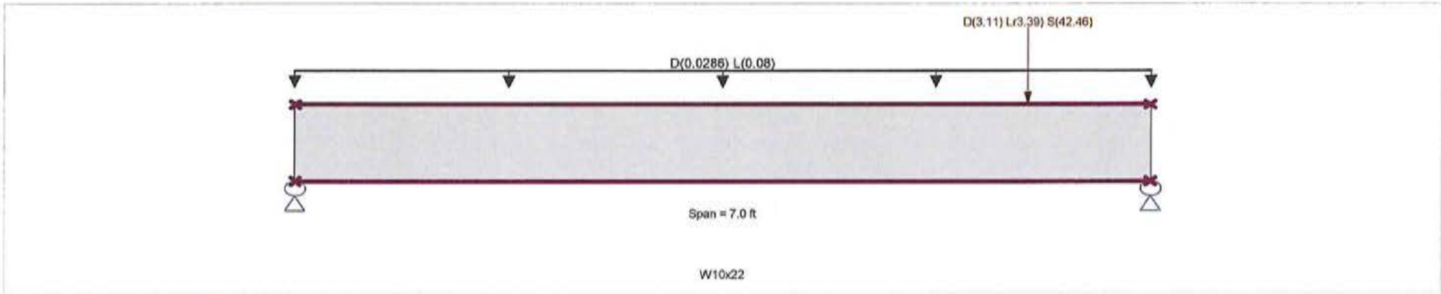
Description : B17 - RIDGE SUPPORT

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 2.0 ft, (FLOOR)
Point Load : D = 3.110, Lr = 3.390, S = 42.460 k @ 6.0 ft, (RIDGE B4 ABV.)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.604 : 1**	**Maximum Shear Stress Ratio =**	**0.801 : 1**
Section used for this span	**W10x22**	Section used for this span	**W10x22**
Ma : Applied	39.212 k-ft	Va : Applied	39.237 k
Mn / Omega : Allowable	64.870 k-ft	Vn/Omega : Allowable	48.960 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	6.000ft	Location of maximum on span	7.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.005 in	Ratio =	16,041 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.011 in	Ratio =	7760 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 7.00 ft	1	0.043	0.058	2.82		2.82	108.33	64.87	1.00	1.00	2.84	73.44	48.96
+D+L													
Dsgn. L = 7.00 ft	1	0.047	0.064	3.06		3.06	108.33	64.87	1.00	1.00	3.12	73.44	48.96
+D+Lr													
Dsgn. L = 7.00 ft	1	0.088	0.117	5.72		5.72	108.33	64.87	1.00	1.00	5.75	73.44	48.96
+D+S													
Dsgn. L = 7.00 ft	1	0.604	0.801	39.21		39.21	108.33	64.87	1.00	1.00	39.24	73.44	48.96
+D+0.750Lr+0.750L													
Dsgn. L = 7.00 ft	1	0.080	0.107	5.18		5.18	108.33	64.87	1.00	1.00	5.23	73.44	48.96
+D+0.750L+0.750S													
Dsgn. L = 7.00 ft	1	0.467	0.620	30.29		30.29	108.33	64.87	1.00	1.00	30.35	73.44	48.96
+0.60D													
Dsgn. L = 7.00 ft	1	0.026	0.035	1.69		1.69	108.33	64.87	1.00	1.00	1.71	73.44	48.96

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0108	3.980		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	6.687	39.237
Overall MINimum	0.280	0.280
D Only	0.621	2.843

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Steel Beam

Description : B17 - RIDGE SUPPORT

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D+L	0.901	3.123
+D+Lr	1.106	5.749
+D+S	6.687	39.237
+D+0.750Lr+0.750L	1.195	5.232
+D+0.750L+0.750S	5.381	30.349
+0.60D	0.373	1.706
Lr Only	0.484	2.906
L Only	0.280	0.280
S Only	6.066	36.394

RICHMOND HOFFMAYER INC.
STRUCTURAL ENGINEERING

Project Title: VILLAGE NEST
Engineer: JJH
Project Descr: MULTI UNIT PROJECT

Project ID: 2017-0610

4c

Printed: 21 JUN 2017, 11:03AM

Steel Column

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Description : B17 SUPPORT

Code References

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Steel Section Name : **Pipe4STD**
Analysis Method : Allowable Strength
Steel Stress Grade
Fy : Steel Yield 36.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 10.0 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 10.0 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for Y-Y Axis buckling = 10.0 ft, K = 1.0

Applied Loads

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

Column self weight included : 108.0 lbs * Dead Load Factor
AXIAL LOADS . . .
B17: Axial Load at 10.0 ft, D = 2.843, LR = 2.906, L = 0.280, S = 36.394 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.8598 : 1	
Load Combination	+D+S	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pa : Axial	39.345	k
Pn / Omega : Allowable	45.760	k
Ma-x : Applied	0.0	k-ft
Mn-x / Omega : Allowable	7.275	k-ft
Ma-y : Applied	0.0	k-ft
Mn-y / Omega : Allowable	7.275	k-ft
PASS Maximum Shear Stress Ratio =	0.0 : 1	
Load Combination		
Location of max.above base	0.0	ft
At maximum location values are . . .		
Va : Applied	0.0	k
Vn / Omega : Allowable	0.0	k

Maximum SERVICE Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum SERVICE Load Deflections . . .

Along Y-Y 0.0 in at 0.0ft above base
for load combination :

Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
Load Combination	Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.064	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+L	0.071	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+Lr	0.128	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+S	0.860	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750Lr+0.750L	0.117	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750L+0.750S	0.666	PASS	0.00 ft	0.000	PASS	0.00 ft
+0.60D	0.039	PASS	0.00 ft	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

	Axial Reaction	X-X Axis Reaction	k	Y-Y Axis Reaction		Mx - End Moments	k-ft	My - End Moments	
Load Combination	@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
D Only	2.951								
+D+L	3.231								
+D+Lr	5.857								
+D+S	39.345								
+D+0.750Lr+0.750L	5.341								
+D+0.750L+0.750S	30.457								
+0.60D	1.771								

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Steel Column

Description : B17 SUPPORT

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction	X-X Axis Reaction		k	Y-Y Axis Reaction		Mx - End Moments		k-ft	My - End Moments	
	@ Base	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top		@ Base	@ Top
Lr Only	2.906										
L Only	0.280										
S Only	36.394										

Extreme Reactions

Item	Extreme Value	Axial Reaction	X-X Axis Reaction		k	Y-Y Axis Reaction		Mx - End Moments		k-ft	My - End Moments	
		@ Base	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top		@ Base	@ Top
Axial @ Base	Maximum	39.345										
"	Minimum	0.280										
Reaction, X-X Axis	Maximum	2.951										
"	Minimum	2.951										
Reaction, Y-Y Axis	Maximum	2.951										
"	Minimum	2.951										
Reaction, X-X Axis	Maximum	2.951										
"	Minimum	2.951										
Reaction, Y-Y Axis	Maximum	2.951										
"	Minimum	2.951										
Moment, X-X Axis Ba	Maximum	2.951										
"	Minimum	2.951										
Moment, Y-Y Axis Ba	Maximum	2.951										
"	Minimum	2.951										
Moment, X-X Axis To	Maximum	2.951										
"	Minimum	2.951										
Moment, Y-Y Axis To	Maximum	2.951										
"	Minimum	2.951										

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+L	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr+0.750L	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750L+0.750S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
Lr Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
L Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
S Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Steel Section Properties : Pipe4STD

Depth	=	4.500 in	I xx	=	6.82 in^4	J	=	13.600 in^4
			S xx	=	3.03 in^3			
Diameter	=	4.500 in	R xx	=	1.510 in			
Wall Thick	=	0.237 in	Zx	=	4.050 in^3			
Area	=	2.960 in^2	I yy	=	6.820 in^4			
Weight	=	10.800 plf	S yy	=	3.030 in^3			
			R yy	=	1.510 in			

Ycg = 0.000 in

Steel Column

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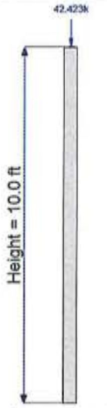
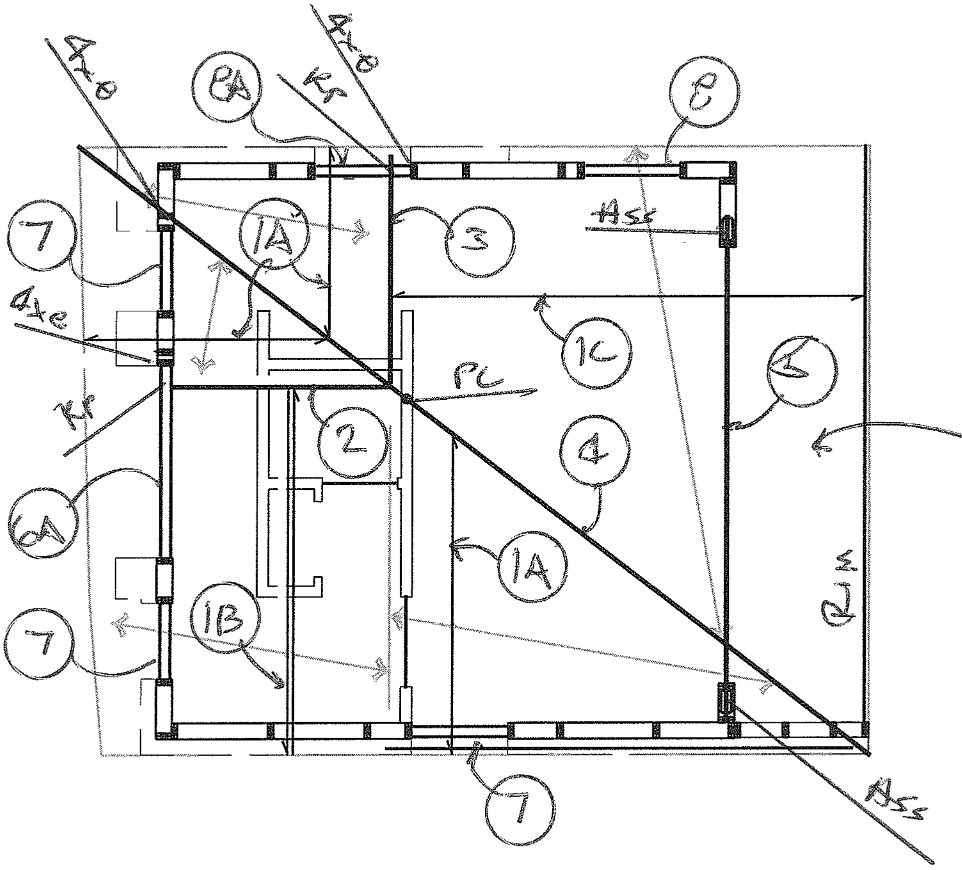
Description : B17 SUPPORT



Loads are total entered value. Arrows do not reflect absolute direction.

PROJECT FELDSPAR LOWER JOB 2017-0610
CLIENT VILLAGE NEST
ADDRESS EDEN, UTAH
DESIGN HUUM
ENGINEER J.H. DATE 6/16/2017
RICHMOND PE 49628 HOFFMAYER SE 3935 SHEET S1 OF


4x8
8A
KP
4x8
8
7
1A
3
A35
4x8
1C
5
KP
PC
2
4
6A
1A
RIM
7
1B
7
A35

OVERHANG
D = 15.8 psf
Lr = 20 psf
S = 444 psf

QUARTZ LVN

ROOF FRAMING PLAN

QUARTZ - LOWER
N.T.S.

○ : BEAM #

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Wood Beam

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Description : B1A - RAFTERS

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

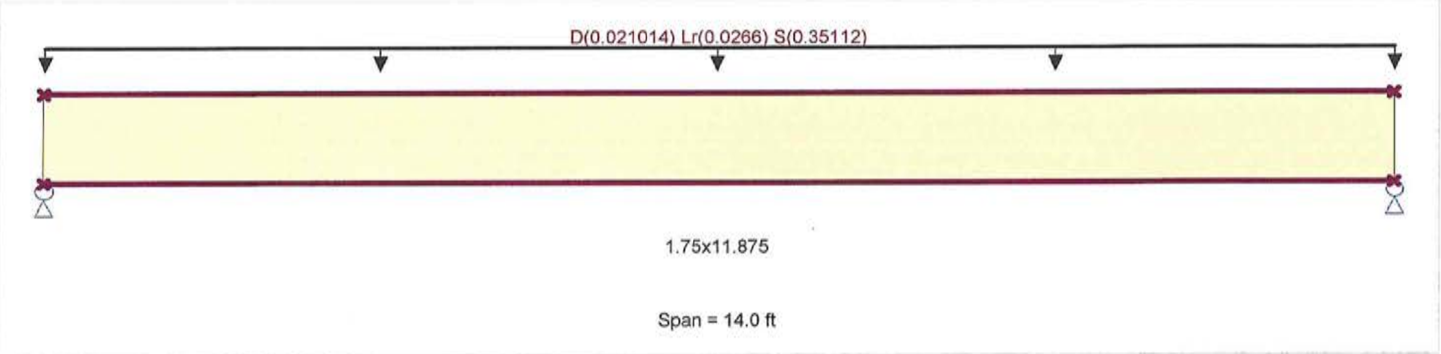
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb - Tension	2,600.0 psi
Fb - Compr	2,600.0 psi
Fc - Prll	2,510.0 psi
Fc - Perp	750.0 psi
Fv	285.0 psi
Ft	1,555.0 psi

E : Modulus of Elasticity

Ebend- xx	1,900.0 ksi
Eminbend - xx	965.71 ksi
Density	42.0 pcf

Repetitive Member Stress Increase



Applied Loads

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

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 1.330 ft, (ROOF LOADS)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.774 1	Maximum Shear Stress Ratio	=	0.494 : 1
Section used for this span		1.75x11.875	Section used for this span		1.75x11.875
fb : Actual	=	2,660.07psi	fv : Actual	=	161.95 psi
FB : Allowable	=	3,438.50psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	7.000ft	Location of maximum on span	=	13.029 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.658 in	Ratio =	255>=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0<240.0
Max Downward Total Deflection	0.697 in	Ratio =	240>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0<240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.605	2.605
Overall MINimum	0.088	0.088
D Only	0.147	0.147
+D+Lr	0.333	0.333
+D+S	2.605	2.605
+D+0.750Lr	0.287	0.287
+D+0.750S	1.990	1.990
+0.60D	0.088	0.088
Lr Only	0.186	0.186
S Only	2.458	2.458

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Wood Beam

Description : B1B - RAFTERS

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

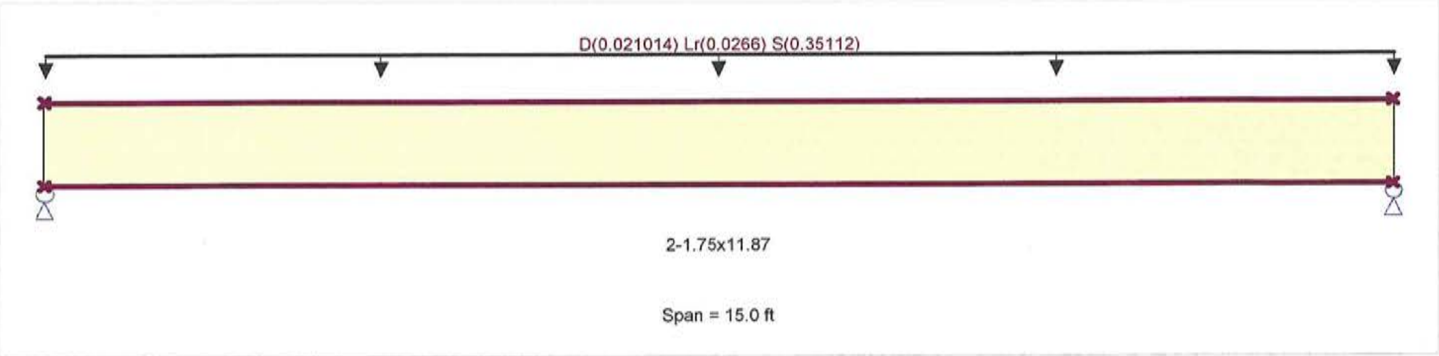
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb - Tension	2,600.0 psi
Fb - Compr	2,600.0 psi
Fc - Prll	2,510.0 psi
Fc - Perp	750.0 psi
Fv	285.0 psi
Ft	1,555.0 psi

E : Modulus of Elasticity

Ebend- xx	1,900.0 ksi
Eminbend - xx	965.71 ksi
Density	42.0 pcf

Repetitive Member Stress Increase



Applied Loads

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

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 1.330 ft, (ROOF LOADS)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.444 1	Maximum Shear Stress Ratio	=	0.267 : 1
Section used for this span		2-1.75x11.87	Section used for this span		2-1.75x11.87
fb : Actual	=	1,526.82psi	fv : Actual	=	87.49 psi
FB : Allowable	=	3,438.50psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	7.500ft	Location of maximum on span	=	14.015 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.434 in	Ratio =	415 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.459 in	Ratio =	391 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.791	2.791
Overall MINimum	0.095	0.095
D Only	0.158	0.158
+D+Lr	0.357	0.357
+D+S	2.791	2.791
+D+0.750Lr	0.307	0.307
+D+0.750S	2.133	2.133
+0.60D	0.095	0.095
Lr Only	0.200	0.200
S Only	2.633	2.633

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Wood Beam

Description : B1C - RAFTERS w OVERHANG

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

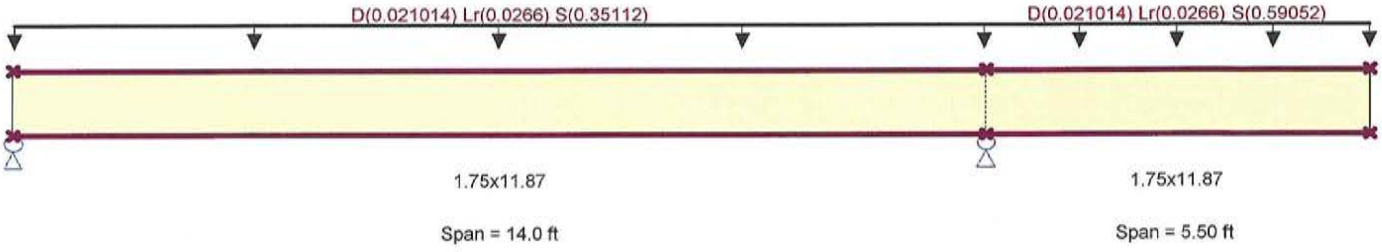
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb - Tension	2,600.0 psi
Fb - Compr	2,600.0 psi
Fc - Prll	2,510.0 psi
Fc - Perp	750.0 psi
Fv	285.0 psi
Ft	1,555.0 psi

E : Modulus of Elasticity

Ebend- xx	1,900.0 ksi
Eminbend - xx	965.71 ksi
Density	42.0 pcf

Repetitive Member Stress Increase


D(0.021014) Lr(0.0266) S(0.35112)
D(0.021014) Lr(0.0266) S(0.59052)
1.75x11.87
Span = 14.0 ft
1.75x11.87
Span = 5.50 ft

Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 1.330 ft, (ROOF LOADS)
Load for Span Number 2
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.4440 ksf, Tributary Width = 1.330 ft, (ROOF LOADS)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.785 1	Maximum Shear Stress Ratio	=	0.642 : 1
Section used for this span		1.75x11.87	Section used for this span		1.75x11.87
fb : Actual	=	2,698.63psi	fv : Actual	=	210.50 psi
FB : Allowable	=	3,438.50psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	14.000ft	Location of maximum on span	=	13.061 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.282 in	Ratio =	466 >=240.
Max Upward Transient Deflection	-0.062 in	Ratio =	2118 >=240.
Max Downward Total Deflection	0.272 in	Ratio =	484 >=240.
Max Upward Total Deflection	-0.072 in	Ratio =	1828 >=240.

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	1.944	6.629	
Overall MINimum	-0.029	0.171	
D Only	0.124	0.285	
+D+Lr, LL Comb Run (*L)	0.096	0.460	
+D+Lr, LL Comb Run (L*)	0.311	0.472	
+D+Lr, LL Comb Run (LL)	0.282	0.647	
+D+S	1.944	6.629	
+D+0.750Lr, LL Comb Run (*L)	0.103	0.417	
+D+0.750Lr, LL Comb Run (L*)	0.264	0.425	
+D+0.750Lr, LL Comb Run (LL)	0.242	0.556	

Wood Beam

Description : B1C - RAFTERS w OVERHANG

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
+D+0.750S	1.489	5.043	
+0.60D	0.075	0.171	
Lr Only, LL Comb Run (*L)	-0.029	0.175	
Lr Only, LL Comb Run (L*)	0.186	0.186	
Lr Only, LL Comb Run (LL)	0.157	0.361	
S Only	1.820	6.344	

Wood Beam

Description : B2 - ROOF BEAM

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

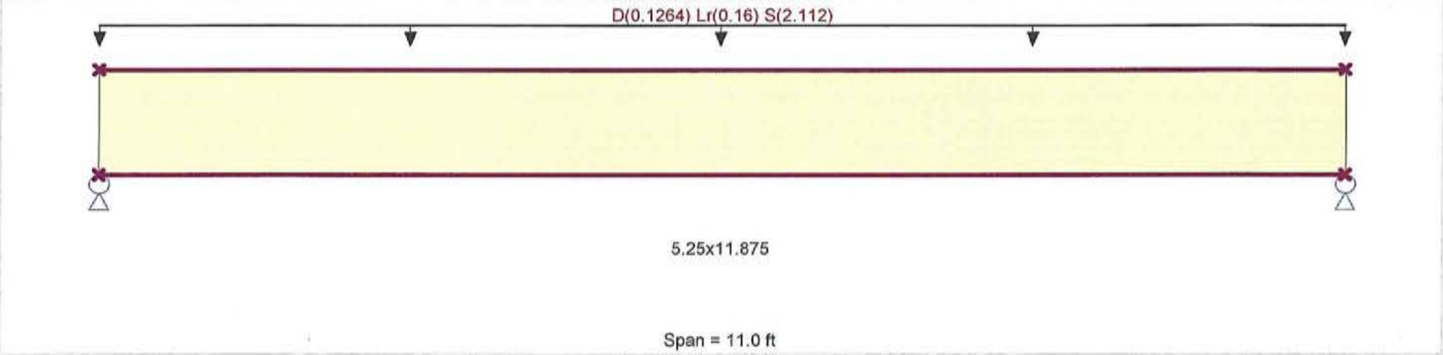
Wood Species : Trus Joist
Wood Grade : Parallam PSL 2.0E

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

Fb - Tension	2,900.0 psi
Fb - Compr	2,900.0 psi
Fc - Prll	2,900.0 psi
Fc - Perp	625.0 psi
Fv	290.0 psi
Ft	2,025.0 psi

E : Modulus of Elasticity

Ebend- xx	2,000.0ksi
Eminbend - xx	1,016.54ksi
Density	45.050pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 8.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.996 1	Maximum Shear Stress Ratio	=	0.739 : 1
Section used for this span		5.25x11.875	Section used for this span		5.25x11.875
fb : Actual	=	3,321.29psi	fv : Actual	=	246.45 psi
FB : Allowable	=	3,335.00psi	Fv : Allowable	=	333.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.500ft	Location of maximum on span	=	10.036 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.478 in	Ratio =	276 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.511 in	Ratio =	258 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	12.418	12.418
Overall MINimum	0.481	0.481
D Only	0.802	0.802
+D+Lr	1.682	1.682
+D+S	12.418	12.418
+D+0.750Lr	1.462	1.462
+D+0.750S	9.514	9.514
+0.60D	0.481	0.481
Lr Only	0.880	0.880
S Only	11.616	11.616

Wood Beam

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Description : B3 - ROOF BEAM

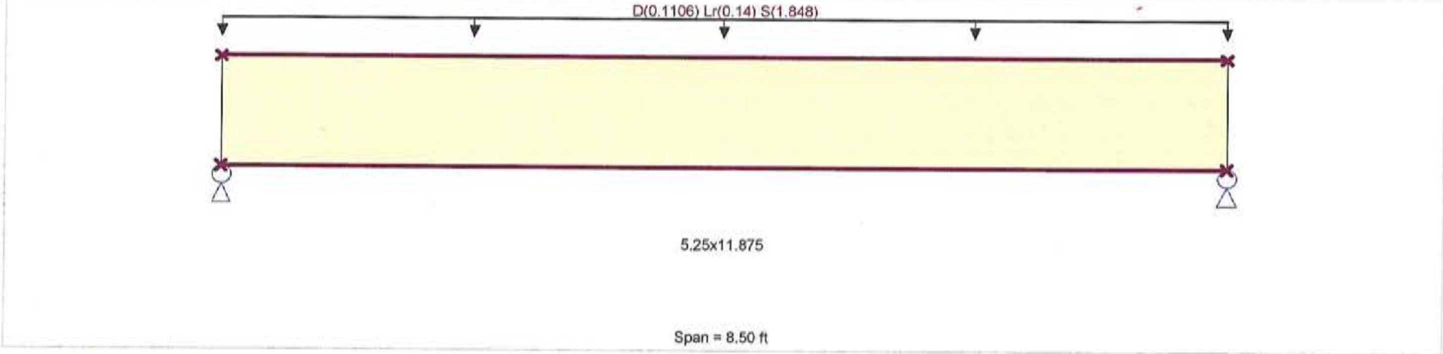
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb - Tension	2,900.0 psi	*E : Modulus of Elasticity*	
Load Combination	ASCE 7-10	Fb - Compr	2,900.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,900.0 psi	Eminbend - xx	1,016.54 ksi
Wood Species	: Trus Joist	Fc - Perp	625.0 psi		
Wood Grade	: Parallam PSL 2.0E	Fv	290.0 psi		
		Ft	2,025.0 psi	Density	45.050 pcf

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



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 7.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.521: 1	Maximum Shear Stress Ratio	=	0.469 : 1
Section used for this span		5.25x11.875	Section used for this span		5.25x11.875
fb : Actual	=	1,737.41psi	fv : Actual	=	156.50 psi
FB : Allowable	=	3,335.00psi	Fv : Allowable	=	333.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.250ft	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	0.149 in	Ratio =	684 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.159 in	Ratio =	639 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	8.407	8.407
Overall MINimum	0.332	0.332
D Only	0.553	0.553
+D+Lr	1.148	1.148
+D+S	8.407	8.407
+D+0.750Lr	0.999	0.999
+D+0.750S	6.443	6.443
+0.60D	0.332	0.332
Lr Only	0.595	0.595
S Only	7.854	7.854

Wood Beam

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Description : B4 - RIDGE

CODE REFERENCES

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

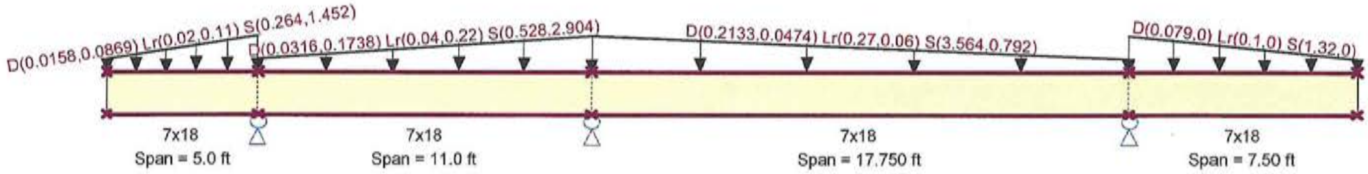
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : Parallam PSL 2.0E

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

Fb - Tension	2,900.0 psi	*E : Modulus of Elasticity*	
Fb - Compr	2,900.0 psi	Ebend- xx	2,000.0 ksi
Fc - Prll	2,900.0 psi	Eminbend - xx	1,016.54 ksi
Fc - Perp	625.0 psi		
Fv	290.0 psi		
Ft	2,025.0 psi	Density	45.050 pcf


D(0.0158,0.0869) Lr(0.02,0.11) S(0.264,1.452)
D(0.0316,0.1738) Lr(0.04,0.22) S(0.528,2.904)
D(0.2133,0.0474) Lr(0.27,0.06) S(3.564,0.792)
D(0.079,0) Lr(0.1,0) S(1.32,0)
7x18
Span = 5.0 ft
7x18
Span = 11.0 ft
7x18
Span = 17.750 ft
7x18
Span = 7.50 ft

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

Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 0.0 -->> 5.0 ft, Trib Width = 1.0->5.50 ft, (

Load for Span Number 2

Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 0.0 -->> 11.0 ft, Trib Width = 2.0->11.0 ft,

Load for Span Number 3

Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 0.0 -->> 17.750 ft, Trib Width = 13.50->3.

Load for Span Number 4

Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 0.0 -->> 7.50 ft, Trib Width = 5.0->0.0 ft, (

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.643 1	Maximum Shear Stress Ratio	=	0.822 : 1
Section used for this span		7x18	Section used for this span		7x18
fb : Actual	=	2,049.11psi	fv : Actual	=	274.07 psi
FB : Allowable	=	3,188.23psi	Fv : Allowable	=	333.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	11.000ft	Location of maximum on span	=	11.000 ft
Span # where maximum occurs	=	Span # 2	Span # where maximum occurs	=	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.357 in	Ratio =	597>=360
Max Upward Transient Deflection	-0.412 in	Ratio =	436>=360
Max Downward Total Deflection	0.382 in	Ratio =	558>=240.
Max Upward Total Deflection	-0.433 in	Ratio =	414>=240.

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
Overall MAXimum		7.627	45.573	19.198	
Overall MINimum		0.012	-0.074	0.007	
D Only		0.799	3.111	1.712	
+D+Lr, LL Comb Run (***L)		0.825	3.016	2.156	

Wood Beam

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Description : B4 - RIDGE

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
+D+Lr, LL Comb Run (**L*)		0.419	5.502	2.630	
+D+Lr, LL Comb Run (**LL)		0.445	5.407	3.074	
+D+Lr, LL Comb Run (*L**)		1.278	4.107	1.668	
+D+Lr, LL Comb Run (*L*L)		1.304	4.012	2.112	
+D+Lr, LL Comb Run (*LL*)		0.897	6.498	2.586	
+D+Lr, LL Comb Run (*LLL)		0.924	6.402	3.030	
+D+Lr, LL Comb Run (L***)		1.192	3.037	1.719	
+D+Lr, LL Comb Run (L**L)		1.218	2.941	2.163	
+D+Lr, LL Comb Run (L*L*)		0.812	5.428	2.637	
+D+Lr, LL Comb Run (L*LL)		0.838	5.332	3.081	
+D+Lr, LL Comb Run (LL**)		1.670	4.033	1.675	
+D+Lr, LL Comb Run (LL*L)		1.697	3.937	2.119	
+D+Lr, LL Comb Run (LLL*)		1.290	6.423	2.593	
+D+Lr, LL Comb Run (LLLL)		1.316	6.328	3.037	
+D+S		7.627	45.573	19.198	
+D+0.750Lr, LL Comb Run (***L)		0.819	3.040	2.045	
+D+0.750Lr, LL Comb Run (**L*)		0.514	4.904	2.401	
+D+0.750Lr, LL Comb Run (**LL)		0.534	4.833	2.734	
+D+0.750Lr, LL Comb Run (*L**)		1.158	3.858	1.679	
+D+0.750Lr, LL Comb Run (*L*L)		1.178	3.786	2.012	
+D+0.750Lr, LL Comb Run (*LL*)		0.873	5.651	2.368	
+D+0.750Lr, LL Comb Run (*LLL)		0.893	5.580	2.701	
+D+0.750Lr, LL Comb Run (L***)		1.094	3.055	1.717	
+D+0.750Lr, LL Comb Run (L**L)		1.113	2.984	2.050	
+D+0.750Lr, LL Comb Run (L*L*)		0.809	4.848	2.406	
+D+0.750Lr, LL Comb Run (L*LL)		0.828	4.777	2.739	
+D+0.750Lr, LL Comb Run (LL**)		1.452	3.802	1.684	
+D+0.750Lr, LL Comb Run (LL*L)		1.472	3.731	2.017	
+D+0.750Lr, LL Comb Run (LLL*)		1.167	5.595	2.373	
+D+0.750Lr, LL Comb Run (LLLL)		1.187	5.524	2.706	
+D+0.750S		5.920	34.958	14.826	
+0.60D		0.479	1.867	1.027	
Lr Only, LL Comb Run (***L)		0.026	-0.095	0.444	
Lr Only, LL Comb Run (**L*)		-0.380	2.391	0.918	
Lr Only, LL Comb Run (**LL)		-0.354	2.295	1.362	
Lr Only, LL Comb Run (*L**)		0.478	0.996	-0.044	
Lr Only, LL Comb Run (*L*L)		0.505	0.900	0.400	
Lr Only, LL Comb Run (*LL*)		0.098	3.387	0.874	
Lr Only, LL Comb Run (*LLL)		0.125	3.291	1.318	
Lr Only, LL Comb Run (L***)		0.393	-0.074	0.007	
Lr Only, LL Comb Run (L**L)		0.419	-0.170	0.451	
Lr Only, LL Comb Run (L*L*)		0.012	2.316	0.925	
Lr Only, LL Comb Run (L*LL)		0.039	2.221	1.369	
Lr Only, LL Comb Run (LL**)		0.871	0.921	-0.038	
Lr Only, LL Comb Run (LL*L)		0.897	0.826	0.407	
Lr Only, LL Comb Run (LLL*)		0.491	3.312	0.881	
Lr Only, LL Comb Run (LLLL)		0.517	3.217	1.325	
S Only		6.828	42.462	17.485	

Wood Beam

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Description : B5 - BIG OPENING

CODE REFERENCES

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

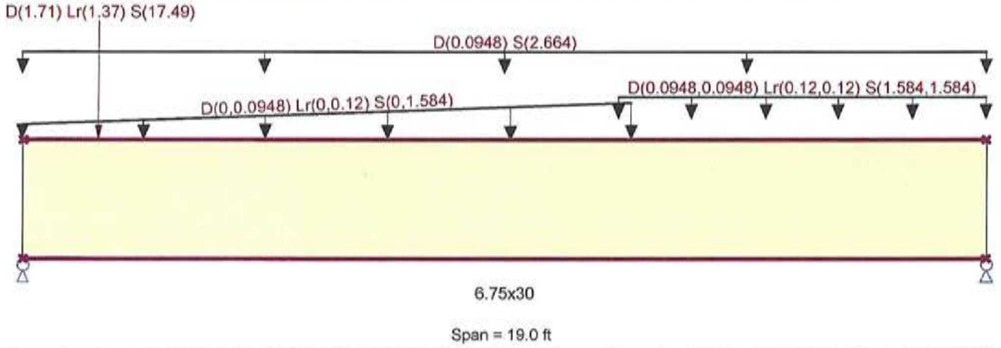
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F - V4

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

		E : Modulus of Elasticity	
Fb - Tension	2,400.0 psi	Ebend- xx	1,800.0 ksi
Fb - Compr	1,850.0 psi	Eminbend - xx	950.0 ksi
Fc - Prll	1,650.0 psi		
Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Fv	265.0 psi	Eminbend - yy	850.0 ksi
Ft	1,100.0 psi	Density	31.20 pcf


D(1.71) Lr(1.37) S(17.49)
D(0.0948) S(2.664)
D(0,0.0948) Lr(0,0.12) S(0,1.584)
D(0.0948,0.0948) Lr(0.12,0.12) S(1.584,1.584)
6.75x30
Span = 19.0 ft

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
Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 0.0 -->> 12.0 ft, Trib Width = 0.0->6.0 ft, (
Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 11.750 -->> 19.0 ft, Trib Width = 6.0 ft, (R
Uniform Load : D = 0.01580, S = 0.4440 ksf, Tributary Width = 6.0 ft, (EAVE)
Point Load : D = 1.710, Lr = 1.370, S = 17.490 k @ 1.50 ft, (B4)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.950: 1	Maximum Shear Stress Ratio	=	0.755 : 1
Section used for this span		6.75x30	Section used for this span		6.75x30
fb : Actual	=	2,350.85psi	fv : Actual	=	229.96 psi
FB : Allowable	=	2,474.58psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	9.569ft	Location of maximum on span	=	16.504 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.453 in	Ratio =	503 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.479 in	Ratio =	475 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	52.463	42.232
Overall MINimum	1.845	1.115
D Only	3.353	2.248
+D+Lr	5.198	3.363
+D+S	52.463	42.232
+D+0.750Lr	4.736	3.085
+D+0.750S	40.186	32.236
+0.60D	2.012	1.349

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Wood Beam

Description : B5 - BIG OPENING

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Lr Only	1.845	1.115
S Only	49.111	39.983

Steel Beam

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : B5 - BIG OPENING - STEEL OPTION

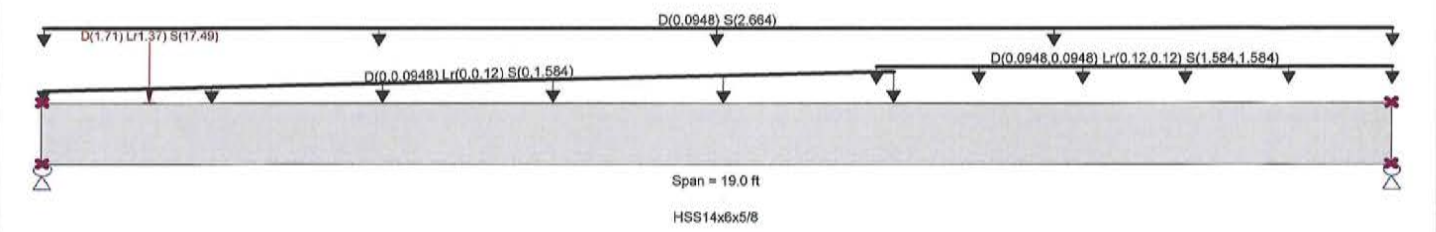
CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending

Fy : Steel Yield : 46.0 ksi
E: Modulus : 29,000.0 ksi


D(0.0948) S(2.664)
D(1.71) Lr(1.37) S(17.49)
D(0.0.0948) Lr(0.0.12) S(0.1.584)
D(0.0948,0.0948) Lr(0.12,0.12) S(1.584,1.584)
Span = 19.0 ft
HSS14x6x5/8

Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 0.0 -->> 12.0 ft, Trib Width = 0.
Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 11.750 -->> 19.0 ft, Trib Width =
Uniform Load : D = 0.01580, S = 0.4440 ksf, Tributary Width = 6.0 ft, (EAVE)
Point Load : D = 1.710, Lr = 1.370, S = 17.490 k @ 1.50 ft, (B4)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.981 : 1	Maximum Shear Stress Ratio =	0.224 : 1
Section used for this span	HSS14x6x5/8	Section used for this span	HSS14x6x5/8
Ma : Applied	199.816 k-ft	Va : Applied	52.772 k
Mn / Omega : Allowable	203.603 k-ft	Vn/Omega : Allowable	235.387 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	9.554ft	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	0.891 in	Ratio =	255 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.951 in	Ratio =	240 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 19.00 ft	1	0.060	0.016	12.25		12.25	340.02	203.60	1.12	1.00	3.66	393.10	235.39
+D+Lr													
Dsgn. L = 19.00 ft	1	0.085	0.023	17.39		17.39	340.02	203.60	1.12	1.00	5.51	393.10	235.39
+D+S													
Dsgn. L = 19.00 ft	1	0.981	0.224	199.82		199.82	340.02	203.60	1.13	1.00	52.77	393.10	235.39
+D+0.750Lr													
Dsgn. L = 19.00 ft	1	0.079	0.021	16.10		16.10	340.02	203.60	1.12	1.00	5.04	393.10	235.39
+D+0.750S													
Dsgn. L = 19.00 ft	1	0.751	0.172	152.92		152.92	340.02	203.60	1.13	1.00	40.49	393.10	235.39
+0.60D													
Dsgn. L = 19.00 ft	1	0.036	0.009	7.35		7.35	340.02	203.60	1.12	1.00	2.20	393.10	235.39

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.9512	9.500		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	52.772	42.540
Overall MINimum	1.845	1.115

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Steel Beam

Description : B5 - BIG OPENING - STEEL OPTION

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
D Only	3.661	2.556
+D+Lr	5.506	3.672
+D+S	52.772	42.540
+D+0.750Lr	5.045	3.393
+D+0.750S	40.494	32.544
+0.60D	2.197	1.534
Lr Only	1.845	1.115
S Only	49.111	39.983

64

Wood Beam

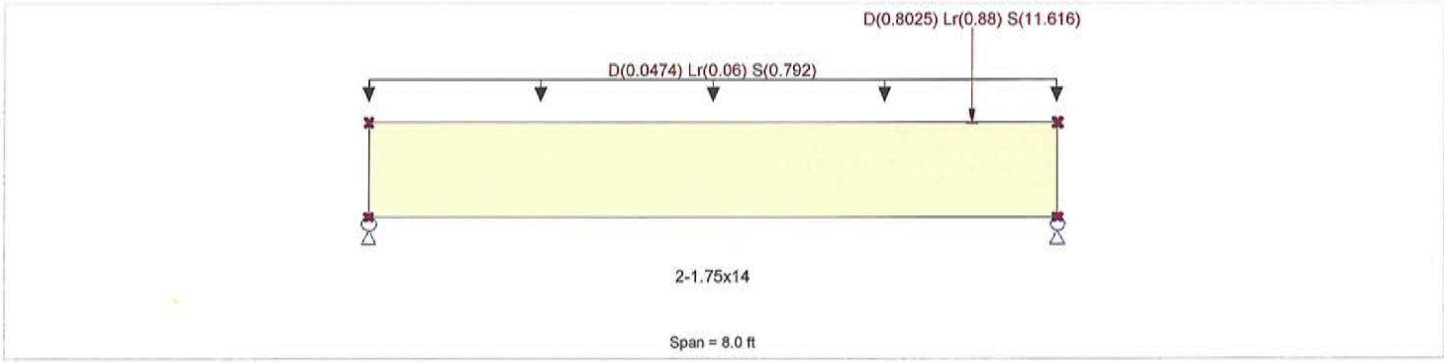
Description : B6A - HEADER

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb - Tension	2,600.0 psi	*E : Modulus of Elasticity*	
Load Combination	ASCE 7-10	Fb - Compr	2,600.0 psi	Ebend- xx	1,900.0 ksi
		Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Wood Species	: Trus Joist	Fc - Perp	750.0 psi		
Wood Grade	: MicroLam LVL 1.9 E	Fv	285.0 psi		
		Ft	1,555.0 psi	Density	42.0 pcf
Beam Bracing	: Completely Unbraced				



Applied Loads

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

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 3.0 ft, (ROOF)
Point Load : D = 0.8025, Lr = 0.880, S = 11.616 k @ 7.0 ft, (B2)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.545 1	**Maximum Shear Stress Ratio**	=	0.369 : 1
Section used for this span		2-1.75x14	Section used for this span		2-1.75x14
fb : Actual	=	1,507.15psi	fv : Actual	=	121.04 psi
FB : Allowable	=	2,765.98psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.839ft	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	0.101 in	Ratio =	951 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.107 in	Ratio =	894 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.910	14.224
Overall MINimum	0.174	0.535
D Only	0.290	0.892
+D+Lr	0.640	1.902
+D+S	4.910	14.224
+D+0.750Lr	0.552	1.649
+D+0.750S	3.755	10.891
+0.60D	0.174	0.535
Lr Only	0.350	1.010
S Only	4.620	13.332

Wood Beam

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Description : B7 - BEARING HEADERS

CODE REFERENCES

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

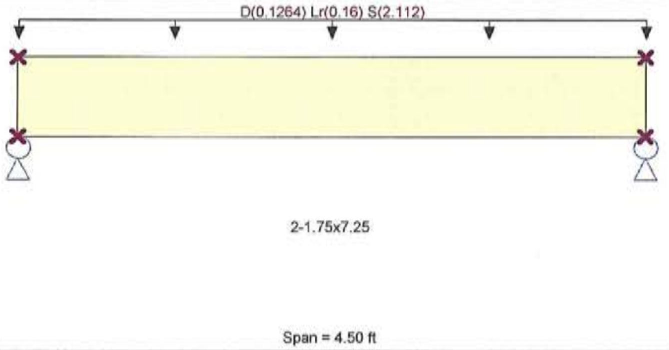
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

Beam Bracing : Completely Unbraced

Fb - Tension	2,600.0 psi	*E : Modulus of Elasticity*	
Fb - Compr	2,600.0 psi	Ebend- xx	1,900.0 ksi
Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Fc - Perp	750.0 psi		
Fv	285.0 psi		
Ft	1,555.0 psi	Density	42.0 pcf



Applied Loads

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

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 8.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.749: 1	Maximum Shear Stress Ratio	=	0.670 : 1
Section used for this span		2-1.75x7.25	Section used for this span		2-1.75x7.25
fb : Actual	=	2,217.49psi	fv : Actual	=	219.49 psi
FB : Allowable	=	2,960.54psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	2.250ft	Location of maximum on span	=	3.909 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.093 in	Ratio =	581 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.098 in	Ratio =	548 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	5.036	5.036
Overall MINimum	0.171	0.171
D Only	0.284	0.284
+D+Lr	0.644	0.644
+D+S	5.036	5.036
+D+0.750Lr	0.554	0.554
+D+0.750S	3.848	3.848
+0.60D	0.171	0.171
Lr Only	0.360	0.360
S Only	4.752	4.752

Wood Beam

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Description : B8 - BEARING HEADERS

CODE REFERENCES

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

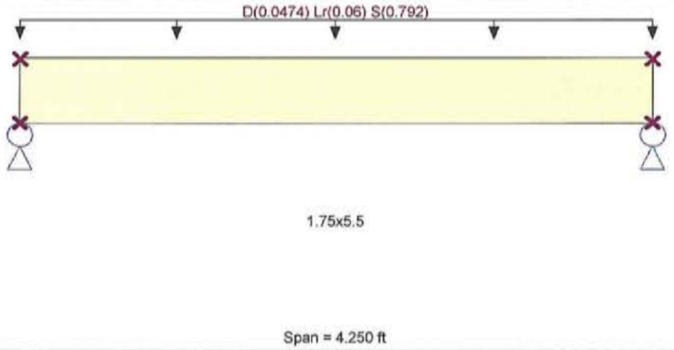
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

Beam Bracing : Completely Unbraced

Fb - Tension	2,600.0 psi	*E : Modulus of Elasticity*	
Fb - Compr	2,600.0 psi	Ebend- xx	1,900.0 ksi
Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Fc - Perp	750.0 psi		
Fv	285.0 psi		
Ft	1,555.0 psi	Density	42.0 pcf



Applied Loads

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

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 3.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.897 : 1	Maximum Shear Stress Ratio	=	0.669 : 1
Section used for this span		1.75x5.5	Section used for this span		1.75x5.5
fb : Actual	=	2,577.66 psi	fv : Actual	=	219.14 psi
FB : Allowable	=	2,875.23 psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	2.125 ft	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	0.127 in	Ratio =	402 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.134 in	Ratio =	379 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.784	1.784
Overall MINimum	0.060	0.060
D Only	0.101	0.101
+D+Lr	0.228	0.228
+D+S	1.784	1.784
+D+0.750Lr	0.196	0.196
+D+0.750S	1.363	1.363
+0.60D	0.060	0.060
Lr Only	0.128	0.128
S Only	1.683	1.683

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Wood Beam

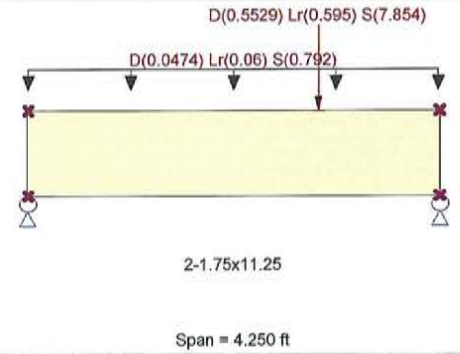
Description : B8A - BEARING HEADERS

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb - Tension	2,600.0 psi	*E : Modulus of Elasticity*	
Load Combination	ASCE 7-10	Fb - Compr	2,600.0 psi	Ebend- xx	1,900.0 ksi
		Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Wood Species	: Trus Joist	Fc - Perp	750.0 psi		
Wood Grade	: MicroLam LVL 1.9 E	Fv	285.0 psi		
		Ft	1,555.0 psi	Density	42.0 pcf
Beam Bracing	: Completely Unbraced				



Applied Loads

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

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 3.0 ft, (ROOF)
Point Load : D = 0.5529, Lr = 0.5950, S = 7.854 k @ 3.0 ft, (B3)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.496 : 1	Maximum Shear Stress Ratio	=	0.806 : 1
Section used for this span		2-1.75x11.25	Section used for this span		2-1.75x11.25
fb : Actual	=	1,459.71 psi	fv : Actual	=	264.26 psi
FB : Allowable	=	2,942.27 psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	2.994 ft	Location of maximum on span	=	3.319 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.029 in	Ratio =	1746 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.031 in	Ratio =	1635 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1   Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.256	7.718
Overall MINimum	0.158	0.295
D Only	0.263	0.491
+D+Lr	0.566	1.039
+D+S	4.256	7.718
+D+0.750Lr	0.490	0.902
+D+0.750S	3.258	5.911
+0.60D	0.158	0.295
Lr Only	0.303	0.548
S Only	3.993	7.227

PROJECT	VILLAGE NEST	JOB	2017-0610
CLIENT	-		
ADDRESS	EDEN, UT		
DESIGN	HUUM		
ENGINEER	J.H.	DATE	6/14/2017
	RICHMOND PE 49628 HOFFMAYER SE 3435	SHEET	68 OF

WIND ON EXTERIOR WALLS:

115 MPH - EXP "C" ASCE 7-10

$W_{MIN} = 16$ PSF

$q_z = 0.00256 K_z K_{zt} K_d V^2$

$K_z = 0.9$ @ 20' TABLE 30.3-1

$K_{zt} = 1$

$K_d = 0.85$

$q_z = 0.00256(0.9 \times 1 \times 0.85)115^2 = \underline{\underline{25.9 \text{ PSF}}}$

$P = 25.9(GC_p - GC_{pi})$

$A_{STUD} = 10' \times 1.33' = 13.3 \text{ ft}^2 \longrightarrow GC_p = 1.8$

$GC_{pi} = 0.18$

$P = 25.9(1.8 + 0.18) = \underline{\underline{51.28 \text{ PSF}}} > 16 \text{ PSF}$

ALL WALL CLADDING MUST RESIST 51.3 PSF WIND

Wood Column

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Description : ROOF SUPPORT POSTS - EXTERIOR WALLS

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method :	Allowable Stress Design		
End Fixities	Top & Bottom Pinned		
Overall Column Height (Used for non-slender calculations)			10.50 ft
Wood Species	Spruce - Pine - Fir		
Wood Grade	No.1		
Fb +	850.0 psi	Fv	125.0 psi
Fb -	850.0 psi	Ft	550.0 psi
Fc - Prll	700.0 psi	Density	26.210 pcf
Fc - Perp	425.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,300.0	1,300.0	1,300.0 ksi
Minimum	470.0	470.0	

Wood Section Name	4x8
Wood Grading/Manuf.	Graded Lumber
Wood Member Type	Sawn
Exact Width	3.50 in
Exact Depth	7.250 in
Area	25.375 in^2
Ix	111.148 in^4
Iy	25.904 in^4

Allow Stress Modification Factors	
Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.050
Cf or Cv for Tension	1.20
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 10 ft, K = 1.0

Applied Loads

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

Column self weight included : 48.495 lbs * Dead Load Factor
AXIAL LOADS . . .
ROOF SUPPORT: Axial Load at 10.50 ft, D = 1.50, Lr = 1.50, S = 15.0 k
BENDING LOADS . . .
WIND: Lat. Uniform Load creating Mx-x, W = 0.2050 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.9256 : 1
	Load Combination	+D+S
	Governing NDS Forumla	Comp Only, fc/Fc'
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Applied Axial	16.549 k
	Applied Mx	0.0 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	704.56 psi
PASS	Maximum Shear Stress Ratio =	0.1272 : 1
	Load Combination	+D+0.60W
	Location of max.above base	0.0 ft
	Applied Design Shear	38.172 psi
	Allowable Shear	200.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	1.076 k	Bottom along Y-Y	1.076 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.3922 in at 5.285 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios Stress Ratio	Status	Location	Maximum Shear Ratios Stress Ratio	Status	Location
D Only	0.900	0.877	0.1052	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+Lr	1.250	0.815	0.1604	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+S	1.150	0.834	0.9256	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.750Lr	1.250	0.815	0.1407	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.750S	1.150	0.834	0.7159	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.60W	1.600	0.750	0.3970	PASS	5.285 ft	0.1272	PASS	0.0 ft
+D+0.750Lr+0.450W	1.600	0.750	0.3184	PASS	5.285 ft	0.09543	PASS	0.0 ft
+D+0.750S+0.450W	1.600	0.750	0.7653	PASS	5.215 ft	0.09543	PASS	0.0 ft
+0.60D+0.60W	1.600	0.750	0.3869	PASS	5.285 ft	0.1272	PASS	0.0 ft

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Wood Column

Description : ROOF SUPPORT POSTS - EXTERIOR WALLS

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios Stress Ratio	Status	Location	Maximum Shear Ratios Stress Ratio	Status	Location
+0.60D	1.600	0.750	0.04153	PASS	0.0 ft	0.0	PASS	10.50 ft

Maximum Reactions

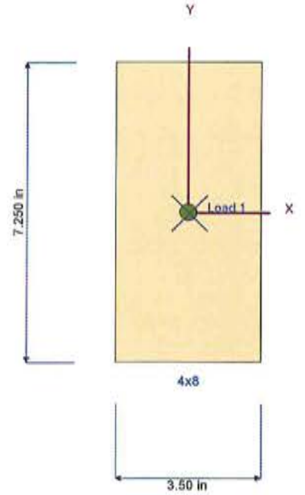
Note: Only non-zero reactions are listed.

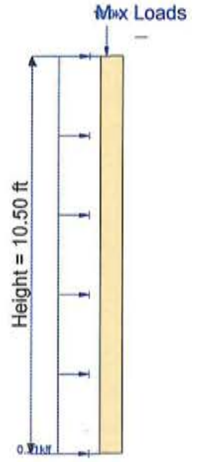
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft Mx - End Moments @ Base	Mx - End Moments @ Top
D Only					1.548				
+D+Lr					3.048				
+D+S					16.548				
+D+0.750Lr					2.673				
+D+0.750S					12.798				
+D+0.60W			0.646	0.646	1.548				
+D+0.750Lr+0.450W			0.484	0.484	2.673				
+D+0.750S+0.450W			0.484	0.484	12.798				
+0.60D+0.60W			0.646	0.646	0.929				
+0.60D					0.929				
Lr Only					1.500				
S Only					15.000				
W Only			1.076	1.076					

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	0.235 in	5.285 ft
+D+0.750Lr+0.450W	0.0000 in	0.000 ft	0.176 in	5.285 ft
+D+0.750S+0.450W	0.0000 in	0.000 ft	0.176 in	5.285 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.235 in	5.285 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.392 in	5.285 ft

Sketches





Loads are total entered value. Arrows do not reflect absolute direction.

Wood Column

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Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : 2ND FLOOR BEARING STUDS @ 48" o.c. - SIP SPLINES

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method :	Allowable Stress Design
End Fixities	Top & Bottom Pinned
Overall Column Height	10.50 ft

(Used for non-slender calculations)

Wood Species	Spruce - Pine - Fir		
Wood Grade	No.1		
Fb +	850.0 psi	Fv	125.0 psi
Fb -	850.0 psi	Ft	550.0 psi
Fc - Prll	700.0 psi	Density	26.210 pcf
Fc - Perp	425.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,300.0	1,300.0	1,300.0 ksi
Minimum	470.0	470.0	

Wood Section Name	2-2x8
Wood Grading/Manuf.	Graded Lumber
Wood Member Type	Sawn
Exact Width	3.0 in
Exact Depth	7.250 in
Area	21.750 in^2
Ix	95.270 in^4
Iy	16.313 in^4

Allow Stress Modification Factors	
Cf or Cv for Bending	1.20
Cf or Cv for Compression	1.050
Cf or Cv for Tension	1.20
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 10.50 ft, K = 1.0

Applied Loads

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

Column self weight included : 41.567 lbs * Dead Load Factor
AXIAL LOADS . . .
BEARING: Axial Load at 10.50 ft, D = 0.5040, Lr = 0.6390, S = 8.430 k
BENDING LOADS . . .
51.3 PSF WIND: Lat. Uniform Load creating Mx-x, W = 0.1020 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.6013 : 1
	Load Combination	+D+S
	Governing NDS Forumla	Comp Only, fc/Fc'
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Applied Axial	8.976 k
	Applied Mx	0.0 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	686.32 psi
PASS	Maximum Shear Stress Ratio =	0.07386 : 1
	Load Combination	+D+0.60W
	Location of max.above base	0.0 ft
	Applied Design Shear	22.159 psi
	Allowable Shear	200.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.5355 k	Bottom along Y-Y	0.5355 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.2277 in at 5.285 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

			Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
Load Combination	C_D	C_P	Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.862	0.04401	PASS	0.0ft	0.0	PASS	10.50 ft
+D+Lr	1.250	0.791	0.07490	PASS	0.0ft	0.0	PASS	10.50 ft
+D+S	1.150	0.812	0.6013	PASS	0.0ft	0.0	PASS	10.50 ft
+D+0.750Lr	1.250	0.791	0.06480	PASS	0.0ft	0.0	PASS	10.50 ft
+D+0.750S	1.150	0.812	0.4601	PASS	0.0ft	0.0	PASS	10.50 ft
+D+0.60W	1.600	0.719	0.2416	PASS	5.285ft	0.07386	PASS	0.0 ft
+D+0.750Lr+0.450W	1.600	0.719	0.1868	PASS	5.215ft	0.05540	PASS	0.0 ft
+D+0.750S+0.450W	1.600	0.719	0.3744	PASS	5.215ft	0.05540	PASS	0.0 ft
+0.60D+0.60W	1.600	0.719	0.2391	PASS	5.285ft	0.07386	PASS	0.0 ft

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Wood Column

Description : 2ND FLOOR BEARING STUDS @ 48" o.c. - SIP SPLINES

Load Combination Results

			Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
Load Combination	C_D	C_P	Stress Ratio	Status	Location	Stress Ratio	Status	Location
+0.60D	1.600	0.719	0.01779	PASS	0.0 ft	0.0	PASS	10.50 ft

Maximum Reactions

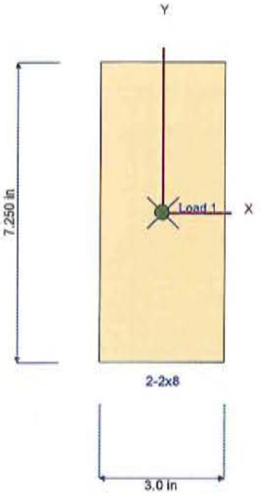
Note: Only non-zero reactions are listed.

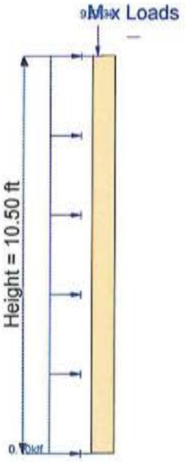
	X-X Axis Reaction k		Y-Y Axis Reaction		Axial Reaction	My - End Moments k-ft		Mx - End Moments	
Load Combination	@ Base	@ Top	@ Base	@ Top	@ Base	@ Base	@ Top	@ Base	@ Top
D Only					0.546				
+D+Lr					1.185				
+D+S					8.976				
+D+0.750Lr					1.025				
+D+0.750S					6.868				
+D+0.60W			0.321	0.321	0.546				
+D+0.750Lr+0.450W			0.241	0.241	1.025				
+D+0.750S+0.450W			0.241	0.241	6.868				
+0.60D+0.60W			0.321	0.321	0.327				
+0.60D					0.327				
Lr Only					0.639				
S Only					8.430				
W Only			0.536	0.535					

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	0.137 in	5.285 ft
+D+0.750Lr+0.450W	0.0000 in	0.000 ft	0.102 in	5.285 ft
+D+0.750S+0.450W	0.0000 in	0.000 ft	0.102 in	5.285 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.137 in	5.285 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.228 in	5.285 ft

Sketches





Loads are total entered value. Arrows do not reflect absolute direction.

Steel Column

Description : RIDGE SUPPORT

Code References

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Steel Section Name :	Pipe4 Std
Analysis Method :	Allowable Strength
Steel Stress Grade	A-36, Carbon Steel, Fy = 36 ksi
Fy : Steel Yield	36.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height	10.0 ft
Top & Bottom Fixity	Top & Bottom Pinned

Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Fully braced against buckling along X-X Axis
Y-Y (depth) axis :
Fully braced against buckling along Y-Y Axis

Applied Loads

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

Column self weight included : 108.581 lbs * Dead Load Factor
AXIAL LOADS . . .
MAX ROOF: Axial Load at 10.0 ft, D = 3.110, LR = 3.390, S = 42.460 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.7135 : 1
	Load Combination	+D+S
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Pa : Axial	45.679 k
	Pn / Omega : Allowable	64.024 k
	Ma-x : Applied	0.0 k-ft
	Mn-x / Omega : Allowable	7.275 k-ft
	Ma-y : Applied	0.0 k-ft
	Mn-y / Omega : Allowable	7.275 k-ft
PASS	Maximum Shear Stress Ratio =	0.0 : 1
	Load Combination	
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Va : Applied	0.0 k
	Vn / Omega : Allowable	0.0 k

Maximum SERVICE Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum SERVICE Load Deflections . . .

Along Y-Y 0.0 in at 0.0ft above base
for load combination :

Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios Stress Ratio	Status	Location	Maximum Shear Ratios Stress Ratio	Status	Location
D Only	0.050	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+Lr	0.103	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+S	0.713	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750Lr	0.090	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750S	0.548	PASS	0.00 ft	0.000	PASS	0.00 ft
+0.60D	0.030	PASS	0.00 ft	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
D Only	3.219								
+D+Lr	6.609								
+D+S	45.679								
+D+0.750Lr	5.761								
+D+0.750S	35.064								
+0.60D	1.931								
Lr Only	3.390								
S Only	42.460								

Steel Column

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Description : RIDGE SUPPORT

Extreme Reactions

		Axial Reaction	X-X Axis Reaction		k	Y-Y Axis Reaction		Mx - End Moments		k-ft	My - End Moments	
Item	Extreme Value	@ Base	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top		@ Base	@ Top
Axial @ Base	Maximum	45.679										
"	Minimum	1.931										
Reaction, X-X Axis	Maximum	3.219										
"	Minimum	3.219										
Reaction, Y-Y Axis	Maximum	3.219										
"	Minimum	3.219										
Reaction, X-X Axis	Maximum	3.219										
"	Minimum	3.219										
Reaction, Y-Y Axis	Maximum	3.219										
"	Minimum	3.219										
Moment, X-X Axis Ba	Maximum	3.219										
"	Minimum	3.219										
Moment, Y-Y Axis Ba	Maximum	3.219										
"	Minimum	3.219										
Moment, X-X Axis To	Maximum	3.219										
"	Minimum	3.219										
Moment, Y-Y Axis To	Maximum	3.219										
"	Minimum	3.219										

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
Lr Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
S Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Steel Section Properties : Pipe4 Std

Depth	=	4.500 in	I xx	=	6.82 in^4	J	=	13.600 in^4
			S xx	=	3.03 in^3			
Diameter	=	4.500 in	R xx	=	1.510 in			
Wall Thick	=	0.237 in	Zx	=	4.050 in^3			
Area	=	2.970 in^2	I yy	=	6.820 in^4			
Weight	=	10.858 plf	S yy	=	3.030 in^3			
			R yy	=	1.510 in			

Ycg = 0.000 in

Steel Column

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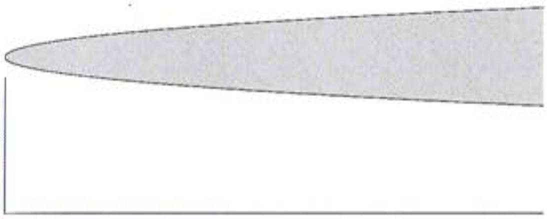
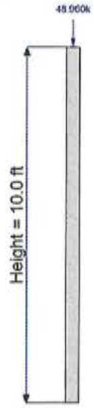
Description : RIDGE SUPPORT





Loads are total entered value. Arrows do not reflect absolute direction.

Steel Column

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Description : B5 SUPPORT

Code References

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Steel Section Name :	**HSS12x6x5/8**
Analysis Method :	Allowable Strength
Steel Stress Grade	
Fy : Steel Yield	46.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height 10.0 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 10.0 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for Y-Y Axis buckling = 10.0 ft, K = 2.1

Applied Loads

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

Column self weight included : 676.18 lbs * Dead Load Factor
AXIAL LOADS . . .
B5: Axial Load at 10.0 ft, D = 3.350, LR = 1.840, S = 49.110 k

< 0.15 ✓ USE FOR LATERAL OPTION

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.1324 : 1	
Load Combination	+D+S	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pa : Axial	53.136	k
Pn / Omega : Allowable	401.460	k
Ma-x : Applied	0.0	k-ft
Mn-x / Omega : Allowable	157.924	k-ft
Ma-y : Applied	0.0	k-ft
Mn-y / Omega : Allowable	96.637	k-ft
PASS Maximum Shear Stress Ratio =	0.0 : 1	
Load Combination		
Location of max.above base	0.0	ft
At maximum location values are . . .		
Va : Applied	0.0	k
Vn / Omega : Allowable	0.0	k

Maximum SERVICE Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum SERVICE Load Deflections . . .

Along Y-Y 0.0 in at 0.0ft above base
for load combination :
Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios Stress Ratio	Status	Location	Maximum Shear Ratios Stress Ratio	Status	Location
D Only	0.010	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+Lr	0.015	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+S	0.132	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750Lr	0.013	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750S	0.102	PASS	0.00 ft	0.000	PASS	0.00 ft
+0.60D	0.006	PASS	0.00 ft	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
D Only	4.026								
+D+Lr	5.866								
+D+S	53.136								
+D+0.750Lr	5.406								
+D+0.750S	40.859								
+0.60D	2.416								
Lr Only	1.840								
S Only	49.110								

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Steel Column

Description : B5 SUPPORT

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	53.136								
"	Minimum	1.840								
Reaction, X-X Axis	Maximum	4.026								
"	Minimum	4.026								
Reaction, Y-Y Axis	Maximum	4.026								
"	Minimum	4.026								
Reaction, X-X Axis	Maximum	4.026								
"	Minimum	4.026								
Reaction, Y-Y Axis	Maximum	4.026								
"	Minimum	4.026								
Moment, X-X Axis Ba	Maximum	4.026								
"	Minimum	4.026								
Moment, Y-Y Axis Ba	Maximum	4.026								
"	Minimum	4.026								
Moment, X-X Axis To	Maximum	4.026								
"	Minimum	4.026								
Moment, Y-Y Axis To	Maximum	4.026								
"	Minimum	4.026								

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
Lr Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
S Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Steel Section Properties : HSS12x6x5/8

Depth	=	12.000 in	I xx	=	321.00 in^4	J	=	271.000 in^4
Design Thick	=	0.581 in	S xx	=	53.40 in^3	Cw	=	71.10 in^6
Width	=	6.000 in	R xx	=	4.140 in			
Wall Thick	=	0.624 in	Zx	=	68.800 in^3			
Area	=	18.700 in^2	I yy	=	107.000 in^4	C	=	71.100 in^3
Weight	=	67.618 plf	S yy	=	35.500 in^3			
			R yy	=	2.390 in			
			Zy	=	42.100 in^3			
Ycg	=	0.000 in						

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Steel Column

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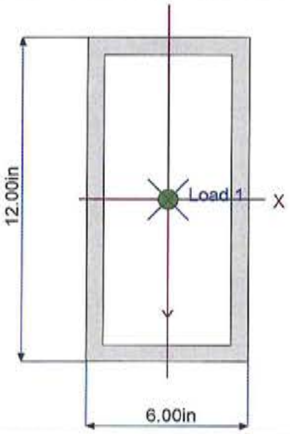
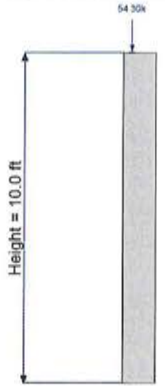
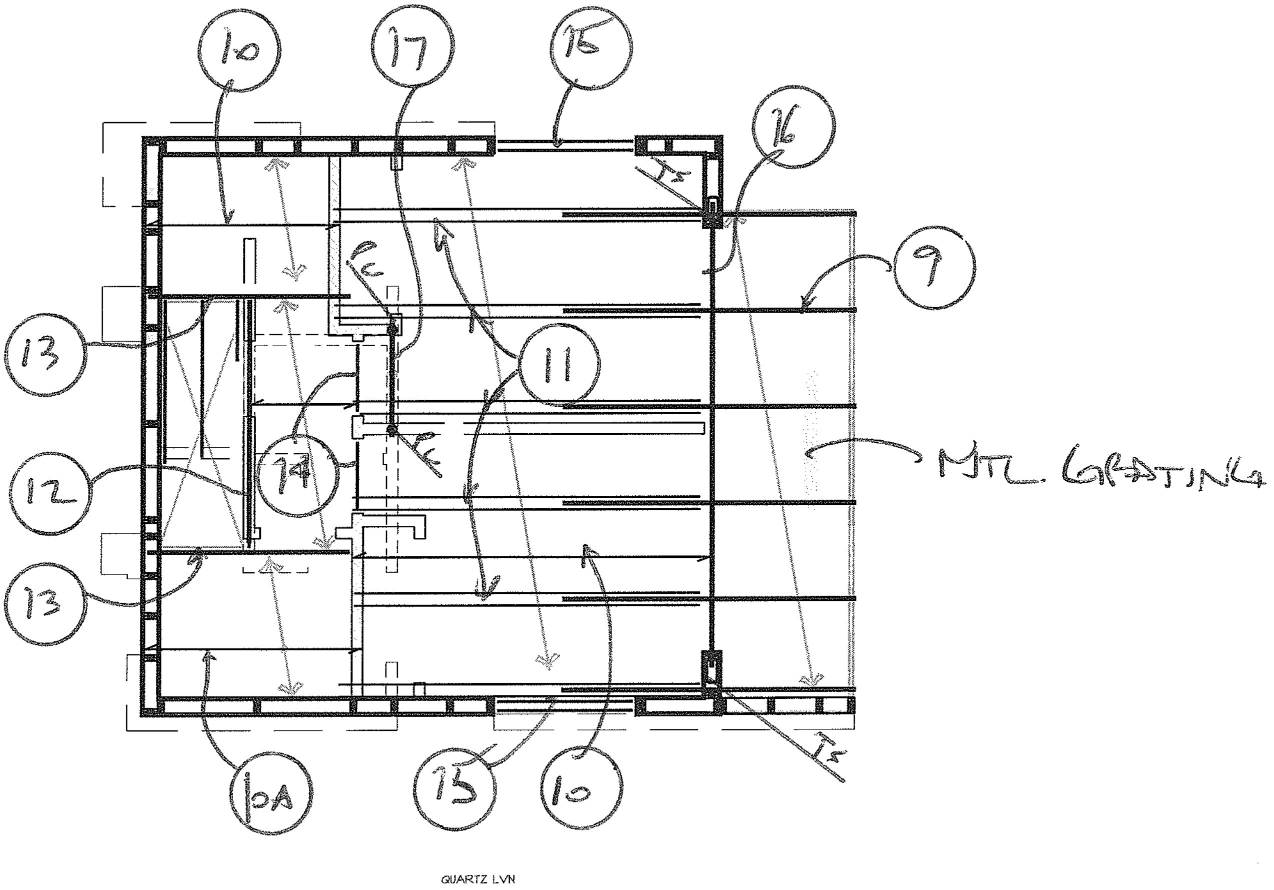
Description : B5 SUPPORT





Loads are total entered value. Arrows do not reflect absolute direction.

PROJECT FELDSPAR LOWER
JOB 2017-0610
CLIENT VILLAGE NEST
ADDRESS EDEN, UTAH
DESIGN HUUM
ENGINEER J.H.
DATE 6/16/2017
RICHMOND PE 49628 HOFFMAYER SE 3935
SHEET 79. OF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MTL. GRATING
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CL
TS
TS
QUARTZ LVN

○ : BEAM #

2ND FLOOR FRAMING PLAN

QUARTZ - LOWER
N.T.S.

PROJECT VILLAGE NEST JOB 2017-0610
CLIENT -
ADDRESS EDEN, UT
DESIGN HUUM
ENGINEER J.H. DATE 6/14/2017
RICHMOND PE 49628 HOFFMAYER SE 3435 SHEET 80 OF

METAL GRATING:

D = 12 PSF
L = 60 PSF
S = 264 PSF – (OPENED GRATING – DON'T DBL SNOW)

LOAD COMBINATIONS:

D + L = 72 PSF
D + S = 276 PSF ← GOVERNS
D + 0.75L + 0.75S = 266 PSF

SPAN	$M = w\ell^2/8$ (1-SPAN OR 2-SPANS)
32"	2944 IN-LB
48"	6624 IN-LB
64"	11776 IN-LB

MAT.	ALUMINUM	STEEL
E =	10000 KSI	29000 KSI
F =	12000 PSI	18000 PSI MIN

FSK > M ¢ EIK > $5w\ell^3/4608 \times 240$ ← SPAN IN INCHES

↑ BARS/FT ↑ 276 PSF

$K = 12/(7/16) = 27.4$

$$\left.\begin{array}{l} 12000(S)27.4 > 2944 \rightarrow S = 0.009\ \text{IN}^3 \\ 12000(S)27.4 > 6624 \rightarrow S = 0.020\ \text{IN}^3 \\ 12000(S)27.4 > 11776 \rightarrow S = 0.035\ \text{IN}^3 \end{array}\right\} \text{AL.}$$

$^{12}/_{1\text{-}8}(0.009) = 0.0061\ \text{IN}^3$
$^{12}/_{1\text{-}8}(0.02) = 0.0133\ \text{IN}^3$
$^{12}/_{1\text{-}8}(0.035) = 0.0233\ \text{IN}^3$
} STEEL.

$$IK > 5(276)\,\ell^3/460BE$$

$$I > 0.0109\,\ell^3/E$$

$0.0109(32^3)/10E6 = 0.00004\ \text{IN}^4$
$0.0109(48^3)/10E6 = 0.0001\ \text{IN}^4$
$0.0109(64^3)/10E6 = 0.0003\ \text{IN}^4$
} AL

$^{10}/_{29}(0.00004) = 0.000013\ \text{IN}^4$
$^{10}/_{29}(0.0001) = 0.000035\ \text{IN}^4$
$^{10}/_{29}(0.0003) = 0.0001\ \text{IN}^4$
} STEEL

1" DEEP × 3/16" AL → $S = 0.0313\ \text{IN}^3$
$I = 0.0156\ \text{IN}^4$

* USE UP TO 48" SPANS W/ BARS @ 7/16"

1" DEEP × 3/16" STEEL → $S = 0.0313\ \text{IN}^3$
$I = 0.0156\ \text{IN}^4$

* USE UP TO 64" SPANS W/ BARS @ 7/16"

1" × 3/16" AL. @ 7/16" o.c. GRATING
W/ SPANS @ 48" o.c.

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Steel Beam

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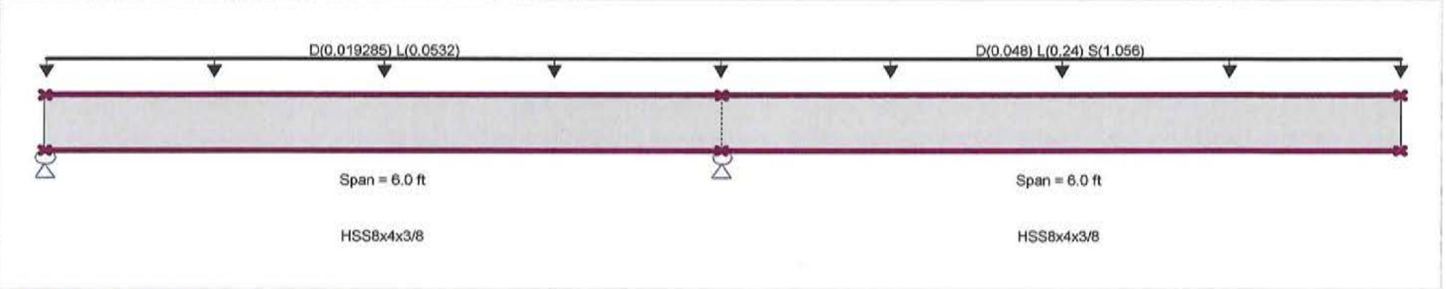
Description : B9 - DECK SUPPORT

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.01450, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)
Load for Span Number 2
Uniform Load : D = 0.0120, L = 0.060, S = 0.2640 ksf, Tributary Width = 4.0 ft, (DECK)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.434 : 1	Maximum Shear Stress Ratio =	0.078 : 1
Section used for this span	HSS8x4x3/8	Section used for this span	HSS8x4x3/8
Ma : Applied	20.367 k-ft	Va : Applied	6.789 k
Mn / Omega : Allowable	46.906 k-ft	Vn/Omega : Allowable	87.183 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	6.000ft	Location of maximum on span	6.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.089 in	Ratio =	1,616 >=360.
Max Upward Transient Deflection	-0.009 in	Ratio =	7,748 >=360.
Max Downward Total Deflection	0.397 in	Ratio =	363 >=240.
Max Upward Total Deflection	-0.043 in	Ratio =	1675 >=240.

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 6.00 ft	1	0.029	0.005		-1.36	1.36	78.33	46.91	1.00	1.00	0.45	145.60	87.18
Dsgn. L = 6.00 ft	2	0.029	0.005		-1.36	1.36	78.33	46.91	1.00	1.00	0.45	145.60	87.18
+D+L													
Dsgn. L = 6.00 ft	1	0.121	0.022		-5.68	5.68	78.33	46.91	1.00	1.00	1.89	145.60	87.18
Dsgn. L = 6.00 ft	2	0.121	0.022		-5.68	5.68	78.33	46.91	1.00	1.00	1.89	145.60	87.18
+D+S													
Dsgn. L = 6.00 ft	1	0.434	0.078		-20.37	20.37	78.33	46.91	1.00	1.00	6.79	145.60	87.18
Dsgn. L = 6.00 ft	2	0.434	0.078		-20.37	20.37	78.33	46.91	1.00	1.00	6.79	145.60	87.18
+D+0.750L													
Dsgn. L = 6.00 ft	1	0.098	0.018		-4.60	4.60	78.33	46.91	1.00	1.00	1.53	145.60	87.18
Dsgn. L = 6.00 ft	2	0.098	0.018		-4.60	4.60	78.33	46.91	1.00	1.00	1.53	145.60	87.18
+D+0.750L+0.750S													
Dsgn. L = 6.00 ft	1	0.402	0.072		-18.85	18.85	78.33	46.91	1.00	1.00	6.28	145.60	87.18
Dsgn. L = 6.00 ft	2	0.402	0.072		-18.85	18.85	78.33	46.91	1.00	1.00	6.28	145.60	87.18
+0.60D													
Dsgn. L = 6.00 ft	1	0.017	0.003		-0.82	0.82	78.33	46.91	1.00	1.00	0.27	145.60	87.18
Dsgn. L = 6.00 ft	2	0.017	0.003		-0.82	0.82	78.33	46.91	1.00	1.00	0.27	145.60	87.18

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+0.750L+0.750S	-0.0430	3.504

Steel Beam

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Description : B9 - DECK SUPPORT

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S	2	0.3967	6.000		0.0000	3.504

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	-3.254	10.324	
Overall MINimum	-0.052	0.492	
D Only	-0.086	0.820	
+D+L	-0.647	3.139	
+D+S	-3.254	10.324	
+D+0.750L	-0.506	2.559	
+D+0.750L+0.750S	-2.882	9.687	
+0.60D	-0.052	0.492	
L Only	-0.560	2.320	
S Only	-3.168	9.504	

84

Wood Beam

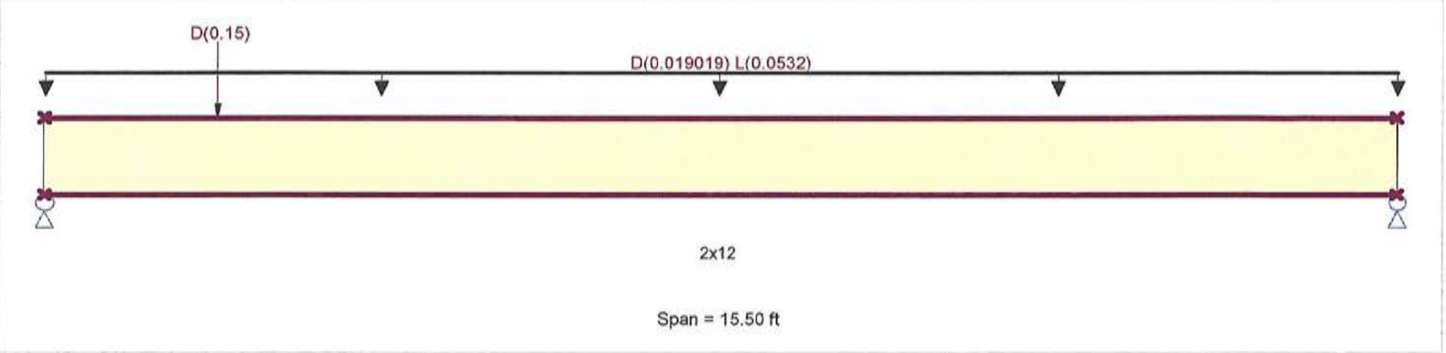
Description : B10 - FLOOR JOISTS w/ PARTITION

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb - Tension	875.0 psi	*E : Modulus of Elasticity*	
Load Combination	ASCE 7-10	Fb - Compr	875.0 psi	Ebend- xx	1,400.0 ksi
		Fc - Prll	1,150.0 psi	Eminbend - xx	510.0 ksi
Wood Species	: Spruce - Pine - Fir	Fc - Perp	425.0 psi		
Wood Grade	: No. 1/No. 2	Fv	135.0 psi		
		Ft	450.0 psi	Density	26.210 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.

Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)
Point Load : D = 0.150 k @ 2.0 ft, (WALL)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.875 1	**Maximum Shear Stress Ratio**	=	0.412 : 1
Section used for this span		2x12	Section used for this span		2x12
fb : Actual	=	880.42psi	fv : Actual	=	55.55 psi
FB : Allowable	=	1,006.25psi	Fv : Allowable	=	135.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	7.467ft	Location of maximum on span	=	0.000ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.279 in	Ratio =	666 >=360.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360.0
Max Downward Total Deflection	0.409 in	Ratio =	454 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.690	0.579
Overall MINimum	0.167	0.100
D Only	0.278	0.167
+D+L	0.690	0.579
+D+0.750L	0.587	0.476
+0.60D	0.167	0.100
L Only	0.412	0.412

Wood Beam

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Description : B10A - FLOOR JOISTS ABV. LANDING

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Spruce - Pine - Fir
Wood Grade : No. 1/No. 2

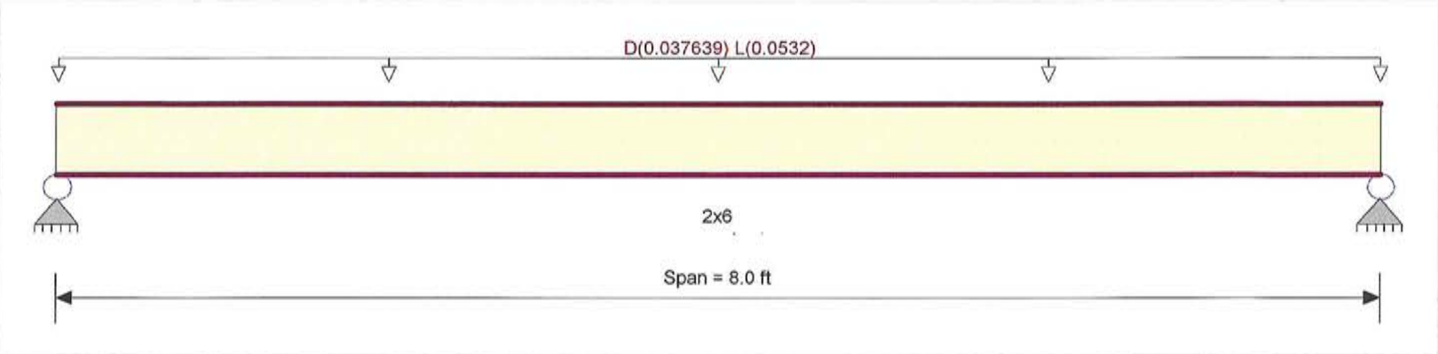
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	875.0 psi
Fb -	875.0 psi
Fc - Prll	1,150.0 psi
Fc - Perp	425.0 psi
Fv	135.0 psi
Ft	450.0 psi

E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	26.210 pcf

Repetitive Member Stress Increase



Applied Loads

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

Uniform Load : D = 0.02830, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.882 1	Maximum Shear Stress Ratio	=	0.436 : 1
Section used for this span		2x6	Section used for this span		2x6
fb : Actual	=	1,153.13psi	fv : Actual	=	58.83 psi
FB : Allowable	=	1,308.13psi	Fv : Allowable	=	135.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	4.000ft	Location of maximum on span	=	7.562 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.169 in	Ratio =	566 >=360.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360.0
Max Downward Total Deflection	0.289 in	Ratio =	331 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.363	0.363
Overall MINimum	0.090	0.090
D Only	0.151	0.151
+D+L	0.363	0.363
+D+0.750L	0.310	0.310
+0.60D	0.090	0.090
L Only	0.213	0.213

Wood Beam

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : B11 - FLOOR JOISTS w/ DECK SUPPORT

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination :ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

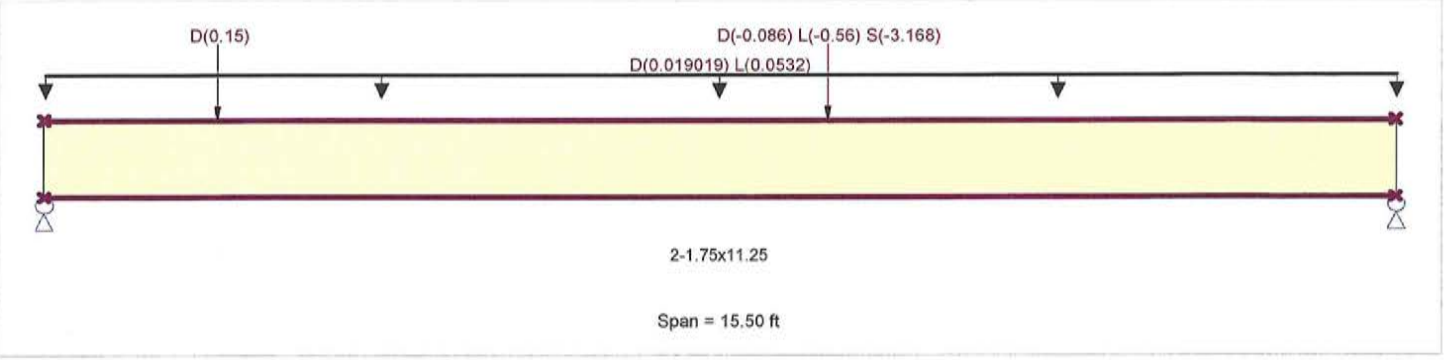
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb - Tension	2,600.0 psi
Fb - Compr	2,600.0 psi
Fc - Prll	2,510.0 psi
Fc - Perp	750.0 psi
Fv	285.0 psi
Ft	1,555.0 psi

E : Modulus of Elasticity

Ebend- xx	1,900.0ksi
Eminbend - xx	965.71ksi
Density	42.0pcf

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 = 0.01430, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)
Point Load : D = 0.150 k @ 2.0 ft, (WALL)
Point Load : D = -0.0860, L = -0.560, S = -3.168 k @ 9.0 ft, (B9)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.532 1	**Maximum Shear Stress Ratio**	=	0.213 : 1
Section used for this span		2-1.75x11.25	Section used for this span		2-1.75x11.25
fb : Actual	=	1,829.45psi	fv : Actual	=	69.73 psi
FB : Allowable	=	3,438.50psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.995ft	Location of maximum on span	=	9.051 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.002 in	Ratio =	92810 >=360.
Max Upward Transient Deflection	-0.005 in	Ratio =	33909 >=360.
Max Downward Total Deflection	0.046 in	Ratio =	4026 >=240.
Max Upward Total Deflection	-0.350 in	Ratio =	531 >=240.

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	-1.329	-1.839
Overall MINimum	0.177	0.087
D Only	0.331	0.206
+D+L	0.508	0.293
+D+S	-0.998	-1.634
+D+0.750L	0.464	0.271
+D+0.750L+0.750S	-0.532	-1.108
+0.60D	0.199	0.123
L Only	0.177	0.087
S Only	-1.329	-1.839

Wood Beam

Lic. # : KW-06002886

Description : B12 - FLOOR SUPPORT

CODE REFERENCES

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

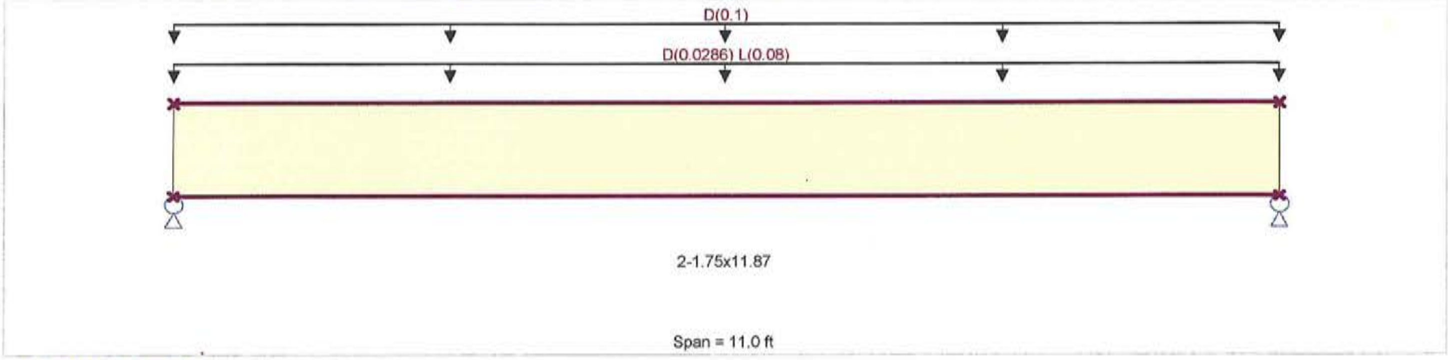
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

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

Fb - Tension	2,600.0 psi	*E : Modulus of Elasticity*	
Fb - Compr	2,600.0 psi	Ebend- xx	1,900.0 ksi
Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Fc - Perp	750.0 psi		
Fv	285.0 psi		
Ft	1,555.0 psi	Density	42.0 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 2.0 ft, (FLOOR)
Uniform Load : D = 0.10 , Tributary Width = 1.0 ft, (WALL)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.187: 1	Maximum Shear Stress Ratio	=	0.127 : 1
Section used for this span		2-1.75x11.87	Section used for this span		2-1.75x11.87
fb : Actual	=	487.01 psi	fv : Actual	=	36.14 psi
FB : Allowable	=	2,600.00 psi	Fv : Allowable	=	285.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	5.500ft	Location of maximum on span	=	10.036 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.029 in	Ratio =	4621 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.079 in	Ratio =	1674 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.214	1.214
Overall MINimum	0.440	0.440
D Only	0.774	0.774
+D+L	1.214	1.214
+D+0.750L	1.104	1.104
+0.60D	0.464	0.464
L Only	0.440	0.440

CC

Wood Beam

Description : B13 - FLOOR SUPPORT ABV. STAIR

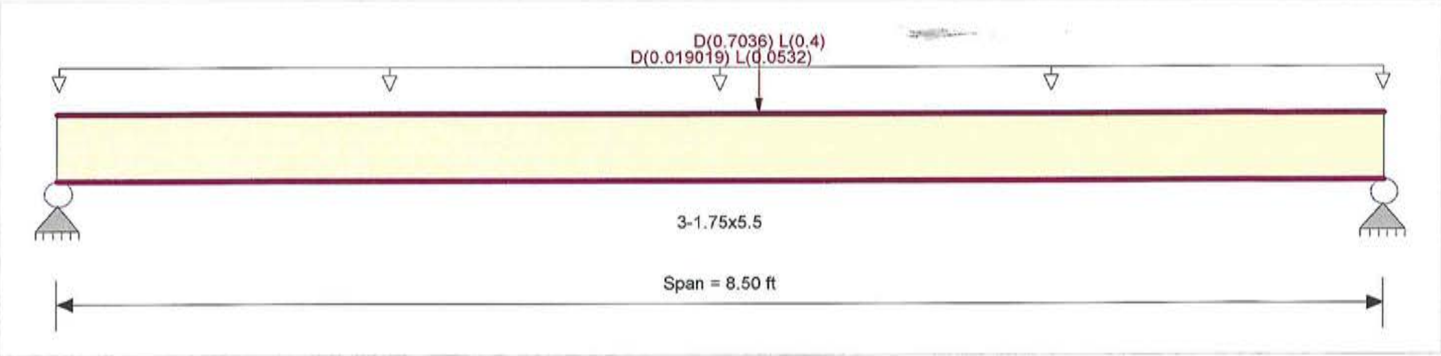
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,600.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-10	Fb -	2,600.0 psi	Ebend- xx	1,900.0 ksi
		Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Wood Species	: Trus Joist	Fc - Perp	750.0 psi		
Wood Grade	: MicroLam LVL 1.9 E	Fv	285.0 psi		
		Ft	1,555.0 psi	Density	42.0 pcf

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



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)
Point Load : D = 0.7036, L = 0.40 k @ 4.50 ft, (B12)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.534 1	**Maximum Shear Stress Ratio**	=	0.163 : 1
Section used for this span		3-1.75x5.5	Section used for this span		3-1.75x5.5
fb : Actual	=	1,388.15psi	fv : Actual	=	46.34 psi
FB : Allowable	=	2,600.00psi	Fv : Allowable	=	285.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	4.498ft	Location of maximum on span	=	8.066 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.109 in	Ratio =	932 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.245 in	Ratio =	415 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.862	0.927
Overall MINimum	0.269	0.293
D Only	0.448	0.489
+D+L	0.862	0.927
+D+0.750L	0.758	0.818
+0.60D	0.269	0.293
L Only	0.414	0.438

C9

Wood Beam

Description : B14 - HEADERS

CODE REFERENCES

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

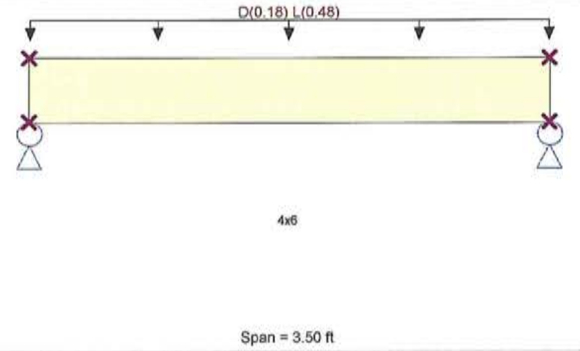
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Spruce - Pine - Fir
Wood Grade : No. 1/No. 2

Beam Bracing : Completely Unbraced

Fb - Tension	875.0 psi	*E : Modulus of Elasticity*	
Fb - Compr	875.0 psi	Ebend- xx	1,400.0ksi
Fc - Prll	1,150.0 psi	Eminbend - xx	510.0ksi
Fc - Perp	425.0 psi		
Fv	135.0 psi		
Ft	450.0 psi	Density	26.210pcf



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, L = 0.040 ksf, Tributary Width = 12.0 ft, (FLOOR)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.610: 1	**Maximum Shear Stress Ratio**	=	0.499 : 1
Section used for this span		4x6	Section used for this span		4x6
fb : Actual	=	690.92psi	fv : Actual	=	67.36 psi
FB : Allowable	=	1,133.19psi	Fv : Allowable	=	135.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	1.750ft	Location of maximum on span	=	3.053 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.024 in	Ratio =	1750 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.033 in	Ratio =	1266 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.161	1.161
Overall MINimum	0.193	0.193
D Only	0.321	0.321
+D+L	1.161	1.161
+D+0.750L	0.951	0.951
+0.60D	0.193	0.193
L Only	0.840	0.840

RICHMOND HOFFMAYER INC.
STRUCTURAL ENGINEERING

Project Title: VILLAGE NEST
Engineer: JJH
Project Descr: MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 26 JUN 2017, 2:51PM

Wood Beam

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Description : B15 - HEADERS

CODE REFERENCES

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

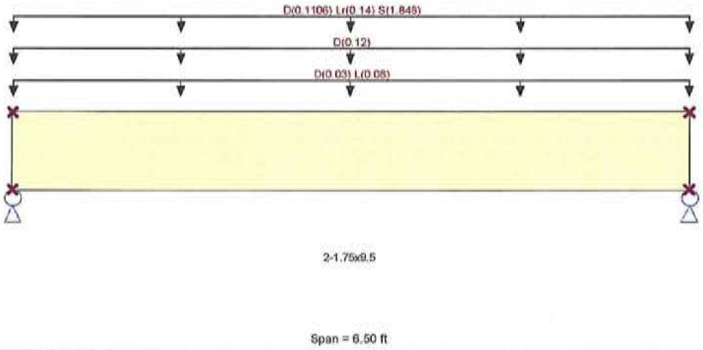
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

Beam Bracing : Completely Unbraced

Fb - Tension	2,600.0 psi	*E : Modulus of Elasticity*	
Fb - Compr	2,600.0 psi	Ebend- xx	1,900.0 ksi
Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Fc - Perp	750.0 psi		
Fv	285.0 psi		
Ft	1,555.0 psi	Density	42.0 pcf



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, L = 0.040 ksf, Tributary Width = 2.0 ft, (FLOOR)
Uniform Load : D = 0.120 , Tributary Width = 1.0 ft, (EXTERIOR WALL)
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 7.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.872 1	Maximum Shear Stress Ratio	=	0.719 : 1
Section used for this span		2-1.75x9.5	Section used for this span		2-1.75x9.5
fb : Actual	=	2,550.00psi	fv : Actual	=	235.77 psi
FB : Allowable	=	2,925.88psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.250ft	Location of maximum on span	=	5.717ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.157 in	Ratio =	496>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0<360
Max Downward Total Deflection	0.180 in	Ratio =	433>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0<240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	6.884	6.884
Overall MINimum	0.260	0.260
D Only	0.878	0.878
+D+L	1.138	1.138
+D+Lr	1.333	1.333
+D+S	6.884	6.884
+D+0.750Lr+0.750L	1.415	1.415
+D+0.750L+0.750S	5.578	5.578
+0.60D	0.527	0.527
Lr Only	0.455	0.455

91

Wood Beam

Lic. # : KW-06002886

Description : B15 - HEADERS

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
L Only	0.260	0.260
S Only	6.006	6.006

92

Steel Beam

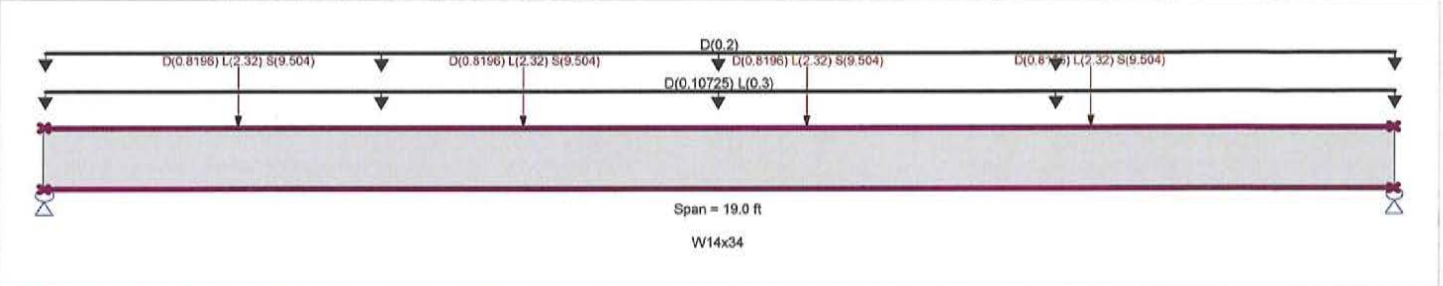
Description : B16 - BALCONY SUPPORT

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		


D(0.2)
D(0.8196) L(2.32) S(9.504)
D(0.8196) L(2.32) S(9.504)
D(0.8196) L(2.32) S(9.504)
D(0.8196) L(2.32) S(9.504)
D(0.10725) L(0.3)
Span = 19.0 ft
W14x34

Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 7.50 ft, (FLOOR)
Uniform Load : D = 0.20 k/ft, Tributary Width = 1.0 ft, (GLASS)
Point Load : D = 0.8196, L = 2.320, S = 9.504 k @ 2.750 ft, (BALCONY - B9)
Point Load : D = 0.8196, L = 2.320, S = 9.504 k @ 6.750 ft, (BALCONY - B9)
Point Load : D = 0.8196, L = 2.320, S = 9.504 k @ 10.750 ft, (BALCONY - B9)
Point Load : D = 0.8196, L = 2.320, S = 9.504 k @ 14.750 ft, (BALCONY - B9)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.981 : 1	Maximum Shear Stress Ratio =	0.329 : 1
Section used for this span	W14x34	Section used for this span	W14x34
Ma : Applied	133.584 k-ft	Va : Applied	26.284 k
Mn / Omega : Allowable	136.228 k-ft	Vn/Omega : Allowable	79.80 k
Load Combination	+D+0.750L+0.750S	Load Combination	+D+0.750L+0.750S
Location of maximum on span	10.749ft	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	0.259 in	Ratio =	878 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.421 in	Ratio =	541 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Maximum Forces & Stresses for Load Combinations

Load Combination Segment Length	Span #	Max Stress Ratios M	 V	Summary of Moment Values Mmax +	 Mmax -	 Ma Max	 Mnx	 Mnx/Omega	 Cb	 Rm	Summary of Shear Values Va Max	 Vnx	 Vnx/Omega
D Only													
Dsgn. L = 19.00 ft	1	0.179	0.063	24.44		24.44	227.50	136.23	1.00	1.00	5.01	119.70	79.80
+D+L													
Dsgn. L = 19.00 ft	1	0.467	0.161	63.66		63.66	227.50	136.23	1.00	1.00	12.87	119.70	79.80
+D+S													
Dsgn. L = 19.00 ft	1	0.960	0.320	130.73		130.73	227.50	136.23	1.00	1.00	25.52	119.70	79.80
+D+0.750L													
Dsgn. L = 19.00 ft	1	0.395	0.137	53.85		53.85	227.50	136.23	1.00	1.00	10.90	119.70	79.80
+D+0.750L+0.750S													
Dsgn. L = 19.00 ft	1	0.981	0.329	133.58		133.58	227.50	136.23	1.00	1.00	26.28	119.70	79.80
+0.60D													
Dsgn. L = 19.00 ft	1	0.108	0.038	14.66		14.66	227.50	136.23	1.00	1.00	3.01	119.70	79.80

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.4214	9.500		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	26.284	23.225

Steel Beam

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ENERCALC, INC. 1983-2017, Build:6.17.3.17, Ver:6.17.3.17

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : B16 - BALCONY SUPPORT

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MINimum	3.006	2.851
D Only	5.010	4.752
+D+L	12.867	11.875
+D+S	25.519	22.259
+D+0.750L	10.903	10.094
+D+0.750L+0.750S	26.284	23.225
+0.60D	3.006	2.851
L Only	7.856	7.124
S Only	20.509	17.507

Steel Beam

File = d:\ENERCALC Projects\2017-0610.ec6
ENERCALC, INC. 1983-2017, Build:6.17.3.17, Ver:6.17.3.17

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

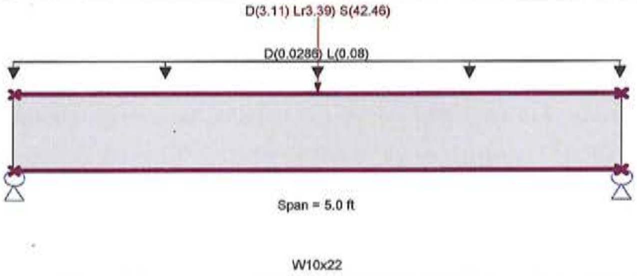
Description : B17 - RIDGE SUPPORT

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 2.0 ft, (FLOOR)
Point Load : D = 3.110, Lr = 3.390, S = 42.460 k @ 2.50 ft, (RIDGE B4 ABV.)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.881 : 1	Maximum Shear Stress Ratio =	0.468 : 1
Section used for this span	W10x22	Section used for this span	W10x22
Ma : Applied	57.121 k-ft	Va : Applied	22.912 k
Mn / Omega : Allowable	64.870 k-ft	Vn/Omega : Allowable	48.960 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	2.500ft	Location of maximum on span	5.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.004 in	Ratio =	13,401 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.009 in	Ratio =	6824 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Maximum Forces & Stresses for Load Combinations

Load Combination			Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
	Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only	Dsgn. L = 5.00 ft	1	0.062	0.034	4.05		4.05	108.33	64.87	1.00	1.00	1.68	73.44	48.96
+D+L	Dsgn. L = 5.00 ft	1	0.066	0.038	4.30		4.30	108.33	64.87	1.00	1.00	1.88	73.44	48.96
+D+Lr	Dsgn. L = 5.00 ft	1	0.128	0.069	8.28		8.28	108.33	64.87	1.00	1.00	3.38	73.44	48.96
+D+S	Dsgn. L = 5.00 ft	1	0.881	0.468	57.12		57.12	108.33	64.87	1.00	1.00	22.91	73.44	48.96
+D+0.750Lr+0.750L	Dsgn. L = 5.00 ft	1	0.114	0.063	7.41		7.41	108.33	64.87	1.00	1.00	3.10	73.44	48.96
+D+0.750L+0.750S	Dsgn. L = 5.00 ft	1	0.679	0.363	44.04		44.04	108.33	64.87	1.00	1.00	17.75	73.44	48.96
+0.60D	Dsgn. L = 5.00 ft	1	0.037	0.021	2.43		2.43	108.33	64.87	1.00	1.00	1.01	73.44	48.96

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0088	2.514		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	22.912	22.912
Overall MINimum	0.200	0.200
D Only	1.682	1.682

95

Steel Beam

Description : B17 - RIDGE SUPPORT

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D+L	1.882	1.882
+D+Lr	3.377	3.377
+D+S	22.912	22.912
+D+0.750Lr+0.750L	3.103	3.103
+D+0.750L+0.750S	17.754	17.754
+0.60D	1.009	1.009
Lr Only	1.695	1.695
L Only	0.200	0.200
S Only	21.230	21.230

Steel Column

Description : B17 SUPPORT

Code References

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Steel Section Name :	Pipe4STD
Analysis Method :	Allowable Strength
Steel Stress Grade	
Fy : Steel Yield	36.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height	10.0 ft
Top & Bottom Fixity	Top & Bottom Pinned

Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 10.0 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for Y-Y Axis buckling = 10.0 ft, K = 1.0

Applied Loads

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

Column self weight included : 108.0 lbs * Dead Load Factor
AXIAL LOADS . . .
B17: Axial Load at 10.0 ft, D = 1.682, LR = 1.695, L = 0.20, S = 21.230 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.5031 : 1
	Load Combination	+D+S
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Pa : Axial	23.020 k
	Pn / Omega : Allowable	45.760 k
	Ma-x : Applied	0.0 k-ft
	Mn-x / Omega : Allowable	7.275 k-ft
	Ma-y : Applied	0.0 k-ft
	Mn-y / Omega : Allowable	7.275 k-ft
PASS	Maximum Shear Stress Ratio =	0.0 : 1
	Load Combination	
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Va : Applied	0.0 k
	Vn / Omega : Allowable	0.0 k

Maximum SERVICE Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum SERVICE Load Deflections . . .

Along Y-Y	0.0 in at	0.0ft	above base
for load combination :			
Along X-X	0.0 in at	0.0ft	above base
for load combination :			

Load Combination Results

	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
Load Combination	Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.039	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+L	0.043	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+Lr	0.076	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+S	0.503	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750Lr+0.750L	0.070	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750L+0.750S	0.390	PASS	0.00 ft	0.000	PASS	0.00 ft
+0.60D	0.023	PASS	0.00 ft	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

	Axial Reaction	X-X Axis Reaction	k	Y-Y Axis Reaction		Mx - End Moments	k-ft	My - End Moments	
Load Combination	@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
D Only	1.790								
+D+L	1.990								
+D+Lr	3.485								
+D+S	23.020								
+D+0.750Lr+0.750L	3.211								
+D+0.750L+0.750S	17.863								
+0.60D	1.074								

Steel Column

Description : B17 SUPPORT

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction	X-X Axis Reaction	k	Y-Y Axis Reaction		Mx - End Moments	k-ft	My - End Moments	
	@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
Lr Only	1.695								
L Only	0.200								
S Only	21.230								

Extreme Reactions

Item	Extreme Value	Axial Reaction	X-X Axis Reaction	k	Y-Y Axis Reaction		Mx - End Moments	k-ft	My - End Moments	
		@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
Axial @ Base	Maximum	23.020								
"	Minimum	0.200								
Reaction, X-X Axis	Maximum	1.790								
"	Minimum	1.790								
Reaction, Y-Y Axis	Maximum	1.790								
"	Minimum	1.790								
Reaction, X-X Axis	Maximum	1.790								
"	Minimum	1.790								
Reaction, Y-Y Axis	Maximum	1.790								
"	Minimum	1.790								
Moment, X-X Axis Ba	Maximum	1.790								
"	Minimum	1.790								
Moment, Y-Y Axis Ba	Maximum	1.790								
"	Minimum	1.790								
Moment, X-X Axis To	Maximum	1.790								
"	Minimum	1.790								
Moment, Y-Y Axis To	Maximum	1.790								
"	Minimum	1.790								

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+L	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr+0.750L	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750L+0.750S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
Lr Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
L Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
S Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Steel Section Properties : Pipe4STD

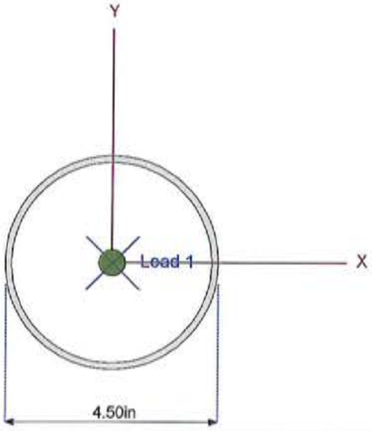
Depth	=	4.500 in	I xx	=	6.82 in^4	J	=	13.600 in^4
			S xx	=	3.03 in^3			
Diameter	=	4.500 in	R xx	=	1.510 in			
Wall Thick	=	0.237 in	Zx	=	4.050 in^3			
Area	=	2.960 in^2	I yy	=	6.820 in^4			
Weight	=	10.800 plf	S yy	=	3.030 in^3			
			R yy	=	1.510 in			

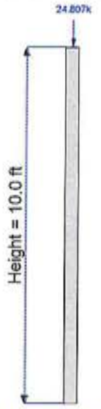
Ycg = 0.000 in

9e

Steel Column

Description : B17 SUPPORT





Loads are total entered value. Arrows do not reflect absolute direction.

SIP WALL INFORMATION:

ICC-ES ESR-1882

TRY TYPE "S" PANELS

SEISMIC DESIGN CATEGORY D - PERIODIC SPECIAL INSPECTION REQUIRED FOR FASTENING AND ANCHORING OF SHEAR WALLS.

TRY 7-1/4" PANEL CORE THICKNESS.

→ MAX HEIGHT = 10'-6"

$W_{UNIF\ ROOF} = (15.8_D + 20_{L_r} + 264_S)\frac{14'}{2} = \underline{\underline{2099\ PLF}}$

$W_{UNIF\ 2ND} = (14.3_D + 40_L)\frac{8'}{2} = \underline{\underline{217\ PLF}}$

$W_{MAX} = 2099 + 217 = \underline{\underline{2316\ PLF}}$

SEE TABLE 5 NEXT PAGE, CAP = $\underline{\underline{4326\ PLF}}$ ✓

→ MAX HEADER SPAN W/O POINT LOAD = 4.5'

$W_{MAX} \leq \underline{\underline{2099\ PLF}}$

SEE TABLE 14, PG#101, CAP = $\underline{\underline{2620\ PLF}}$ ✓

DBL TRIMMER, PREMIER INSUL-BEAM II

TABLE 4—UNIFORM TRANSVERSE LOADS FOR FACE SUPPORTED PREMIER TYPE L PANELS[1,2,3] (psf)

PANEL CORE THICKNESS (inches)	DEFLECTION	PANEL SPAN									
		4 ft[4]	8 ft	10 ft	12 ft	14 ft	16 ft	18 ft	20 ft	22 ft	24 ft
$3^{1}/_{2}$[5]	$^{L}/_{360}$	98	45	32	24	16	11	----	----	----	----
	$^{L}/_{240}$	215	67	47	34	24	16	----	----	----	----
	$^{L}/_{180}$	298*	90	61	44	34	22	----	----	----	----
$5^{1}/_{2}$[6]	$^{L}/_{360}$	241	128	57	41	33	25	20	15	----	----
	$^{L}/_{240}$	288*	182*	86	60	49	37	29	22	----	----
	$^{L}/_{180}$	288*	182*	112*	79	65	49	39	29	----	----
$7^{1}/_{4}$[7]	$^{L}/_{360}$	241	168	80	65	54	42	33	24	----	----
	$^{L}/_{240}$	288*	188*	126	99	81	61	49	34	----	----
	$^{L}/_{180}$	288*	188*	133*	117*	105	80	62	44	----	----
$9^{1}/_{4}$[8]	$^{L}/_{360}$	274	188*	116	100	80	62	47	35	32	28
	$^{L}/_{240}$	326*	188*	147*	134*	120	92	70	52	46	41
	$^{L}/_{180}$	326*	188*	147*	134*	121*	108*	93	68	61	53
$11^{1}/_{4}$[8]	$^{L}/_{360}$	327*	188*	167*	140	116	90	75	57	47	36
	$^{L}/_{240}$	327*	188*	167*	153*	132*	110*	97*	83*	69	53
	$^{L}/_{180}$	327*	188*	167*	153*	132*	110*	97*	83*	83*	70

For **SI:** 1 inch = 25.4 mm, 1 psf = 47.9 Pa, 1 foot = 304.8 mm.

[1] Floor panels must have a minimum $^{3}/_{4}$-inch-thick top skin or a minimum $^{7}/_{16}$-inch-thick top skin overlaid with minimum $^{7}/_{16}$-inch-thick finish flooring perpendicular to the panels.

[2] The tabulated values are for roof and floor panels installed with simply supported single span conditions with panels supported at each end on a minimum $1^{1}/_{2}$-inch-wide continuous support in contact with the panel face. Tabulated values are applicable to panels installed with the strong axis of the OSB panel facers parallel to the panel span.

[3] Allowable loads with an asterisk, *, indicates a capacity based on the average peak test load divided by 3.

[4] Panels spanning 4 feet shall be a minimum of 8 feet long spanning a minimum of two 4 foot spans. No single span conditions must be permitted.

[5] $3^{1}/_{2}$-inch thick core panels must be limited to a maximum span of 10 feet when used in roof applications.

[6] $5^{1}/_{2}$-inch thick core panels must be limited to a maximum span of 14 feet when used in roof applications.

[7] $7^{1}/_{4}$-inch thick core panels must be limited to a maximum span of 18 feet when used in roof applications.

[8] $9^{1}/_{4}$ and $11^{1}/_{4}$-inch thick core panels shall be limited to a maximum span of 20 feet when used in roof applications.

TABLE 5—ALLOWABLE UNIFORM AXIAL LOADS FOR PREMIER TYPE S PANELS (plf)[1,2,3,4]

PANEL CORE THICKNESS (inches)	PANEL SPAN					
	8 ft	10 ft	12 ft	16 ft	20 ft	24 ft
$3^{1}/_{2}$	3,500	2,555	2,450	2,120	----	----
$5^{1}/_{2}$	4,250	4,040	3,375	3,920	2,815	----
$7^{1}/_{4}$	4,915	4,325	4,475	4,195	3,495	3,065
$9^{1}/_{4}$	4,200	4,200	4,200	4,200	3,389	3,247
$11^{1}/_{4}$	3,890	3,890	3,890	3,890	3,890	3,333

For **SI:** 1 inch = 25.4 mm, 1 plf = 14.6 N/m, 1 foot = 304.8 mm.

[1] For the allowable axial load on the fire-resistance-rated assembly, see Section 4.2.6.

[2] For combined loads; requirements in Section 4.1 must be applied.

[3] The tabulated loads are uniform axial loads applied concentrically to the full thickness of the panels, including panel facings.

[4] The tabulated values are for panels installed with strong axis of the OSB panel facers parallel to the wall height (panel span) and on concrete foundations. The member, element, or structure supporting the bearing wall panels, as shown in Figures 7, 8 and 9, must be designed for the bearing stress of the wall panels to the satisfaction to the code official.

TABLE 6—ALLOWABLE UNIFORM AXIAL LOADS FOR PREMIER TYPE L PANELS (plf)[1,2,3,4]

PANEL CORE THICKNESS (inches)	PANEL SPAN					
	8 ft	10 ft	12 ft	16 ft	20 ft	24 ft
$3^{1}/_{2}$	4,725	3,905	3,095	2,350	----	----
$5^{1}/_{2}$	5,850	5,890	4,280	4,310	2,933	----
$7^{1}/_{4}$	6,850	6,110	5,555	5,180	4,835	4,080
$9^{1}/_{4}$	5,470	5,470	5,470	5,470	5,470	4,250
$11^{1}/_{4}$	4,500	4,333	4,167	3,750	3,750	3,333

For **SI:** 1 inch = 25.4 mm, 1 plf = 14.6 N/m, 1 foot = 304.8 mm.

[1] For the allowable axial load on fire-resistance-rated assembly, see Section 4.2.7.

[2] For combined loads; requirements in Section 4.1 must be applied.

[3] The tabulated loads are uniform axial loads applied concentrically to the full thickness of the panels, including panel facings.

[4] The tabulated values are for panels installed with strong axis of the OSB panel facers parallel to the wall height (panel span) on concrete foundations. The member, element, or structure supporting the bearing wall panels, as shown in Figures 7, 8 and 9, must be designed for the bearing stress of the wall panels to the satisfaction to the code official.

Table 13: Wind Speed vs. Pressure

Wall Loads (psf) - End Zone (Zone 5) for 100sf to 500 sf effective wind area												
Mean Roof Height (ft)	90 MPH			100 MPH			110 MPH			120 MPH		
	Exp B	Exp C	Exp D	Exp B	Exp C	Exp D	Exp B	Exp C	Exp D	Exp B	Exp C	Exp D
15	-15.1	-18.3	-22.2	-18.7	-22.6	-27.5	-22.6	-27.3	-33.2	-26.9	-32.5	-39.5
20	-15.1	-19.5	-23.4	-18.7	-24.1	-29.0	-22.6	-29.2	-35.0	-26.9	-34.7	-41.7
25	-15.1	-20.4	-24.3	-18.7	-25.2	-30.1	-22.6	-30.5	-36.4	-26.9	-36.3	-43.3
30	-15.1	-21.1	-25.1	-18.7	-26.2	-31.0	-22.6	-31.6	-37.5	-26.9	-37.7	-44.7
35	-15.9	-21.9	-25.7	-19.6	-27.1	-31.8	-23.7	-32.8	-38.4	-28.2	-39.0	-45.7
40	-16.5	-22.5	-26.3	-20.4	-27.9	-32.5	-24.6	-33.7	-39.3	-29.3	-40.1	-46.8
45	-16.9	-23.1	-26.9	-20.9	-28.6	-33.3	-25.3	-34.6	-40.2	-30.1	-41.2	-47.9
50	-17.5	-23.6	-27.3	-21.7	-29.2	-33.8	-26.2	-35.3	-40.9	-31.2	-42.0	-48.7
55	-18.0	-24.0	-27.8	-22.3	-29.7	-34.4	-26.9	-35.9	-41.6	-32.0	-42.8	-49.5
60	-18.4	-24.5	-28.2	-22.8	-30.3	-35.0	-27.6	-36.6	-42.3	-32.8	-43.6	-50.3
Net Design wind pressure	-15.1			-18.7			-22.6			-26.9		

Wall Loads (psf) - End Zone (Zone 5) for 100sf to 500sf effective wind area												
Mean Roof Height (ft)	130 MPH			140 MPH			150 MPH			170 MPH		
	Exp B	Exp C	Exp D	Exp B	Exp C	Exp D	Exp B	Exp C	Exp D	Exp B	Exp C	Exp D
15	-31.6	-38.2	-46.5	-36.7	-44.4	-53.9	-42.1	-50.9	-61.9	-54.1	-65.5	-79.5
20	-31.6	-40.8	-49.0	-36.7	-47.3	-56.9	-42.1	-54.3	-65.3	-54.1	-69.8	-83.9
25	-31.6	-42.7	-50.9	-36.7	-49.5	-59.1	-42.1	-56.8	-67.8	-54.1	-73.0	-87.1
30	-31.6	-44.2	-52.5	-36.7	-51.4	-60.9	-42.1	-58.9	-69.9	-54.1	-75.7	-89.8
35	-33.2	-45.8	-53.7	-38.5	-53.2	-62.4	-44.2	-61.1	-71.6	-56.8	-78.4	-92.0
40	-34.4	-47.1	-55.0	-40.0	-54.7	-63.9	-45.9	-62.7	-73.3	-59.0	-80.6	-94.1
45	-35.4	-48.3	-56.2	-41.1	-56.2	-65.3	-47.2	-64.4	-74.9	-60.6	-82.8	-96.3
50	-36.7	-49.3	-57.2	-42.6	-57.3	-66.4	-48.8	-65.7	-76.2	-62.8	-84.4	-97.9
55	-37.6	-50.2	-58.1	-43.7	-58.4	-67.5	-50.1	-66.9	-77.5	-64.4	-86.0	-99.5
60	-38.6	-51.2	-59.1	-44.8	-59.5	-68.6	-51.4	-68.2	-78.7	-66.0	-87.6	-101.2
Net Design wind pressure	-31.6			-36.7			-42.1			-54.1		

More information on this chart can be found in Technical Bulletin #15 (www.premiersips.com).

Table 14: Premier Insul-Beam II Header Loads (plf)

No. of Trimmer Studs	Deflection	Header Span (ft.)						
		2'	3'	4'	5'	6'	7'	8'
1	L/480	3150	2100	1575	1260	1050	900	788
	L/360	3150	2100	1575	1260	1050	900	788
	L/240	3150	2100	1575	1260	1050	900	788
2	L/480	6300	4200	3150	2520	2100	1800	1545
	L/360	6300	4200	3150	2520	2100	1800	1575
	L/240	6300	4200	3150	2520	2100	1800	1575

No. of Trimmer Studs	Deflection	Header Span (ft.)							
		9'	10'	11'	12'	13'	14'	15'	16'
1	L/480	700	630	573	458	360	288	234	193
	L/360	700	630	573	525	480	384	313	257
	L/240	700	630	573	525	485	450	420	386
2	L/480	1085	791	594	458	360	288	234	193
	L/360	1400	1055	792	610	480	384	313	257
	L/240	1400	1245	792	864	720	577	469	386

Values listed for each deflection represent the least value of the bearing capacity of the trimmer, shear or bending capacity of the header or the actual deflection at the design load.
Refer to Technical Bulletin #30 for supporting headers in Premier SIPs wall panels (www.premiersips.com).
Note: Trimmer stud design capacities must be reviewed.

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SINGLE TOP PLATE W/ CAP

Table 6: Maximum Allowable Axial Compression Point Loads (lbs) – Type S Panels[1,2,3,4]

Top Plate Configuration	1.5" Minimum Bearing Width	3" Minimum Bearing Width
Single 2x4 #2 or Better Hem-Fir Plate	2040	2450
Single 2x4 #2 or Better Hem-Fir Plate with 1-1/8 in. wide, 1.3E Rim Board Cap Plate	4030	4678

[1] Top plate secured to facings as required in Section 6.3
[2] Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.
[3] Concentrated loads shall be applied concentrically to the top of the panel.
[4] Tabulated values are based on the strong-axis of the facing material oriented parallel to the span direction.

2099 PLF × 16/12

= 2799 lb LOAD FROM RF

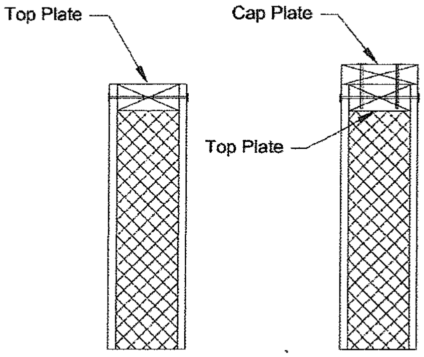


Figure 3: Top Plate Configurations

Table 7: Maximum Allowable Uniform SIP Header Vertical Loads (plf)
3-1/2 in. through 11-1/4 in. Core Thickness[1,2]

Header Depth[3] (in)	Deflection Limit[4]	Header Span (ft)			
		4	6	8	10
12	L/480	740	384	228	142
	L/360	740	384	229	142
	L/240	740	384	229	142
18	L/480	798	574	385	311
	L/360	798	574	385	311
	L/240	798	574	385	311
24	L/480	886	629	429	361
	L/360	886	629	429	361
	L/240	886	629	429	361

[1] Vertical loads only. Lateral loads shall be transferred to the edges of the openings through continuous plate(s) designed in accordance with accepted engineering practice. Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.
[2] Tabulated values are based on the strong-axis of the facing material oriented perpendicular to the direction of header span.
[3] Minimum depth of facing above opening.
[4] Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code.

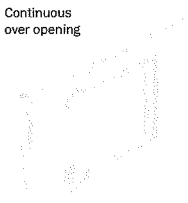


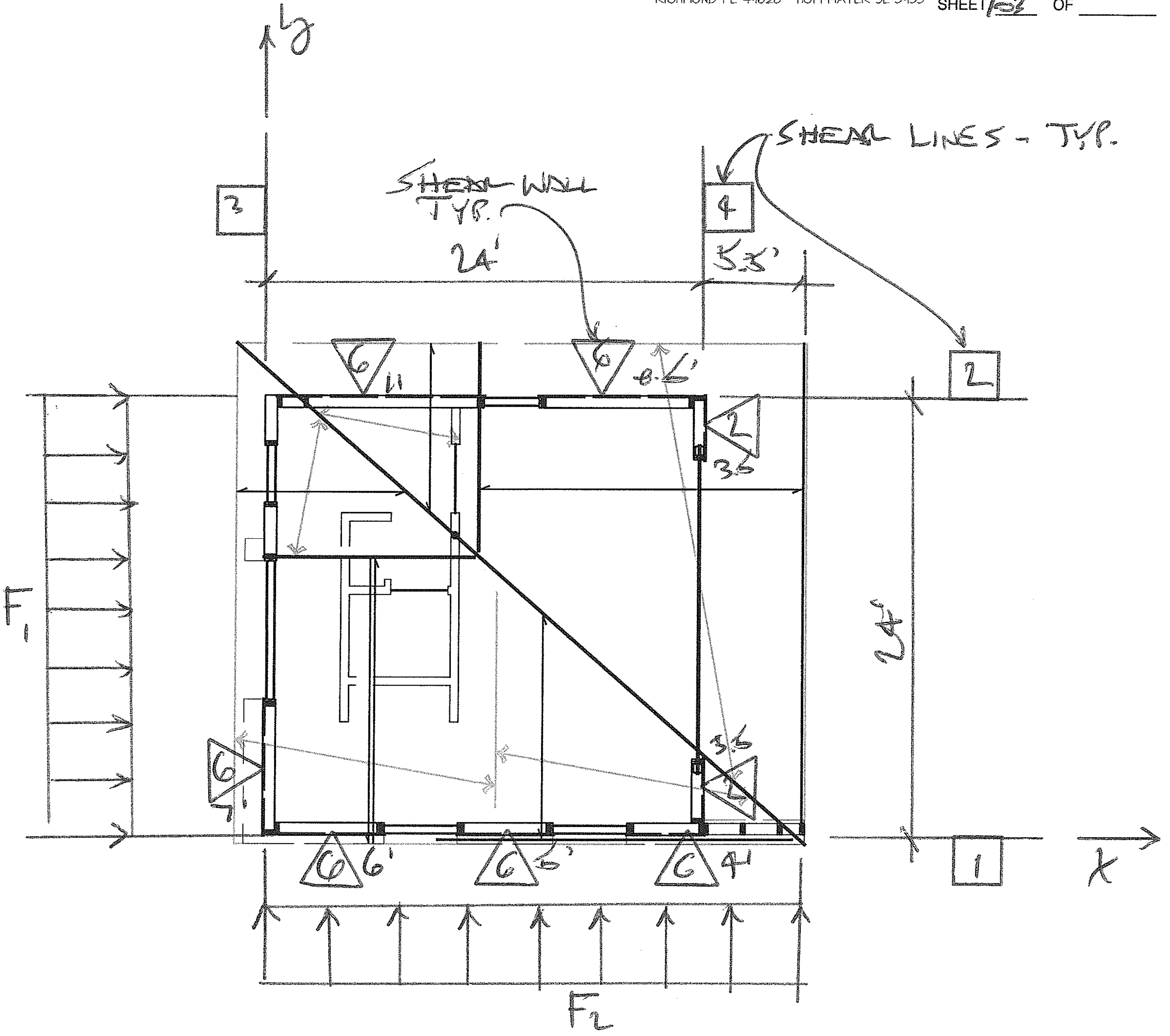
Table 8: Maximum Allowable Uniform Header Loads (plf)
(Panel Splice a minimum of 6 in. from edge of opening) 3-1/2 in. through 11-1/4 in. Core Thickness[1,2]

Header Depth[3] (in)	Deflection Limit[4]	Header Span (ft)			
		4	6	8	10
12	L/480	345	243	156	99
	L/360	450	295	190	125
	L/240	630	382	236	153
18	L/480	705	388	254	235
	L/360	750	482	302	281
	L/240	750	482	302	281
24	L/480	698	556	368	350
	L/360	896	556	368	350
	L/240	896	556	368	350

SIPs wall panel spline minimum 6" from edge of opening

[1] Vertical loads only. Lateral loads shall be transferred to the edges of the openings through continuous plate(s) designed in accordance with accepted engineering practice. Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.
[2] Tabulated values are based on the strong-axis of the facing material oriented perpendicular to the direction of header span.
[3] Minimum depth of facing above opening.
[4] Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code.

PROJECT FELDSPAR LOWER JOB 2017-0610
CLIENT VILLAGE NEST
ADDRESS EDEN, UTAH
DESIGN HUUM
ENGINEER J.H. DATE 6/16/2017
RICHMOND PE 49628 HOFFMAYER SE 3935 SHEET 102 OF


y
x
SHEAR LINES - TYP.
SHEAR WALL TYP.
3
4
2
1
24'
5.5'
24'
6
11
6
8.6'
2
3.5
3.5
2
6
7'
6
6'
6
5'
6
4'
F1
F2

2ND FLOOR SHEAR WALL

FELDSPAR LVN

N.T.S.

* SHEAR WALL NOTE -
COMPARE LENGTHS PER
LINE W/ FELDSPAR LVN -
USE MIN LENGTH / LINE

PROJECT FELDSPAR LOWER JOB 2017-0610
CLIENT VILLAGE NEST
ADDRESS EDEN, UTAH
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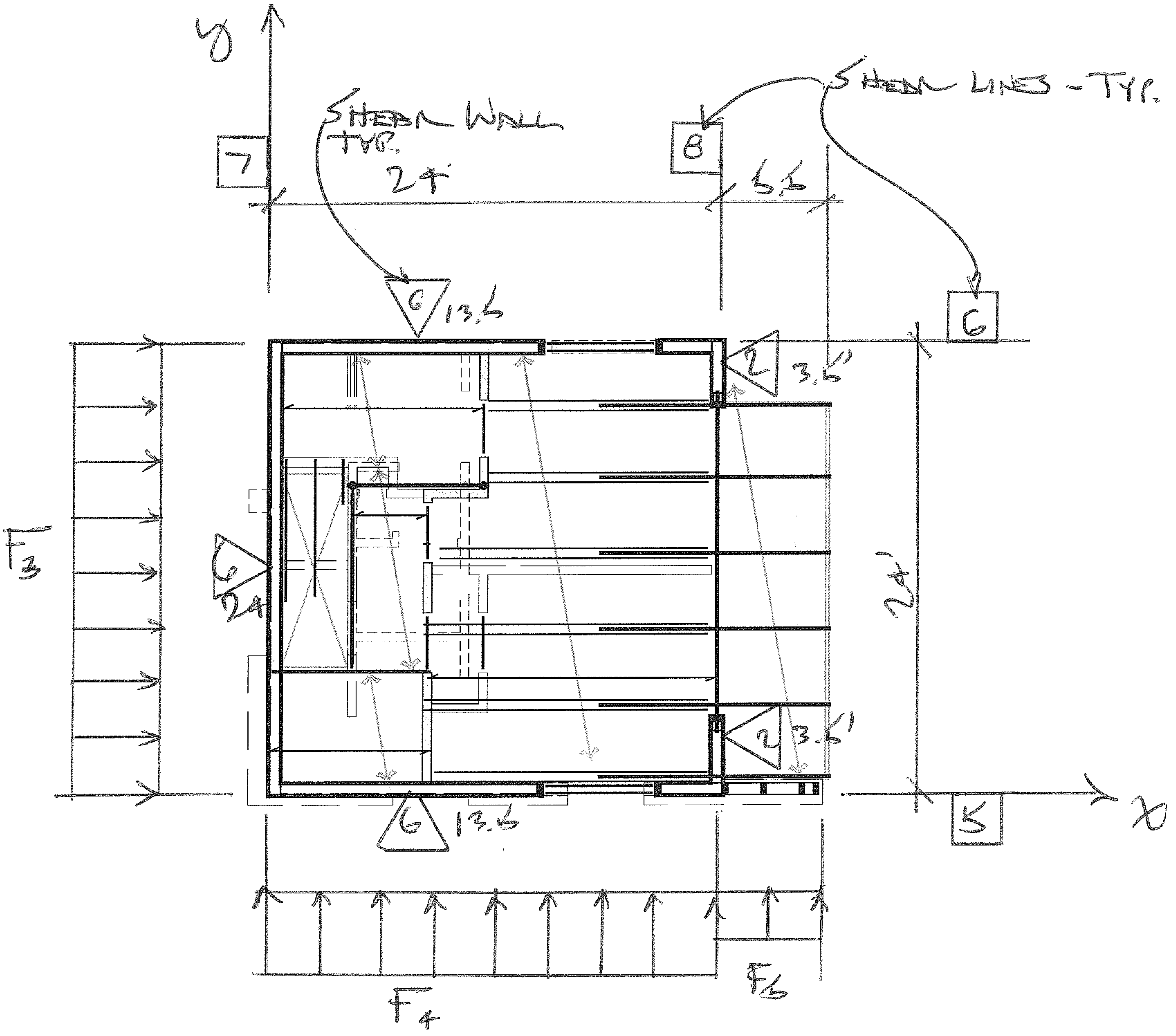
ENGINEER J.H. DATE 6/16/2017
RICHMOND PE 49628 HOFFMAYER SE 3935 SHEET 109 OF

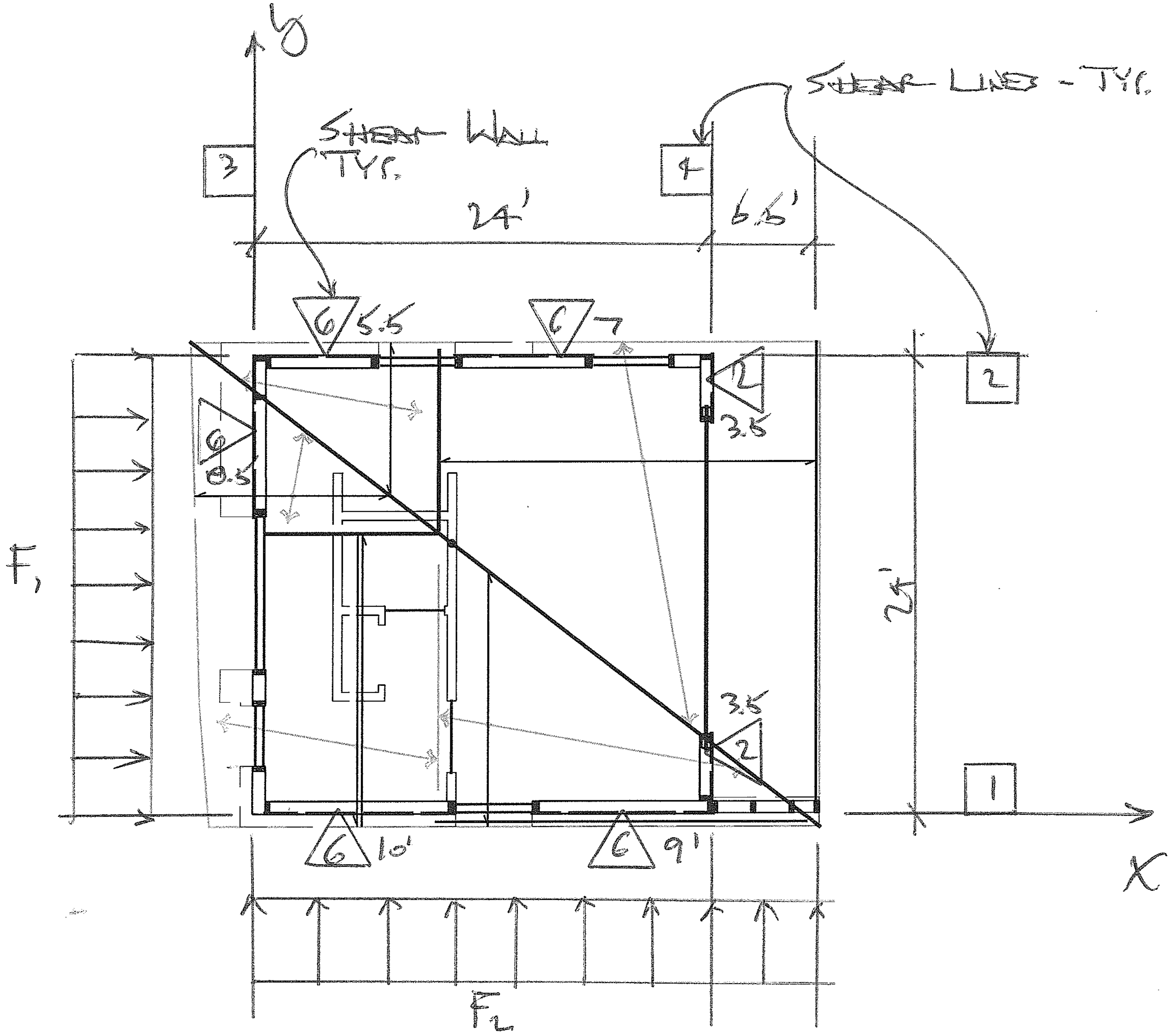

y
7
8
SHEAR WALL TYP.
SHEAR LINES - TYP.
24'
5.5
6
6 13.5
2 3.5'
F3
6 24
24'
2 3.5'
6 13.5
5
x
F4
F5

1st FLOOR SHEAR WALLS

FELDSPAR LVN

N.T.S.

PROJECT	FELDSPAR LOWER	JOB	2017-0610
CLIENT	VILLAGE NEST		
ADDRESS	EDEN, UTAH		
DESIGN	HUUM		
ENGINEER	J.H. RICHMOND PE 49628 HOFFMAYER SE 3935	DATE	6/16/2017
		SHEET	15 OF

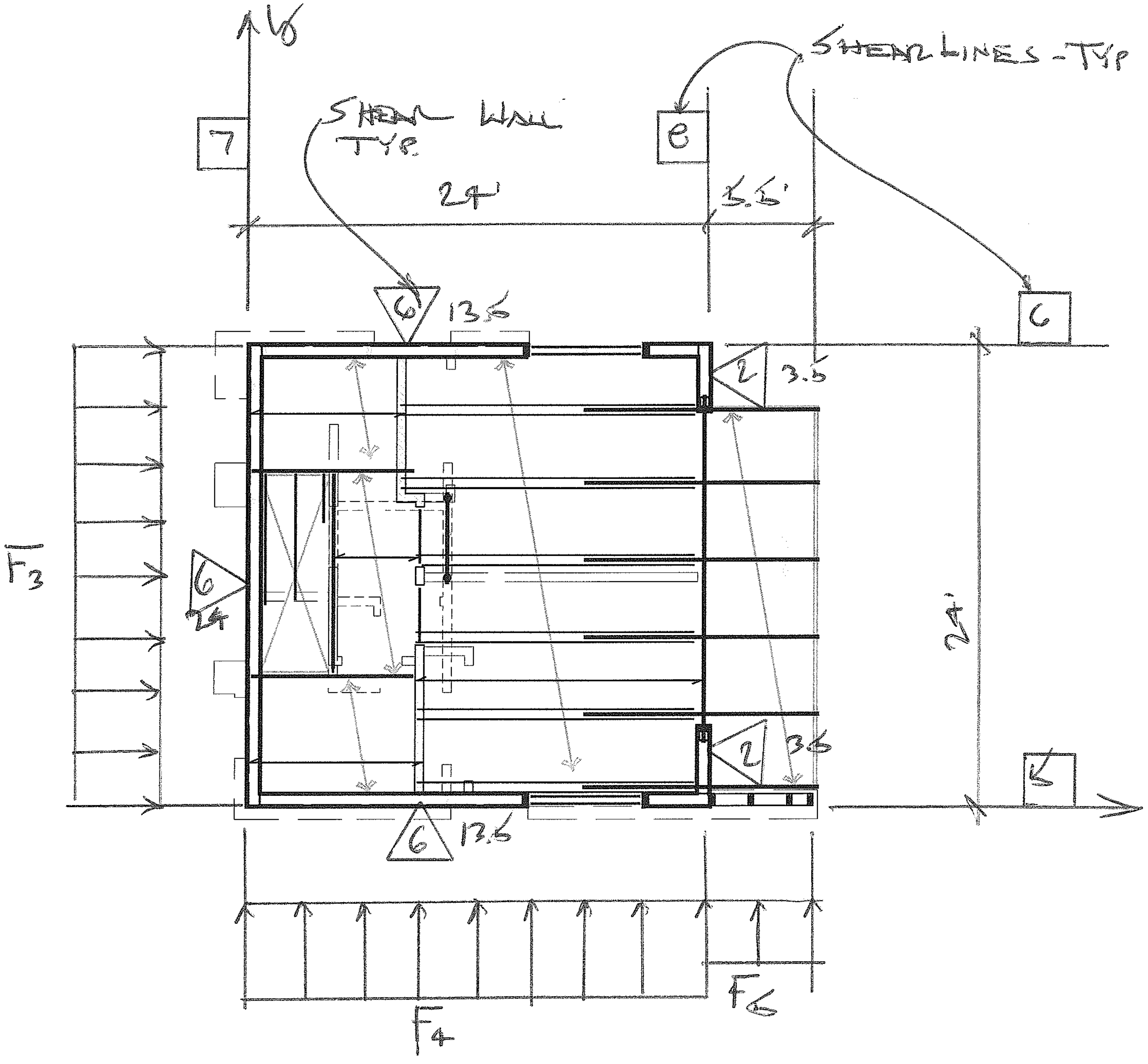

y
SHEAR WALL TYP.
SHEAR LINES - TYP.
3
4
24'
6.5'
6
5.5
6
7
2
3.5
6
8.5
F1
25'
2
3.5
2
1
6
10'
6
9'
X
F2

2ND FLOOR SHEAR WALLS

QUARTZ LVN
NTS

* SHEAR WALL NOTE -
COMPARE LENGTHS PER
LINE W/ FELDSPAR LVN -
USE MIN LENGTH / LINE

PROJECT	FELDSPAR LOWER	JOB	2017-0610
CLIENT	VILLAGE NEST		
ADDRESS	EDEN, UTAH		
DESIGN	HUUM		
ENGINEER	J.H.	DATE	6/16/2017
	RICHMOND PE 49628 HOFFMAYER SE 3935	SHEET	106 OF


y
Shear Lines - Typ
Shear Wall Typ
7
e
24'
3.5'
6
13.5
c
2
3.5
F3
6
24
24'
2
3.5
k
6
13.5
F4
F6

1st Floor Shear Walls

Quartz LVN

NTS.

USGS Design Maps Detailed Report

2012/2015 International Building Code (41.36°N, 111.74°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 1613.3.1 — Mapped acceleration parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2012/2015 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1)[1]	$S_S = 0.806$ g
From Figure 1613.3.1(2)[2]	$S_1 = 0.267$ g

Section 1613.3.2 — Site class definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Section 1613.

2010 ASCE-7 Standard – Table 20.3-1
SITE CLASS DEFINITIONS

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 1613.3.3 — Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_S = 0.806 g, F_a = 1.178

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1-s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.267 g, F_v = 1.866

Equation (16-37): $S_{MS} = F_aS_S = 1.178 \times 0.806 = 0.949$ g

Equation (16-38): $S_{M1} = F_vS_1 = 1.866 \times 0.267 = 0.498$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.949 = 0.633$ g

Equation (16-40): $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.498 = 0.332$ g

Section 1613.3.5 — Determination of seismic design category

TABLE 1613.3.5(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATION

VALUE OF S_{DS}	**RISK CATEGORY**		
	I or II	**III**	**IV**
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.633 g, Seismic Design Category = D

TABLE 1613.3.5(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	**RISK CATEGORY**		
	I or II	**III**	**IV**
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.332 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 1613.3.5(1) or 1613.3.5(2)" = D

Note: See Section 1613.3.5.1 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 1613.3.1(1)*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(1).pdf
2. *Figure 1613.3.1(2)*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(2).pdf

SEISMIC DESIGN:

ADDRESS		5754 DAYBREAK RIDGE, EDEN, UTAH					
		GOOGLE EARTH					
		LATITUDE	N	=	41.3600		
		LONGTITUDE	W	=	-111.7400		
		USGS NSHMP - 2012 IBC					
		BASED ON SITE CLASS B, Fa = 1.0 AND Fv = 1.0	Ss	=	0.806		
			S1	=	0.267		
2016CBC / ASCE 7-10							
1603.1.5		**EARTHQUAKE DESIGN DATA**					
	1	SEISMIC IMPORTANCE FACTOR AND OCCUPANCY	I	=	1.00		
		OCCUPANCY		=	II		*ASCE 7-10 TABLE 11.5-1*
	2	MAPPED SPECTRAL RESPONSE ACCELERATIONS	Ss	=	0.81		*USGS MAPS OF SPECTRAL RESPONSE*
			S1	=	0.27		
	3	SITE CLASS			D		IBC DEFAULT
		SITE COEFFICIENTS	1.00	=	1.18		*TABLE 11.4-1*
			Fv	=	1.87		*TABLE 11.4-2*
	4	SPECTRAL RESPONSE COEFFICIENTS	SMS	=	0.949		
			SM1	=	0.498		
			SDS	=	0.633	= 2/3 SMS	*ASCE 7-10 (11.4-3)*
			SD1	=	0.332	= 2/3 SM1	*ASCE 7-10 (11.4-4)*
	5	SEISMIC DESIGN CATEGORY			D		*IBC2012 TABLES 1613.5.6(1) & 1613.5.6(2)*
MAIN BUILDING DIRECTION - X							
	6	BASIC SEISMIC-FORCE-RESISTING SYSTEM					
		A. BEARING WALL SYSTEMS					
		A-LIGHT-FRAMED WALLS SHEATHED WITH WOOD PANELS					
			R	=	6.50		*ASCE 7-10 TABLE 12.2-1*
			Ωo	=	3.00		*ASCE 7-10 TABLE 12.2-1*
			Cd	=	4.00		*ASCE 7-10 TABLE 12.2-1*
	7	DESIGN BASE SHEAR	V	=	Cs W		
	8	SEISMIC RESPONSE COEFFICIENT	Cs	=	0.097	= SDS / (R / I)	*ASCE 7-10 (12.8-2)*
			T	=	Cu Ta		
			Cu	=	1.40		*ASCE 7-10 TABLE 12.8-1*
			hn	=	10.00	ft	
			Ct	=	0.02		*ASCE 7-10 TABLE 12.8-2*
			x	=	0.75		*ASCE 7-10 TABLE 12.8-2*
			Ta	=	0.112	= Ct hn^x	*ASCE 7-10 (12.8-7)*
			T	=	0.157	s	*ASCE 7-10 12.8.2*
			TL	=	8.00	s	*ASCE 7-10 FIG.22-16*
			Cs'	=	0.325	MAX	*ASCE 7-10 (12.8-3)*
			Cs"	=	0.010	MIN	*ASCE 7-10 (12.8-5)*
			Cs'"	=	0.010	MIN'	*ASCE 7-10 (12.8-6)*
			Cs USE	=	0.097		Cs OVERIDE 0.000
	9	RESPONSE MODIFICATION FACTOR	R	=	6.50		
	10	ANALYSIS PROCEDURE USED			EQUIVALENT LATERAL FORCE PROCEDURE		
			V	=	0.0973	W	*X-DIRECTION BASE SHEAR FORCE*
MAIN BUILDING DIRECTION - Y							
	6	BASIC SEISMIC-FORCE-RESISTING SYSTEM					
		A. BEARING WALL SYSTEMS					
		A-LIGHT-FRAMED WALLS SHEATHED WITH WOOD PANELS					
			R	=	6.50		*ASCE 7-10 TABLE 12.2-1*
			Ωo	=	3.00		*ASCE 7-10 TABLE 12.2-1*
			Cd	=	4.00		*ASCE 7-10 TABLE 12.2-1*
	7	DESIGN BASE SHEAR	V	=	Cs W		
	8	SEISMIC RESPONSE COEFFICIENT	Cs	=	0.097	= SDS / (R / I)	*ASCE 7-10 (12.8-2)*
			T	=	Cu Ta		
			Cu	=	1.40		*ASCE 7-10 TABLE 12.8-1*
			hn	=	10.00	ft	
			Ct	=	0.02		*ASCE 7-10 TABLE 12.8-2*
			x	=	0.75		*ASCE 7-10 TABLE 12.8-2*
			Ta	=	0.112	= Ct hn^x	*ASCE 7-10 (12.8-7)*
			T	=	0.157	s	*ASCE 7-10 12.8.2*
			TL	=	8.00	s	*ASCE 7-10 FIG.22-16*
			Cs'	=	0.325	MAX	*ASCE 7-10 (12.8-3)*
			Cs"	=	0.010	MIN	*ASCE 7-10 (12.8-5)*
			Cs'"	=	0.010	MIN'	*ASCE 7-10 (12.8-6)*
			Cs USE	=	0.097		Cs OVERIDE 0.000
	9	RESPONSE MODIFICATION FACTOR	R	=	6.50		
	10	ANALYSIS PROCEDURE USED			EQUIVALENT LATERAL FORCE PROCEDURE		
			V	=	0.0973	W	*Y-DIRECTION BASE SHEAR FORCE*

DATE 6/27/2017
SHEET ______

AREA WEIGHTS - 2ND LEVEL:

ROOF

9.00 ft WALL HT.

WEIGHT TABULATIONS:

2X = 62496.90 lb

2Y = 58787.95 lb

WEIGHT TYPES:

A	METAL ROOF	15.76 psf	U	EXTERIOR	15.23 psf
B	OPENED BALCONY	7.00 psf	V	INTERIOR	8.85 psf
C	2nd FLOOR	14.25 psf	W	-	0.00 psf
D	DRIVEWAY LOADS	85.38 psf	X	-	0.00 psf
E	20% SNOW LOAD FOR SEISM	52.80 psf	Y	-	0.00 psf
F	GARAGE LOADS	63.50 psf	Z	-	0.00 psf

W#	F#		UNIT WEIGHT	TYPE	LENGTH	UNIT WEIGHT	TYPE	HEIGHT	NUMBER	UNIT WEIGHT	TYPE	HEIGHT	NUMBER	TRIB. WEIGHT	WIDTH	TOTAL WEIGHT
2-1	1	2X	15.76 psf	A	x 33.00 ft	+ 15.23 psf	U	x 5.00 ft	x 2.00	+ 8.85 psf	V	x 5.00 ft	x 1.00	= 716.44 plf	x 24.00 ft =	17194.50 lb
		+	52.80 psf	E	x 33.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 1742.40 plf	x 26.00 ft =	45302.40 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
																62496.90 lb
2-2	2	2Y	15.76 psf	A	x 26.00 ft	+ 15.23 psf	U	x 5.00 ft	x 2.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 561.90 plf	x 24.00 ft =	13485.55 lb
		+	52.80 psf	E	x 26.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 1372.80 plf	x 33.00 ft =	45302.40 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
																58787.95 lb
2--	-	-	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
																0.00 lb
2--	-	-	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
																0.00 lb
2--	-	-	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
																0.00 lb
2--	-	-	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
																0.00 lb

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DATE 6/27/2017
SHEET ______

AREA WEIGHTS - 1ST LEVEL:

2ND FLOOR

9.00 ft WALL HT.

WEIGHT TABULATIONS:

1X = 18564.00 lb

1Y = 18564.00 lb

WEIGHT TYPES:

A	METAL ROOF	15.76 psf	U	EXTERIOR	15.23 psf
B	OPENED BALCONY	7.00 psf	V	INTERIOR	8.85 psf
C	2nd FLOOR	14.25 psf	W	-	0.00 psf
D	DRIVEWAY LOADS	85.38 psf	X	-	0.00 psf
E	20% SNOW LOAD FOR SEISM	52.80 psf	Y	-	0.00 psf
F	GARAGE LOADS	63.50 psf	Z	-	0.00 psf

W#	F#		UNIT WEIGHT	TYPE	LENGTH	UNIT WEIGHT	TYPE	HEIGHT	NUMBER	UNIT WEIGHT	TYPE	HEIGHT	NUMBER	TRIB. WEIGHT	WIDTH	TOTAL WEIGHT
1-3	3	1X	14.25 psf	C	x 24.00 ft	+ 15.23 psf	U	x 10.00 ft	x 2.00	+ 8.85 psf	V	x 10.00 ft	x 1.00	= 735.00 plf	x 24.00 ft =	17640.00 lb
		+	7.00 psf	B	x 5.50 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 38.50 plf	x 24.00 ft =	924.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
																18564.00 lb
1-4	4	1Y	14.25 psf	C	x 24.00 ft	+ 15.23 psf	U	x 10.00 ft	x 2.00	+ 8.85 psf	V	x 10.00 ft	x 1.00	= 735.00 plf	x 24.00 ft =	17640.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
																17640.00 lb
1-5	5	1Y	7.00 psf	B	x 24.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 168.00 plf	x 5.50 ft =	924.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
																924.00 lb
1--	-	-	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
																0.00 lb
1--	-	-	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
		+	0.00 psf	-	x 0.00 ft	+ 0.00 psf	-	x 0.00 ft	x 0.00	+ 0.00 psf	-	x 0.00 ft	x 0.00	= 0.00 plf	x 0.00 ft =	0.00 lb
																0.00 lb

EQUIVALENT LATERAL FORCE PROCEDURE:

BASE SHEAR	V	=	Cs W	*ASCE 7-10 (12.8-1)*	
TOTAL BUILDING DEAD WEIGHT - X DIRECTION	Wx	=	81060.90 lb		
X-DIRECTION SEISMIC RESPONSE COEFICIENT	Csx	=	0.0973	SEE PREVIOUS PAGES ... Rx =	6.50
STRENGTH LEVEL BASE SHEAR FORCE - X	Vx	=	7891.14 lb		
TOTAL BUILDING DEAD WEIGHT - Y DIRECTION	Wy	=	77351.95 lb		
Y-DIRECTION SEISMIC RESPONSE COEFICIENT	Csy	=	0.0973	SEE PREVIOUS PAGES ... Ry =	6.50
STRENGTH LEVEL BASE SHEAR FORCE - Y	Vy	=	7530.08 lb		

ASCE 7-10
12.8.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES
PER EACH ORTHOGONAL DIRECTION x & y

$$F = C_v V \qquad \textit{ASCE 7-10 (12.8-11)}$$

$$C_v = \frac{w\,h^{\wedge}k}{\Sigma\, w_i\, h_i^{\wedge}k} \qquad \textit{ASCE 7-10 (12.8-12)}$$

X-DIRECTION BUILDING PERIOD T = 0.2648 s SEE PREVIOIUS PAGES
k = 1.00 *ASCE 7-10 12.8.3*

LEVEL	wx	hx	hx ^ k	wx hx ^ k	Cvx
-	0.00	0.00	0.000	0.00	0.000
-	0.00	0.00	0.000	0.00	0.000
-	0.00	0.00	0.000	0.00	0.000
-	0.00	0.00	0.000	0.00	0.000
2	62496.90	20.00	20.000	1249938.02	0.871
1	18564.00	10.00	10.000	185640.00	0.129
SUM	81060.90			1435578.02	1.000

STRENGTH SHEAR FORCE PER LEVEL Fx	Wx Csx	DISTRIBUTION FACTOR Fx / (Wx Csx)
0.00	0.00	0.000
0.00	0.00	0.000
0.00	0.00	0.000
0.00	0.00	0.000
6870.71	6083.96	1.129
1020.43	1807.17	0.565
7891.14		

Y-DIRECTION BUILDING PERIOD T = 0.2648 s SEE PREVIOIUS PAGES
k = 1.00 *ASCE 7-10 12.8.3*

LEVEL	wy	hy	hxy ^ k	wy hy ^ k	Cvy
-	0.00	0.00	0.000	0.00	0.000
-	0.00	0.00	0.000	0.00	0.000
-	0.00	0.00	0.000	0.00	0.000
-	0.00	0.00	0.000	0.00	0.000
2	58787.95	20.00	20.000	1175758.92	0.864
1	18564.00	10.00	10.000	185640.00	0.136
SUM	77351.95			1361398.92	1.000

STRENGTH SHEAR FORCE PER LEVEL Fy	Wy Csy	DISTRIBUTION FACTOR Fy / (Wy Csy)
0.00	0.00	0.000
0.00	0.00	0.000
0.00	0.00	0.000
0.00	0.00	0.000
6503.28	5722.90	1.136
1026.80	1807.17	0.568
7530.08		

116 DATE 6/27/2017
SHEET ______

SEISMIC FORCES:

4TH LEVEL FORCES -

F#		TOTAL TRIB. WEIGHT		DIST. FACTOR		Cs		DESIGN SHEAR FORCE (STRENGTH LEVEL)		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00

DISTRIBUTION OVERIDE	R OVERIDE

3RD LEVEL FORCES -

F#		TOTAL TRIB. WEIGHT		DIST. FACTOR		Cs		DESIGN SHEAR FORCE (STRENGTH LEVEL)		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00

DISTRIBUTION OVERIDE	R OVERIDE

2ND LEVEL FORCES ROOF

F#		TOTAL TRIB. WEIGHT		DIST. FACTOR		Cs		DESIGN SHEAR FORCE (STRENGTH LEVEL)		
1	2X	2403.73 plf	x	1.13	x	0.0973	=	264.26 plf	R =	6.50
2	2Y	1781.45 plf	x	1.14	x	0.0973	=	197.07 plf	R =	6.50
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00

DISTRIBUTION OVERIDE	R OVERIDE

1ST LEVEL FORCES 2ND FLOOR

F#		TOTAL TRIB. WEIGHT		DIST. FACTOR		Cs		DESIGN SHEAR FORCE (STRENGTH LEVEL)		
3	1X	773.50 plf	x	0.56	x	0.0973	=	42.52 plf	R =	6.50
4	1Y	735.00 plf	x	0.57	x	0.0973	=	40.65 plf	R =	6.50
5	1Y	168.00 plf	x	0.57	x	0.0973	=	9.29 plf	R =	6.50
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00

DISTRIBUTION OVERIDE	R OVERIDE

SEISMIC FORCES TO SHEAR LINES:

2ND LEVEL ROOF

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
1	F1	-	(	264.26 plf	) x	24.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	3171.09 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(-	) x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
												6.50								3171.09 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
2	F1	-	(	264.26 plf	) x	24.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	3171.09 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(-	) x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
												6.50								3171.09 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
3	F2	-	(	197.07 plf	) x	24.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	2364.83 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(-	) x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
												6.50								2364.83 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
4	F2	-	(	197.07 plf	) x	24.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	2364.83 lb
+	F2	-	(	197.07 plf	) x	5.50	x	26.75	/	24.00	x	6.50	/	6.50	x	1.00	/	1.00	=	1208.07 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(-	) x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
												6.50								3572.90 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
-	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(-	) x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
												100.00								0.00 lb

SEISMIC FORCES TO SHEAR LINES:

1ST LEVEL 2ND FLOOR

SHEAR LINE FORCES

LINE	F#			FORCE			WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
5	F3	-	(	42.52 plf		)x	24.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	510.22 lb
+	-	LINE1	(		3171.09 lb	)x	1.00	x	1.00	/	1.00	x	6.50	/	6.50	x	1.00	/	1.00	=	3171.09 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(	-	)x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
													6.50								3681.31 lb

SHEAR LINE FORCES

LINE	F#			FORCE			WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
6	F3	-	(	42.52 plf		)x	24.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	510.22 lb
+	-	LINE2	(		3171.09 lb	)x	1.00	x	1.00	/	1.00	x	6.50	/	6.50	x	1.00	/	1.00	=	3171.09 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(	-	)x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
													6.50								3681.31 lb

SHEAR LINE FORCES

LINE	F#			FORCE			WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
7	F4	-	(	40.65 plf		)x	24.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	487.85 lb
+	-	LINE3	(		2364.83 lb	)x	1.00	x	1.00	/	1.00	x	6.50	/	6.50	x	1.00	/	1.00	=	2364.83 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(	-	)x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
													6.50								2852.67 lb

SHEAR LINE FORCES

LINE	F#			FORCE			WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
8	F4	-	(	40.65 plf		)x	24.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	487.85 lb
+	F5	-	(	9.29 plf		)x	5.50	x	26.75	/	24.00	x	6.50	/	6.50	x	1.00	/	1.00	=	56.96 lb
+	-	LINE4	(		3572.90 lb	)x	1.00	x	1.00	/	1.00	x	6.50	/	6.50	x	1.00	/	1.00	=	3572.90 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(	-	)x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
													6.50								4117.71 lb

SHEAR LINE FORCES

LINE	F#			FORCE			WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
-	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(	-	)x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
													100.00								0.00 lb

REDUNDANCY CHECK:

IN LIEU OF CHECKING REDUNDANCY ON EACH SHEAR LINE, USE DEFAULT VALUE OF rho = 1.3

STRENGTH DESIGN SHEAR FORCE SUMMARY:

	LINE	FORCE		rho		V SEISMIC (STRENGTH)		ϕ		V SEISMIC (ASD)
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
ROOF	1	3171.09 lb	x	1.30	=	4122.42 lb	x	0.70	=	2885.70 lb
	2	3171.09 lb	x	1.30	=	4122.42 lb	x	0.70	=	2885.70 lb
	3	2364.83 lb	x	1.30	=	3074.28 lb	x	0.70	=	2151.99 lb
	4	3572.90 lb	x	1.30	=	4644.77 lb	x	0.70	=	3251.34 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
2ND FLOOR	5	3681.31 lb	x	1.30	=	4785.70 lb	x	0.70	=	3349.99 lb
	6	3681.31 lb	x	1.30	=	4785.70 lb	x	0.70	=	3349.99 lb
	7	2852.67 lb	x	1.30	=	3708.48 lb	x	0.70	=	2595.93 lb
	8	4117.71 lb	x	1.30	=	5353.03 lb	x	0.70	=	3747.12 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb

WIND LOAD DESIGN:

2016CBC / ASCE 7-10								
1609.6.4	**DIRECTIONAL PROCEDURE - MWFRS**							
	ENCLOSED BUILDING						*ASCE 7-10 27.4.1*	
26.5-1A	**BASIC WIND SPEEDS FOR RISK CATEGORY II BUILDINGS**							
1	BASIC WIND SPEED	V	=	115.00 MPH			*ASCE 7-10 FIG 26.5-1*	
	WIND DIRECTIONALLY FACTOR	Kd	=	0.85			*ASCE 7-10 TABLE 26.6-1*	
2	IMPORTANCE FACTOR, CATEGORY II	I	=	1.00			*ASCE 7-10 TABLE 6-1*	

BUILDING DIRECTION - X

3	EXPOSURE CATEGORY			C			*ASCE 7-10 26.7.3*	
				CASE 1			*ASCE 7-10 TABLE 6-3*	
	HEIGHT ABOVE GROUND LEVEL	z	=	20.00 ft			MEAN ROOF HEIGHT	
	TERRAIN EXPOSURE CONSTRANT	α	=	9.50			*ASCE 7-10 TABLE 26.9-1*	
	TERRAIN EXPOSURE CONSTRANT	zg	=	900.00 ft			*ASCE 7-10 TABLE 26.9-1*	
	VELOCITY PRESSURE	Kz	=	0.90			*ASCE 7-10 TABLE 27.3-1*	
4	TOPOGRAPHY FACTOR	Kzt	=	1.000			*ASCE 7-10 26.8.2*	
5	VELOCITY PRESSURE	qz	=	.00256 Kz Kzt Kd V^2 I			*ASCE 7-10 (27.3-1)*	
		qz	=	25.954 psf			PRESSURE AT MEAN ROOF HEIGHT	
6	GUST FACTOR	G	=	0.85			*ASCE 7-10 26.9.1*	
7	ENCLOSURE CLASSIFICATION			ENCLOSED			*ASCE 7-10 26.10*	
8	INTERNAL PRESSURE	GCpi	=	0.18 + / -			*ASCE 7-10 TABLE 26.11-1*	
9	EXTERNAL PRESSURE COEFFICIENTS	L	=	24.00			*ASCE 7-10 FIG 27.4.1*	
		B	=	24.00			*ASCE 7-10 FIG 27.4.1*	
		L/B	=	1.000			*ASCE 7-10 FIG 27.4.1*	
		h/L	=	0.833			*ASCE 7-10 FIG 27.4.1*	
		Cp 1	=	0.800	WINDWARD WALL		*ASCE 7-10 FIG 27.4.1*	
		Cp 2	=	-0.500	LEEWARD WALL		*ASCE 7-10 FIG 27.4.1*	
		Cp 3	=	-0.700	SIDE WALLS		*ASCE 7-10 FIG 27.4.1*	
	PARALLEL TO RIDGE	Cp 4	=	-0.900	ROOF 1	0.00 deg	*ASCE 7-10 FIG 27.4.1*	PITCH 0.00 :12
		Cp 5	=	-0.900	ROOF 2	0.00 deg		PITCH 0.00 :12
10	DESIGN WIND LOAD	p	=	qGCp - qi (Gcpi)			*ASCE 7-10 TABLE 26.11-1*	
	WINDWARD q = qz & qi = qh	p	=	17.649 -	-4.672 =	22.321 psf		LATERAL FORCE
	LEEWARD q = qh	p	=	-11.030 -	4.672 =	-15.702 psf		P WIND 38.02 psf
	SIDE WALLS qi = qh	p	=	-15.443 -	4.672 =	-20.114 psf		
	ROOF 1 qi = qh	p	=	-19.855 +	-4.672 =	-24.527 psf		WIND -24.53 psf
	ROOF 2 qi = qh	p	=	-19.855 +	-4.672 =	-24.527 psf		LEE -24.53 psf
	DESIGN WIND LOAD x DIRECTION	p WALL	=	22.32 -	-15.70 =	38.02 psf		UPLIFT FORCE
		p ROOF		0.00 -	0.00 =	0.00 psf		

BUILDING DIRECTION - Y

3	EXPOSURE CATEGORY			C			*ASCE 7-10 26.7.3*	
				CASE 1			*ASCE 7-10 TABLE 6-3*	
	HEIGHT ABOVE GROUND LEVEL	z	=	20.00 ft			MEAN ROOF HEIGHT	
	TERRAIN EXPOSURE CONSTRANT	α	=	9.50			*ASCE 7-10 TABLE 26.9-1*	
	TERRAIN EXPOSURE CONSTRANT	zg	=	900.00 ft			*ASCE 7-10 TABLE 26.9-1*	
	VELOCITY PRESSURE	Kz	=	0.90			*ASCE 7-10 TABLE 27.3-1*	
4	TOPOGRAPHY FACTOR	Kzt	=	1.000			*ASCE 7-10 26.8.2*	
5	VELOCITY PRESSURE	qz	=	.00256 Kz Kzt Kd V^2 I			*ASCE 7-10 (27.3-1)*	
		qz	=	25.954 psf			PRESSURE AT MEAN ROOF HEIGHT	
6	GUST FACTOR	G	=	0.85			*ASCE 7-10 26.9.1*	
7	ENCLOSURE CLASSIFICATION			ENCLOSED			*ASCE 7-10 26.10*	
8	INTERNAL PRESSURE	GCpi	=	0.18 + / -			*ASCE 7-10 TABLE 26.11-1*	
9	EXTERNAL PRESSURE COEFFICIENTS	L	=	24.00			*ASCE 7-10 FIG 27.4.1*	
		B	=	24.00			*ASCE 7-10 FIG 27.4.1*	
		L/B	=	1.000			*ASCE 7-10 FIG 27.4.1*	
		h/L	=	0.833			*ASCE 7-10 FIG 27.4.1*	
		Cp 1	=	0.800	WINDWARD WALL		*ASCE 7-10 FIG 27.4.1*	
		Cp 2	=	-0.500	LEEWARD WALL		*ASCE 7-10 FIG 27.4.1*	
		Cp 3	=	-0.700	SIDE WALLS		*ASCE 7-10 FIG 27.4.1*	
	NORMAL TO RIDGE (> 10 deg)	Cp 4	=	-0.900	ROOF 1	0.00 deg	*ASCE 7-10 FIG 27.4.1*	PITCH 0.00 :12
		Cp 4	=	-0.900	ROOF 2	0.00 deg		PITCH 0.00 :12
10	DESIGN WIND LOAD	p	=	qGCp - qi (Gcpi)			*ASCE 7-10 TABLE 26.11-1*	
	WINDWARD q = qz & qi = qh	p	=	17.649 -	-4.672 =	22.321 psf		LATERAL FORCE
	LEEWARD q = qh	p	=	-11.030 -	4.672 =	-15.702 psf		P WIND 38.02 psf
	SIDE WALLS qi = qh	p	=	-15.443 -	4.672 =	-20.114 psf		
	ROOF 1 qi = qh	p	=	-19.855 +	-4.672 =	-24.527 psf		WIND -24.53 psf
	ROOF 2 qi = qh	p	=	-19.855 +	-4.672 =	-24.527 psf		LEE -24.53 psf
	DESIGN WIND LOAD y DIRECTION	P WALL	=	22.32 -	-15.70 =	38.02 psf		UPLIFT FORCE
		P ROOF		0.00 -	0.00 =	0.00 psf		

WIND FORCES:

	P#		WIND FORCE WALL	TRIB. HEIGHT	WIND FORCE ROOF		TRIB. HT.	LEE FORCE ROOF		TRIB. HT.	V WIND (STRENGTH)
- = 0.00ft	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
- = 0.00ft	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
ROOF = 10.00ft	1	2X	38.02 psf x	5.00 + (	-24.53 psf x	SIN 0.00 x	0.00 ft -	-24.53 psf x	SIN 0.00 x	0.00 ft) =	190.11 plf
	2	2Y	38.02 psf x	5.00 + (	-24.53 psf x	SIN 0.00 x	0.00 ft -	-24.53 psf x	SIN 0.00 x	0.00 ft) =	190.11 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
2ND FLOOR = 10.00ft	3	1X	38.02 psf x	10.00 + (	-24.53 psf x	SIN 0.00 x	0.00 ft -	-24.53 psf x	SIN 0.00 x	0.00 ft) =	380.23 plf
	4	1Y	38.02 psf x	10.00 + (	-24.53 psf x	SIN 0.00 x	0.00 ft -	-24.53 psf x	SIN 0.00 x	0.00 ft) =	380.23 plf
	5	1Y	38.02 psf x	10.00 + (	-24.53 psf x	SIN 0.00 x	0.00 ft -	-24.53 psf x	SIN 0.00 x	0.00 ft) =	380.23 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	7.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	7.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf

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WIND FORCES TO SHEAR LINES:

2ND LEVEL ROOF

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		FACTORS				% LOAD EFFECT				
1	F1	-	(	190.11 plf	) x	24.00	x	1.00	/	2.00	x	1.00	/	1.00	x	1.00	/	1.00	=	2281.36 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-			(	) x			0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
																				2281.36 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		FACTORS				% LOAD EFFECT				
2	F1	-	(	190.11 plf	) x	24.00	x	1.00	/	2.00	x	1.00	/	1.00	x	1.00	/	1.00	=	2281.36 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-			(	) x			0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
																				2281.36 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		FACTORS				% LOAD EFFECT				
3	F2	-	(	190.11 plf	) x	24.00	x	1.00	/	2.00	x	1.00	/	1.00	x	1.00	/	1.00	=	2281.36 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-			(	) x			0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
																				2281.36 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		FACTORS				% LOAD EFFECT				
4	F2	-	(	190.11 plf	) x	24.00	x	1.00	/	2.00	x	1.00	/	1.00	x	1.00	/	1.00	=	2281.36 lb
+	F2	-	(	190.11 plf	) x	5.50	x	26.75	/	24.00	x	1.00	/	1.00	x	1.00	/	1.00	=	1165.44 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-			(	) x			0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
																				3446.80 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		FACTORS				% LOAD EFFECT				
-	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-			(	) x			0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
																				0.00 lb

WIND FORCES TO SHEAR LINES:

1ST LEVEL 2ND FLOOR

SHEAR LINE FORCES

LINE	F#		(	FORCE		) x	WIDTH	x	X BAR	/	L	x	FACTORS	/		x	% LOAD EFFECT	/		=	
5	F3	-	(	380.23 plf		) x	24.00	x	1.00	/	2.00	x	1.00	/	1.00	x	1.00	/	1.00	=	4562.73 lb
+	-	F1	(		2281.36 lb	) x	1.00	x	1.00	/	1.00	x	1.00	/	1.00	x	1.00	/	1.00	=	2281.36 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-			(		) x			0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
																					6844.09 lb

SHEAR LINE FORCES

LINE	F#		(	FORCE		) x	WIDTH	x	X BAR	/	L	x	FACTORS	/		x	% LOAD EFFECT	/		=	
6	F3	-	(	380.23 plf		) x	24.00	x	1.00	/	2.00	x	1.00	/	1.00	x	1.00	/	1.00	=	4562.73 lb
+	-	F2	(		2281.36 lb	) x	1.00	x	1.00	/	1.00	x	1.00	/	1.00	x	1.00	/	1.00	=	2281.36 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-			(		) x			0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
																					6844.09 lb

SHEAR LINE FORCES

LINE	F#		(	FORCE		) x	WIDTH	x	X BAR	/	L	x	FACTORS	/		x	% LOAD EFFECT	/		=	
7	F4	-	(	380.23 plf		) x	24.00	x	1.00	/	2.00	x	1.00	/	1.00	x	1.00	/	1.00	=	4562.73 lb
+	-	F3	(		2281.36 lb	) x	1.00	x	1.00	/	1.00	x	1.00	/	1.00	x	1.00	/	1.00	=	2281.36 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-			(		) x			0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
																					6844.09 lb

SHEAR LINE FORCES

LINE	F#		(	FORCE		) x	WIDTH	x	X BAR	/	L	x	FACTORS	/		x	% LOAD EFFECT	/		=	
8	F4	-	(	380.23 plf		) x	24.00	x	1.00	/	2.00	x	1.00	/	1.00	x	1.00	/	1.00	=	4562.73 lb
+	F5	-	(	380.23 plf		) x	5.50	x	26.75	/	24.00	x	1.00	/	1.00	x	1.00	/	1.00	=	2330.87 lb
+	-	F4	(		3446.80 lb	) x	1.00	x	1.00	/	1.00	x	1.00	/	1.00	x	1.00	/	1.00	=	3446.80 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-			(		) x			0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
																					10340.41 lb

SHEAR LINE FORCES

LINE	F#		(	FORCE		) x	WIDTH	x	X BAR	/	L	x	FACTORS	/		x	% LOAD EFFECT	/		=	
-	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(			) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-			(		) x			0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
																					0.00 lb

WIND FORCE SUMMARY:

ALLOWABLE STRESS DESIGN SHEAR FORCE SUMMARY:

	LINE		V WIND (STRENGTH)		φ		V WIND (ASD)
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
ROOF	1	=	2281.36 lb	x	0.60	=	1368.82 lb
	2	=	2281.36 lb	x	0.60	=	1368.82 lb
	3	=	2281.36 lb	x	0.60	=	1368.82 lb
	4	=	3446.80 lb	x	0.60	=	2068.08 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
2ND FLOOR	5	=	6844.09 lb	x	0.60	=	4106.46 lb
	6	=	6844.09 lb	x	0.60	=	4106.46 lb
	7	=	6844.09 lb	x	0.60	=	4106.46 lb
	8	=	10340.41 lb	x	0.60	=	6204.24 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb

Table 10: Allowable In-Plane Shear Strength (Pounds per Foot) for SIP Shear Walls (Seismic Loads in Seismic Design Categories A, B, C, D, E and F)[1,2]

Spline Type[3]	Framing Minimum SG[4]	Minimum Facing Connections[2] Chord[2]	Plate[2]	Spline[3]	Shear Strengt h[5] (plf)	
Block, Surface, or Lumber Spline (Type S, Type L)	0.50	0.113"x 2-1/4" nails, 6" on center	0.113"x 2-1/4" nails, 3" on center	(7/16" thick, 3" wide spline) 0.113"x 2-1/4" nails 6" on center	360	△6
	0.50	0.113"x 2-1/4" nails, 6" on center	0.113"x 2-1/4" nails, 6" on center	(3/4" thick, 3" wide spline) 0.113"x 2-1/4" nails, 6" on center	360	
	0.50	0.113" x 2-3/8" nails, 3" on center Staggered (3/8" edge distance and 3/4" edge distance)	0.113" x 2-3/8" round head nails, 3" on center Staggered (3/8", 3/4" edge distance)	(23/32" thick, 3" wide spline) 0.113" x 2-3/8" nails 3" on center Staggered (3/8" edge distance and 3/4" edge distance)	720	△3
	0.50	0.113" x 2-3/8" nails, 2" on center Staggered (3/8" edge distance and 3/4" edge distance)	0.113" x 2-3/8" round head nails, 2" on center Staggered (3/8", 3/4" edge distance)	(23/32" thick, 3" wide spline) 0.113" x 2-3/8" nails, 2" on center Staggered (3/8" edge distance and 3/4" edge distance)	920	△2

[1] Shear strength values, as published in this table, are limited to assemblies resisting wind or seismic forces where the aspect ratio (height:width) does not exceed 1:1 for Type S panel connections or 2:1 for Type L panel connections. (IM 014 ACU17)
[2] Chords, hold-downs and connections to other structural elements must be designed by a registered design professional in accordance with accepted engineering practice.
[3] Spline type at interior panel-to-panel joints only, solid chord members are required at each end of each shear wall segment.
[4] Required connections must be made on each side of the panel. Dimensional or engineered lumber shall have an equivalent specific gravity not less than specified.
[5] For design to resist seismic forces, shear wall height-width ratios greater than 2:1, but not exceeding 3.5:1, are permitted for assemblies using lumber splines provided the allowable shear strength values in this table are multiplied by 2w/h.

USE FOR DESIGN NEXT PAGES

SIP 6 = △6
SIP 3 = △3
SIP 2 = △2

SEISMIC STRENGTH SHEAR FORCES TO SHEAR LINES AND ALLOWABLE STRESS SHEAR WALL FORCES:

ADJUST H/W < 3.5/1 USING 2w/h FOR CAPACITIES

LINE	V STRENGTH	A	B	C	D	E	F	G	H	LENGTH	v ASD = 0.7 V / L	TYPE	CAPACITY	H	W	H / W		2 W / H	ADJUSTED CAPACITY
1	4122.42 lb	6.00 ft +	5.00 ft +	4.00 ft +	ft +	ft +	ft +	ft +	ft =	15.00 ft	192.38 plf	SIP6	360.00	10.00	4.00	2.50	x	0.80	288.00
2	4122.42 lb	5.50 ft +	7.00 ft +	ft +	ft +	ft +	ft +	ft +	ft =	12.50 ft	230.86 plf	SIP6	360.00	10.00	5.50	1.82			360.00
3	3074.28 lb	7.00 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	7.00 ft	307.43 plf	SIP6	360.00	10.00	7.00	1.43			360.00
4	4644.77 lb	3.50 ft +	3.50 ft +	ft +	ft +	ft +	ft +	ft +	ft =	7.00 ft	464.48 plf	SIP2	920.00	10.00	3.50	2.86	x	0.70	644.00
5	4785.70 lb	13.50 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	13.50 ft	248.15 plf	SIP6	360.00	9.00	13.50	0.67			360.00
6	4785.70 lb	13.50 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	13.50 ft	248.15 plf	SIP6	360.00	9.00	13.50	0.67			360.00
7	3708.48 lb	24.00 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	24.00 ft	108.16 plf	SIP6	360.00	9.00	24.00	0.38			360.00
8	5353.03 lb	3.50 ft +	3.50 ft +	ft +	ft +	ft +	ft +	ft +	ft =	7.00 ft	535.30 plf	SIP2	920.00	9.00	3.50	2.57	x	0.78	715.56
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							

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WIND SHEAR FORCES TO SHEAR LINES AND ALLOWABLE STRESS SHEAR WALL FORCES:

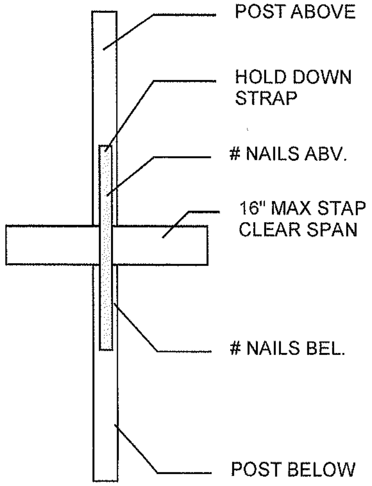
ALLOW FOR H/W < 3.5/1 PER CBC TA 2305.3.4

LINE	V ASD	A	B	C	D	E	F	G	H	LENGTH	v ASD = V / L	TYPE	CAPACITY	H	W	H / W < 3.5	ADJUSTED CAPACITY
1	1368.82 lb	6.00 ft +	5.00 ft +	4.00 ft +	ft +	ft +	ft +	ft +	ft =	15.00 ft	91.25 plf	SIP6	360.00	10.00	4.00	2.50	360.00 plf
2	1368.82 lb	5.50 ft +	7.00 ft +	ft +	ft +	ft +	ft +	ft +	ft =	12.50 ft	109.51 plf	SIP6	360.00	10.00	5.50	1.82	360.00 plf
3	1368.82 lb	7.00 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	7.00 ft	195.55 plf	SIP6	360.00	10.00	7.00	1.43	360.00 plf
4	2068.08 lb	3.50 ft +	3.50 ft +	ft +	ft +	ft +	ft +	ft +	ft =	7.00 ft	295.44 plf	SIP2	920.00	10.00	3.50	2.86	920.00 plf
5	4106.46 lb	13.50 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	13.50 ft	304.18 plf	SIP6	360.00	9.00	13.50	0.67	360.00 plf
6	4106.46 lb	13.50 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	13.50 ft	304.18 plf	SIP6	360.00	9.00	13.50	0.67	360.00 plf
7	4106.46 lb	24.00 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	24.00 ft	171.10 plf	SIP6	360.00	9.00	24.00	0.38	360.00 plf
8	6204.24 lb	3.50 ft +	3.50 ft +	ft +	ft +	ft +	ft +	ft +	ft =	7.00 ft	886.32 plf	SIP2	920.00	9.00	3.50	2.57	920.00 plf
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					

STRAP HOLD DOWN CAPACITY SCHEDULE:

UPLIFT CAPACITY IS BASED ON THE MINIMUM RATING x 0.75
SIMPSON STRAPS

HOLD DOWN	MFG RATING	POST	TEST RATING	LA REPORT	ICC REPORT	LARR 25910 25% REDUCTION	MAX DEFLECTION	NAIL SIZE	TOTAL NAILS
MST37	2710.00 lbs	4x	2710.00 lbs	RR25713	ESR-2105	2032.50 lbs	0.030	16d	22
MST48	4205.00 lbs	4x	4205.00 lbs	RR25713	ESR-2105	3153.75 lbs	0.030	16d	34
MST60	4605.00 lbs	4x	4605.00 lbs	RR25713	ESR-2105	3453.75 lbs	0.030	16d	34
MST72	6505.00 lbs	4x	6505.00 lbs	RR25713	ESR-2105	4878.75 lbs	0.030	16d	48
CMST12-42	10710.00 lbs	4x	9215.00 lbs	RR25713	ESR-2105	6911.25 lbs	0.030	16d	84
CMST14-33	7755.00 lbs	4x	6490.00 lbs	RR25713	ESR-2105	4867.50 lbs	0.030	16d	66
CS16-11	2080.00 lbs	4x	1705.00 lbs	RR25713	ESR-2105	1278.75 lbs	0.030	10d	22



MST STRAP CALCULATION

STRAP	GAGE	NAIL CAPACITY	# NAILS	TOTAL CAP.
MST37	12	149.00	11.00	1639.00
MST48	12	149.00	17.00	2533.00
MST60	10	154.00	17.00	2618.00
MST72	10	154.00	24.00	3696.00

NAIL VALUES BASE ON 2012 NDS 11P - 16d w/ G=0.5

HOLD DOWN CAPACITY SCHEDULE:

HOLDOWN CAPACITIES BASED ON CODE TABLES AND RESEARCH REPORTS

SIMPSON HOLD DOWN CAPACITIES

SIMPSON HOLD DOWN	POST SIZE	ANCHOR DIAMETER	CATALOG CAPACITY lbs	da in	ICC REPORT	ICC CAPACITY lbs	ICC STRENGTH lbs	da in	LARR REPORT	75% CAP. lbs
HDU2-SDS2.5	4x4	0.625	3075.00	0.088	ICC-ESR 2330	3505.00	4907.00	0.088	25720	2628.75
HDU4-SDS2.5	4x4	0.625	4565.00	0.114	ICC-ESR 2330	4990.00	6986.00	0.114	25720	3742.50
HDU5-SDS2.5	4x4	0.625	5645.00	0.115	ICC-ESR 2330	5670.00	7938.00	0.115	25720	4252.50
HDQ8-SDS3	4x6	0.875	9230.00	0.095	ICC-ESR 2330	9230.00	12922.00	0.095	25720	6922.50
HDU11-SDS2.5	6x6	1.000	11175.00	0.137	ICC-ESR 2330	11175.00	15645.00	0.137	25270	8381.25
HHDQ14-SDS2.5	6x6	1.000	13710.00	0.107	ICC-ESR 2330	13710.00	19194.00	0.107	25270	10282.50
HD19	6x6	1.25	19070.00	0.137	ICC ES-0143	19070.00	26698.00	0.137	25828	19371.00
ZONE FOUR 48-9X	(2) 6x6	1.125	31174.00	0.032	ICC-ESR 5302	31174.00	43643.60	0.032	25334	31174.00

ANCHOR BOLTS IN 2,500 psi CONCRETE
TABLES FROM SIMPSON CATALOG C-2009

SIMPSON ANCHOR	ANCHOR BOLT	ANCHOR DIAMETER	CAPACITY lbs	de in	F in	ICC REPORT	ICC CAPACITY lbs	ICC STRENGTH lbs	LARR REPORT	LARR CAP lbs
SSTB16	5/8	0.625	4420.0	12.6	1.75	N.A.			25248	2695.00
SSTB20	5/8	0.625	4600.0	16.6	1.75	N.A.			25248	2987.00
SSTB24	5/8	0.625	5175.0	20.6	1.75	N.A.			25248	3360.00
SSTB28	7/8	0.875	10100.0	24.9	1.75	N.A.			25248	6558.00
	1	1.00	14120.0	10.0	15.00	N.A.			-	-
	1 1/8	1.13	45808.0	14.0	7.00	ICC-ESR 5302			25828	45808.00
	1 1/4	1.250	22580.0	14.0	21.00	N.A.			-	-

HOLD DOWN SUMMARY

SIMPSON HOLD DOWN	**POST** SIZE	**ANCHOR** LA CITY	**ANCHOR** ALTERNATE	**75% ASSEMBLY CAPACITY** lbs
HDU2-SDS2.5	4x4	SSTB16	5/8	2628.75
HDU4-SDS2.5	4x4	SSTB24	5/8	3360.00
HDU5-SDS2.5	4x4	SSTB24	5/8	3360.00
HDQ8-SDS3	4x6	SSTB28	7/8	6558.00
HDU11-SDS2.5	6x6	SEE ALT >	1	8381.25
HHDQ14-SDS2.5	6x8	SEE ALT >	1	10282.50
HD19	6x6	SEE ALT >	1 1/4	19371.00
ZONE FOUR 48-9X	(2) 6x6	SEE ALT >	1 1/8	31174.00
				USE FOR UPLIFT DESIGN

100% ASSEMBLY CAPACITY lbs	**ds** in
2695.00	0.1180
3360.00	0.1540
3360.00	0.1580
9230.00	0.1300
11175.00	0.1820
13710.00	0.1440
25828.00	0.1855
31174.00	0.032
USE FOR DEFLECTION DESIGN	

SEISMIC OVERTURNING DESIGN:

ASD LEVEL FORCES
USE EQUATION ASCE 7-11 12.4.2.3 #8
(0.6 - 0.14 Sds) D + 0.7 ρ Qe
Sds = 0.633
0.6 - 0.14 Sds = 0.511
USE THE LESSER OF ICC-ES AND LARR VALUES FOR HOLD DOWNS...CAPACITY IS BASED ON 0.75 x HOLD DOWN ALLOWABLE STRESS DESIGN CAPACITY

LINE #		LENGTH	HEIGHT	ADJUSTED ASD	OVERTURNING	0.7 ρ Qe	WEIGHT	ADD LOAD	ADD LOAD	(0.6-0.14Sds) D RST	OVERTURNING	+/- LEVER ARM	ADDED UPLIFT	WALL	NET UPLIFT	HOLDOWN	CAPACITY
#		ft	ft	plf	ft-lb	lb	plf	plf	lb	ft-lb	ft-lb	ft	lb	ABV.	lb	TYPE	lb
1	A	6.00	10.00	192.38	11542.8	1923.8	150.0	370.0		4786.8	6756.0	-0.50		-	1228.4	MST37	2032.5
1	B	5.00	10.00	192.38	9619.0	1923.8	150.0	370.0		3324.2	6294.8	-0.50		-	1398.8	MST37	2032.5
1	C	4.00	10.00	192.38	7695.2	1923.8	150.0	100.0		1022.8	6672.4	-0.50		-	1906.4	MST37	2032.5
2	A	5.50	10.00	230.86	12697.1	2308.6	150.0	100.0		1933.8	10763.3	-0.50		-	2152.7	MST48	3153.8
2	B	7.00	10.00	230.86	16159.9	2308.6	150.0	50.0		2505.9	13654.0	-0.50		-	2100.6	MST48	3153.8
3	A	7.00	10.00	307.43	21519.9	3074.3	150.0	50.0		2505.9	19014.0	-0.50		-	2925.2	MST48	3153.8
4	A	3.50	10.00	464.48	16256.7	4644.8	150.0	100.0		783.1	15473.6	-0.50		-	5157.9	CMST12-45	6911.3
4	B	3.50	10.00	464.48	16256.7	4644.8	150.0	100.0		783.1	15473.6	-0.50		-	5157.9	CMST12-45	6911.3
5	A	13.50	9.00	55.77	6775.8	501.9	100.0	100.0		9320.5	-2544.7	-0.50	1228.36	1A	1032.6	HDQ8-SDS3	6558.0
6	A	13.50	9.00	17.29	2101.0	155.6	100.0	100.0		9320.5	-7219.5	-0.50	2152.66	2A	1597.3	HDQ8-SDS3	6558.0
7	A	24.00	9.00	-199.26	-43041.0	-1793.4	100.0	100.0		29457.4	-72498.4	-0.50	2925.23	3A	-159.8	HDQ8-SDS3	6558.0
8	A	3.50	9.00	70.83	2231.0	637.4	100.0	100.0		626.5	1604.5	-0.50	5157.87	4A	5692.7	HHDQ14-SDS2.5	10282.5
8	B	3.50	9.00	70.83	2231.0	637.4	100.0	100.0		626.5	1604.5	-0.50	5157.87	4B	5692.7	HHDQ14-SDS2.5	10282.5
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WIND OVERTURNING DESIGN:

ASD LEVEL FORCES
USE EQUATION ASCE 7-11 2.4.1 #7
0.6D + W
USE THE LESSER OF ICC-ES AND LARR VALUES FOR HOLD DOWNS

LINE # #	LENGTH ft		HEIGHT ft	ASD FORCE plf	OVERTURNING ft-lb	W lb	WEIGHT plf	ADD LOAD plf	ADD LOAD lb	0.6 D RST ft-lb	OVERTURNING ft-lb	+/- LEVER ARM ft	ADDED UPLIFT lb	WALL ABV.	NET UPLIFT lb	HOLDOWN TYPE	CAPACITY lb
1	6.00	A	10.00	91.25	5475.28	912.55	150.00	370.0		5616.00	-140.72	-0.50		-	-25.59	MST37	2032.50
1	5.00	B	10.00	91.25	4562.73	912.55	150.00	370.0		3900.00	662.73	-0.50		-	147.27	MST37	2032.50
1	4.00	C	10.00	91.25	3650.18	912.55	150.00	100.0		1200.00	2450.18	-0.50		-	700.05	MST37	2032.50
2	5.50	A	10.00	109.51	6022.80	1095.06	150.00	100.0		2268.75	3754.05	-0.50		-	750.81	MST48	3153.75
2	7.00	B	10.00	109.51	7665.39	1095.06	150.00	50.0		2940.00	4725.39	-0.50		-	726.98	MST48	3153.75
3	7.00	A	10.00	195.55	13688.19	1955.46	150.00	50.0		2940.00	10748.19	-0.50		-	1653.57	MST48	3153.75
4	3.50	A	10.00	295.44	10340.41	2954.40	150.00	100.0		918.75	9421.66	-0.50		-	3140.55	CMST12-45	6911.25
4	3.50	B	10.00	295.44	10340.41	2954.40	150.00	100.0		918.75	9421.66	-0.50		-	3140.55	CMST12-45	6911.25
5	13.50	A	9.00	212.93	25870.68	1916.35	100.00	100.0		10935.00	14935.68	-0.50		1A	1148.90	HDQ8-SDS3	6558.00
6	13.50	A	9.00	194.68	23653.19	1752.09	100.00	100.0		10935.00	12718.19	-0.50	750.81	2A	1729.13	HDQ8-SDS3	6558.00
7	24.00	A	9.00	-24.44	-5279.73	-219.99	100.00	100.0		34560.00	-39839.73	-0.50	1653.57	3A	-41.74	HDQ8-SDS3	6558.00
8	3.50	A	9.00	590.88	18612.73	5317.92	100.00	100.0		735.00	17877.73	-0.50	3140.55	4A	9099.80	HHDQ14-SDS2.5	10282.50
8	3.50	B	9.00	590.88	18612.73	5317.92	100.00	100.0		735.00	17877.73	-0.50	3140.55	4B	9099.80	HHDQ14-SDS2.5	10282.50
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SHEAR WALL SEISMIC LOAD DEFLECTION CHECK - FLEXIBLE DIAPHRAGM ASSUMPTION:

STRENGTH LEVEL DESIGN

STRUCT I PLY - SPECIES GROUP 1, GRADE STESS LEVEL S-2 - E = 1,800,000 psi G = 90,000 psi & 19% MOISTURE CONTENT ASSUMED

I = 1.00
δ xe = (8vh^3 / Eab + vh/1000Ga + Σ da h/b)
δ x = Cd δxe / I
δ x' = (δx x 1.00)
Δa = 0.0250 hsx
ρ = 1.30 PER ASCE 7-10 12.3.4.1 ρ = 1.0 FOR DRIFT CALCULATIONS - ADJUST v ASD BY 1 / ρ

					STRENGTH			ASD		HOLDOWN		USE		VERTICAL ELONGATION								COMPARE	
LINE #	LENGTH ft	h ft	PLY TYPE	v ASD plf	1.4 v / ρ plf	HOLDOWN TYPE	A POST in^2	T HD / ρ lb	Ta HD lb	D/C RATIO	ds in	ds D/C in	Ga	ds SLIP in	ds CRUSH in	ds SHRINK in	8vh^3 /Eab in	vh/1000Ga in	Σ ds x h/b in	δ xe in	Cd	δ x' in	Δa in
1	6.00	10.00	SIP6	192.4	207.2	MST37	12.25	945	2033	0.4649	0.030	0.014	13.0	0.010	0.020	0.050	0.013	0.159	0.157	0.336	4.0	1.34	3.00
1	5.00	10.00	SIP6	192.4	207.2	MST37	12.25	1076	2033	0.5294	0.030	0.016	13.0	0.010	0.020	0.050	0.015	0.159	0.192	0.374	4.0	1.50	3.00
1	4.00	10.00	SIP6	192.4	207.2	MST37	12.25	1466	2033	0.7215	0.030	0.022	13.0	0.010	0.020	0.050	0.019	0.159	0.254	0.440	4.0	1.76	3.00
2	5.50	10.00	SIP6	230.9	248.6	MST48	12.25	1656	3154	0.5251	0.030	0.016	13.0	0.010	0.020	0.050	0.016	0.191	0.174	0.393	4.0	1.57	3.00
2	7.00	10.00	SIP6	230.9	248.6	MST48	12.25	1616	3154	0.5124	0.030	0.015	13.0	0.010	0.020	0.050	0.013	0.191	0.136	0.351	4.0	1.41	3.00
3	7.00	10.00	SIP6	307.4	331.1	MST48	12.25	2250	3154	0.7135	0.030	0.021	13.0	0.010	0.020	0.050	0.017	0.255	0.145	0.436	4.0	1.74	3.00
4	3.50	10.00	SIP2	464.5	500.2	CMST12-45	12.25	3968	6911	0.5741	0.030	0.017	24.0	0.010	0.020	0.050	0.052	0.208	0.278	0.543	4.0	2.17	3.00
4	3.50	10.00	SIP2	464.5	500.2	CMST12-45	12.25	3968	6911	0.5741	0.030	0.017	24.0	0.010	0.020	0.050	0.052	0.208	0.278	0.543	4.0	2.17	3.00
5	13.50	9.00	SIP6	248.1	267.2	HDQ8-SDS3	19.25	794	6558	0.1211	0.130	0.016	13.0	0.010	0.020	0.050	0.003	0.185	0.064	0.263	4.0	1.05	2.70
6	13.50	9.00	SIP6	248.1	267.2	HDQ8-SDS3	19.25	1229	6558	0.1874	0.130	0.024	13.0	0.010	0.020	0.050	0.003	0.185	0.070	0.269	4.0	1.08	2.70
7	24.00	9.00	SIP6	108.2	116.5	HDQ8-SDS3	19.25	-123	6558	-0.0187	0.130		13.0	0.010	0.020	0.050	0.001	0.081	0.030	0.114	4.0	0.46	2.70
8	3.50	9.00	SIP2	535.3	576.5	HHDQ14-SDS2.5	39.88	4379	10283	0.4259	0.144	0.061	24.0	0.010	0.020	0.050	0.013	0.216	0.363	0.599	4.0	2.40	2.70
8	3.50	9.00	SIP2	535.3	576.5	HHDQ14-SDS2.5	39.88	4379	10283	0.4259	0.144	0.061	24.0	0.010	0.020	0.050	0.013	0.216	0.363	0.599	4.0	2.40	2.70
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SHEAR WALL SUMMARY:

COMPARE SEISMIC AND WIND REQUIREMENTS FOR FINAL RESULTS

LINE #	LENGTH ft	SEISMIC FORCE plf	WIND FORCE plf	GOVERNING FORCE plf	WALL TYPE PER SCHED.	HOLD DOWN SIMPSON	WALL CAPACITY plf	
1	6.0	192.4	91.3	192.4	SIP6	MST37	360.0	S
1	5.0	192.4	91.3	192.4	SIP6	MST37	360.0	S
1	4.0	192.4	91.3	192.4	SIP6	MST37	360.0	S
2	5.5	230.9	109.5	230.9	SIP6	MST48	360.0	S
2	7.0	230.9	109.5	230.9	SIP6	MST48	360.0	S
3	7.0	307.4	195.5	307.4	SIP6	MST48	360.0	S
4	3.5	464.5	295.4	464.5	SIP2	CMST12-45	920.0	S
4	3.5	464.5	295.4	464.5	SIP2	CMST12-45	920.0	S
5	13.5	248.1	304.2	304.2	SIP6	HDQ8-SDS3	360.0	W
6	13.5	248.1	304.2	304.2	SIP6	HDQ8-SDS3	360.0	W
7	24.0	108.2	171.1	171.1	SIP6	HDQ8-SDS3	360.0	W
8	3.5	535.3	886.3	886.3	SIP2	HHDQ14-SDS2.5	920.0	W
8	3.5	535.3	886.3	886.3	SIP2	HHDQ14-SDS2.5	920.0	W
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						-		-
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						-		-
						-		-

OVERSTRENGTH LOAD SUMMARY

ASD LEVEL FORCES

USE EQUATION ASCE 7-11 12.4.3.2 #7

(0.6 - 0.14 Sds) D + Ωo Qe

Sds = 0.633

0.6 - 0.14 Sds = 0.511

Ωo = 3.00

LINE # #		LENGTH ft	HEIGHT ft	ADJUSTED V ASD plf	Ωo OVERTURNING ft-lb	Ωo Qe lb	WEIGHT plf	ADD LOAD plf	ADD LOAD lb	(0.6-0.14Sds) D RST ft-lb	NET O.T. ft-lb	+/- LEVER ARM ft	ADDED UPLIFT lb	WALL ABV.	OVERSTRENGTH (0.6 - 0.14 Sds) D + Ωo Qe lbs
1	A	6.00	10.00	192.38	26637.2	10245.1	150.0	370.0		4786.8	21850.4	-0.50		-	3972.8
1	B	5.00	10.00	192.38	22197.7	10245.1	150.0	370.0		3324.2	18873.5	-0.50		-	4194.1
1	C	4.00	10.00	192.38	17758.1	10245.1	150.0	100.0		1022.8	16735.3	-0.50		-	4781.5
2	A	5.50	10.00	230.86	29300.9	12294.1	150.0	100.0		1933.8	27367.1	-0.50		-	5473.4
2	B	7.00	10.00	230.86	37292.1	12294.1	150.0	50.0		2505.9	34786.2	-0.50		-	5351.7
3	A	7.00	10.00	307.43	49661.4	16371.9	150.0	50.0		2505.9	47155.5	-0.50		-	7254.7
4	A	3.50	10.00	464.48	37515.5	24735.5	150.0	100.0		783.1	36732.4	-0.50		-	12244.1
4	B	3.50	10.00	464.48	37515.5	24735.5	150.0	100.0		783.1	36732.4	-0.50		-	12244.1
5	A	13.50	9.00	55.77	15636.5	2672.9	100.0	100.0		9320.5	6316.0	-0.50	3972.79	1A	4458.6
6	A	13.50	9.00	17.29	4848.4	828.8	100.0	100.0		9320.5	-4472.1	-0.50	5473.43	2A	5129.4
7	A	24.00	9.00	-199.26	-99325.3	-9550.5	100.0	100.0		29457.4	-128782.7	-0.50	7254.69	3A	1774.6
8	A	3.50	9.00	70.83	5148.5	3394.6	100.0	100.0		626.5	4522.0	-0.50	12244.13	4A	13751.4
8	B	3.50	9.00	70.83	5148.5	3394.6	100.0	100.0		626.5	4522.0	-0.50	12244.13	4B	13751.4
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														-	
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														-	

DIAPHRAGMS:

$F_{x_{ROOF}} = \left.\begin{matrix}264\\191\end{matrix}\right\} 264\ PLF$

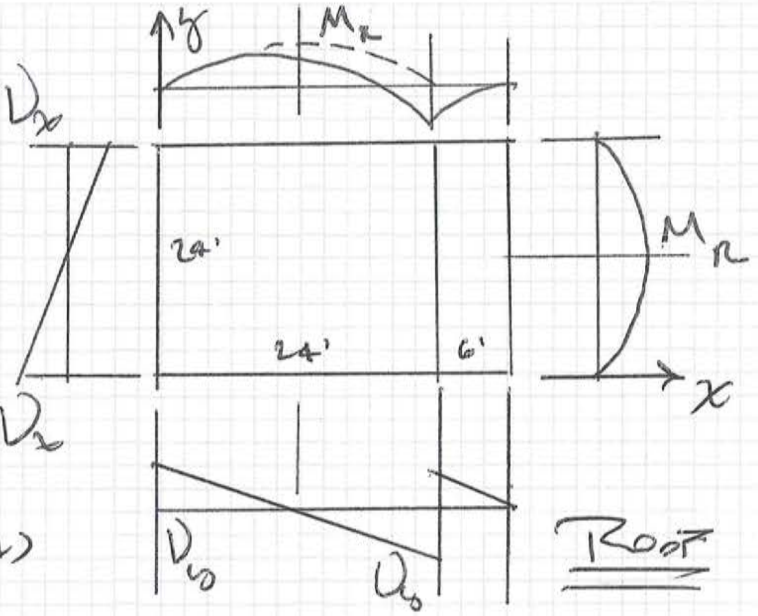
$F_{y_{ROOF}} = \left.\begin{matrix}197\\191\end{matrix}\right\} 197\ PLF$

STRENGTH SEISMIC

$V_{\substack{MAX\\ASD}} = 0.7(264)24/2(24)$

$= \underline{93\ PLF}$

ROOF



ROOF

STRENGTH WIND

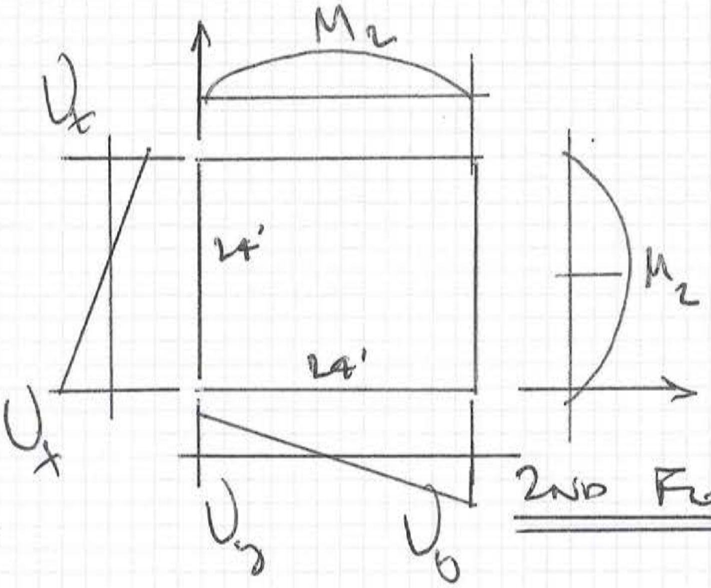
$F_{x_{2ND}} = \left.\begin{matrix}43\\380\end{matrix}\right\} 380\ PLF$

$F_{y_{2ND}} = \left.\begin{matrix}41\\380\end{matrix}\right\} 380\ PLF$

$V_{\substack{MAX\\ASD}} = 0.6(380)24/2(24)$

$= \underline{114\ PLF}$

2ND FLOOR



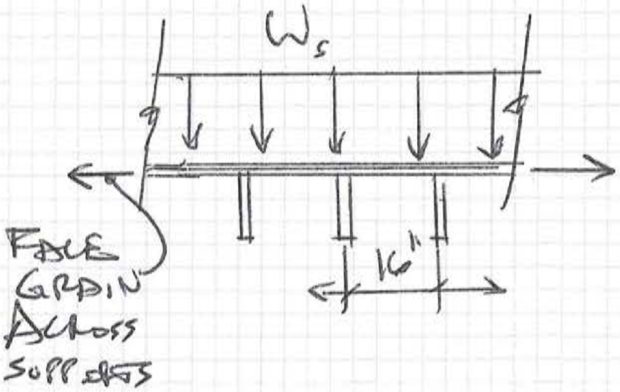
2ND FLOOR

ROOF: TYP 15/32 CDX / OSB w/ 8d @ 6" o.c. — CAP = 180 PLF
UNBLOCKED PI 32/16 — SEE NEXT PAGE...

FLOOR: TYP 5/8" STRUCT 1 or THICKER w/ 10d @ 6" o.c.
T&G PI 40/24 — CAP = 215 PLF

PLY W/ SNOW LOAD:

SPAN = 16" oc
W_s = 264 PSF



PER APA
REQ'D SPAN RATING
40/20 –

LIMITED TO 293 PSF MAX (BENDING GOVERNS)
$C_D = 1.15 \rightarrow 264/1.15 = \underline{229}$ PSF < 293 PSF OK

ROOF REQUIRES 5/8" OSB PLY
W/ PI 40/20 – T&G
W/ 10d @ 6", 12"

PROJECT	VILLAGE NEST	JOB	2017-0610
CLIENT	-		
ADDRESS	EDEN, UT		
DESIGN	HUUM		
ENGINEER	J.H.	DATE	6/14/2017
RICHMOND PE 49628 HOFFMAYER SE 3435		SHEET	36 OF

CHORDS & DRAGS:

$$\text{MAX CHORD} \Rightarrow \left(\overset{M_{max}}{0.6(380)\,24^2/8}\right)/24 = \underline{684\ \text{lb}}$$

USE SINGLE SIP TOP PLATE
w/ LSTA12 STRAP - CAP = 795 lb ✓

N.A.

USE

OPTION:

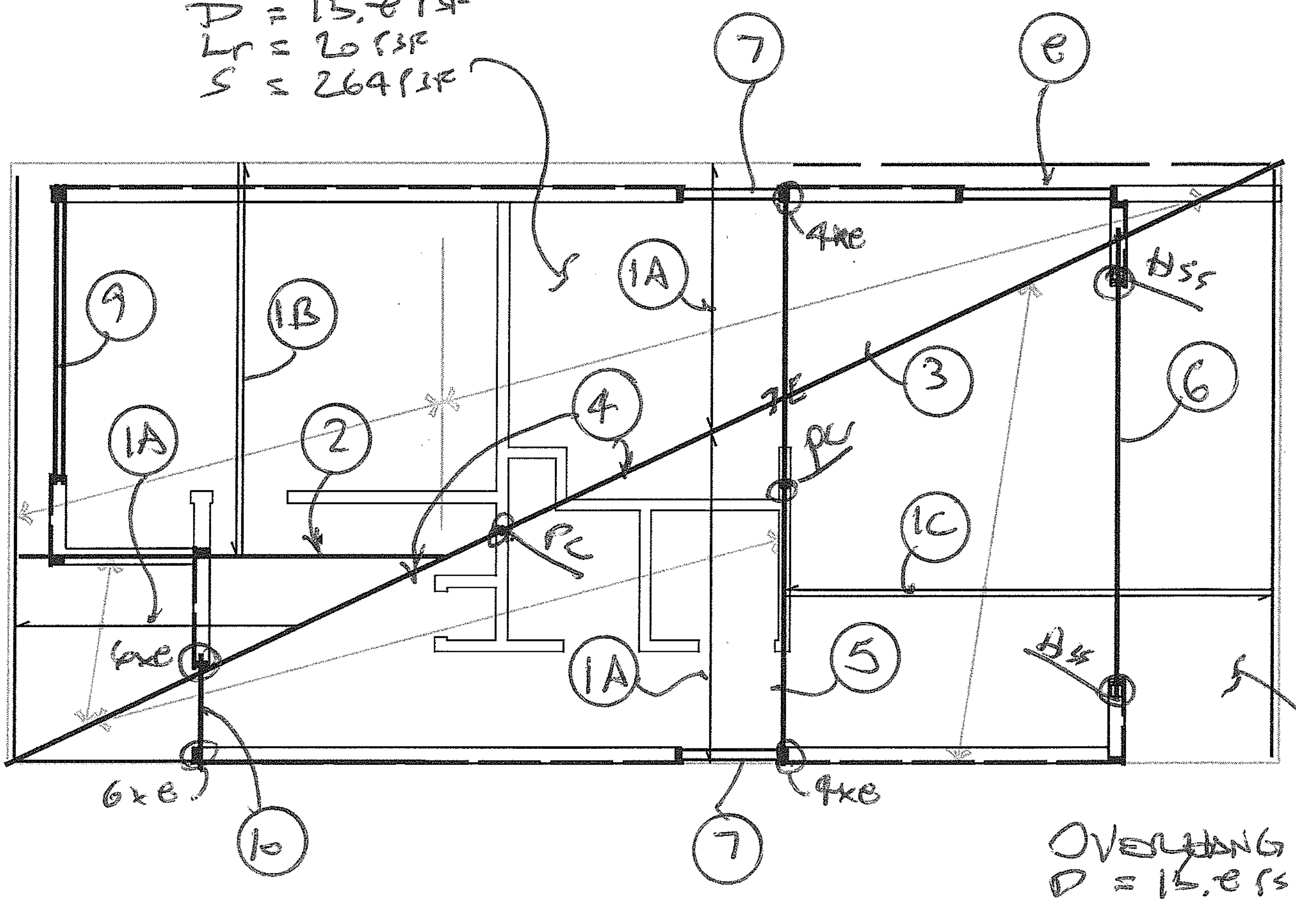
* USE SINGLE 2x TOP PLATE
w/ 1.3E RIM BOARD - SEE PG # 157

REQUIRED

SEE PLATE SPLICE, PG # 22-B

PROJECT FELDSPAR LOWER JOB 2017-0610
CLIENT VILLAGE NEST
ADDRESS EDEN, UTAH
DESIGN HUUM
ENGINEER J.H. DATE 6/16/2017
RICHMOND PE 49628 HOFFMAYER SE 3935 SHEET 137 OF

Roof
D = 15.8 psf
Lr = 20 psf
S = 264 psf


7
8
9
1B
1A
4x8
HSS
3
1A
2
4
PL
6
PL
1C
6x8
1A
5
HSS
6x8
4x8
10
7

OVERHANG
D = 15.8 psf
Lr = 20 psf
S = 444 psf

ROOF FRAMING PLAN
FELDSPAR - UPPER
N.T.S.

○: BEAM #

Wood Beam

File = d:\ENERCALC Projects\2017-0610.ec6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : B1A - RAFTERS

CODE REFERENCES

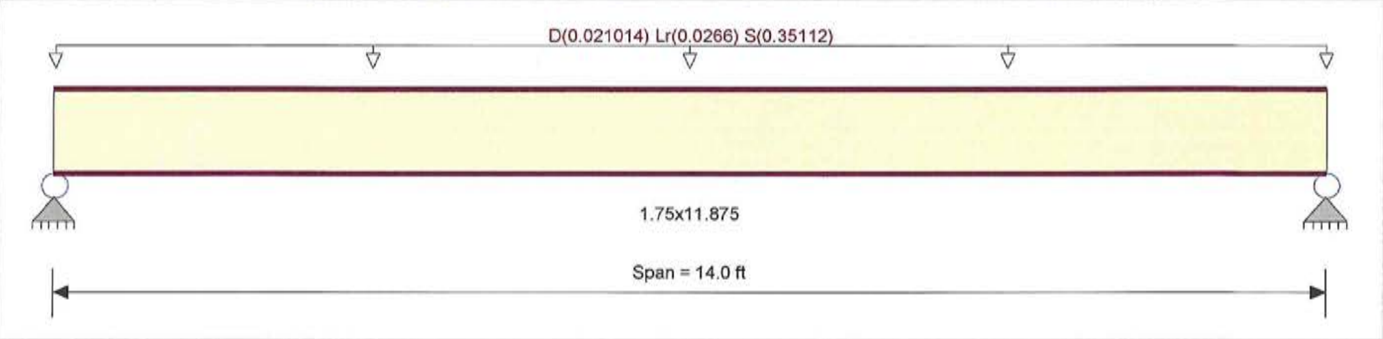
Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,600.0 psi	*E : Modulus of Elasticity*	
Load Combination	ASCE 7-10	Fb -	2,600.0 psi	Ebend- xx	1,900.0 ksi
		Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Wood Species	: Trus Joist	Fc - Perp	750.0 psi		
Wood Grade	: MicroLam LVL 1.9 E	Fv	285.0 psi		
		Ft	1,555.0 psi	Density	42.0 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.

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 1.330 ft, (ROOF LOADS)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.774 1	Maximum Shear Stress Ratio	=	0.494 : 1
Section used for this span		**1.75x11.875**	Section used for this span		**1.75x11.875**
fb : Actual	=	2,660.07psi	fv : Actual	=	161.95 psi
FB : Allowable	=	3,438.50psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	7.000ft	Location of maximum on span	=	13.029 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.658 in	Ratio =	255 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.697 in	Ratio =	240 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.605	2.605
Overall MINimum	0.088	0.088
D Only	0.147	0.147
+D+Lr	0.333	0.333
+D+S	2.605	2.605
+D+0.750Lr	0.287	0.287
+D+0.750S	1.990	1.990
+0.60D	0.088	0.088
Lr Only	0.186	0.186
S Only	2.458	2.458

Wood Beam

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Lic. # : KW-06002886 **Licensee : RICHMOND HOFFMAYER**

Description : B1B - RAFTERS

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

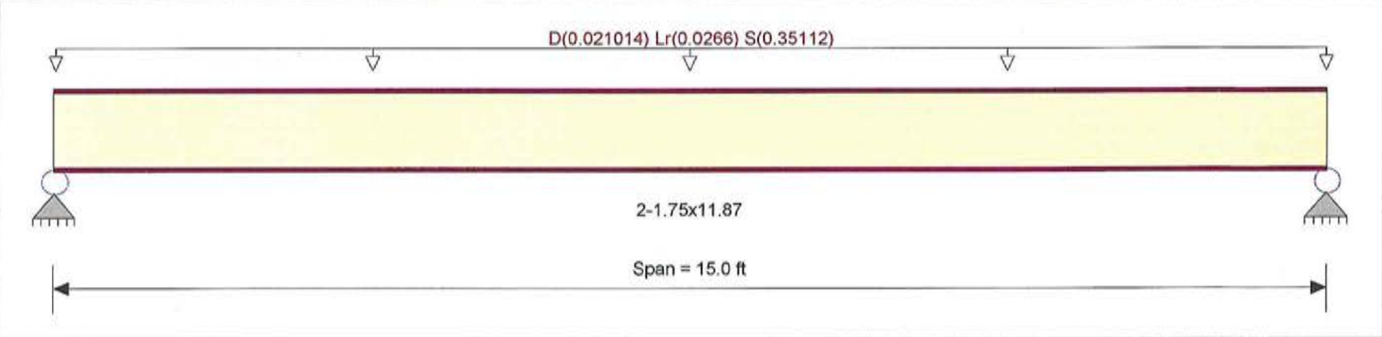
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,600.0 psi
Fb -	2,600.0 psi
Fc - Prll	2,510.0 psi
Fc - Perp	750.0 psi
Fv	285.0 psi
Ft	1,555.0 psi

E : Modulus of Elasticity

Ebend- xx	1,900.0 ksi
Eminbend - xx	965.71 ksi
Density	42.0 pcf

Repetitive Member Stress Increase



Applied Loads

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

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 1.330 ft, (ROOF LOADS)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.444 : 1	Maximum Shear Stress Ratio	=	0.267 : 1
Section used for this span		2-1.75x11.87	Section used for this span		2-1.75x11.87
fb : Actual	=	1,526.82 psi	fv : Actual	=	87.49 psi
FB : Allowable	=	3,438.50 psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	7.500 ft	Location of maximum on span	=	14.015 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.434 in	Ratio =	415 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.459 in	Ratio =	391 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.791	2.791
Overall MINimum	0.095	0.095
D Only	0.158	0.158
+D+Lr	0.357	0.357
+D+S	2.791	2.791
+D+0.750Lr	0.307	0.307
+D+0.750S	2.133	2.133
+0.60D	0.095	0.095
Lr Only	0.200	0.200
S Only	2.633	2.633

Wood Beam

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Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : B1C - RAFTERS w OVERHANG

CODE REFERENCES

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

Material Properties

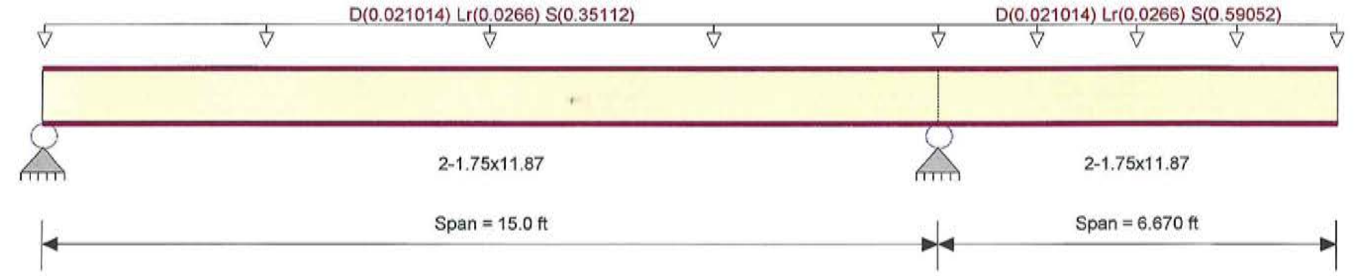
Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

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

Fb +	2,600.0 psi	*E : Modulus of Elasticity*	
Fb -	2,600.0 psi	Ebend- xx	1,900.0 ksi
Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Fc - Perp	750.0 psi		
Fv	285.0 psi		
Ft	1,555.0 psi	Density	42.0 pcf

Repetitive Member Stress Increase


D(0.021014) Lr(0.0266) S(0.35112)
D(0.021014) Lr(0.0266) S(0.59052)
2-1.75x11.87
2-1.75x11.87
Span = 15.0 ft
Span = 6.670 ft

Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 1.330 ft, (ROOF LOADS)
Load for Span Number 2
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.4440 ksf, Tributary Width = 1.330 ft, (ROOF LOADS)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.577: 1	Maximum Shear Stress Ratio	=	0.384 : 1
Section used for this span		2-1.75x11.87	Section used for this span		2-1.75x11.87
fb : Actual	=	1,984.45psi	fv : Actual	=	125.83 psi
FB : Allowable	=	3,438.50psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	15.000ft	Location of maximum on span	=	15.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.474 in	Ratio =	338 >=240.
Max Upward Transient Deflection	-0.046 in	Ratio =	3444 >=240.
Max Downward Total Deflection	0.475 in	Ratio =	336 >=240.
Max Upward Total Deflection	-0.044 in	Ratio =	3598 >=240.

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	1.884	7.777	
Overall MINimum	-0.039	0.197	
D Only	0.126	0.329	
+D+Lr, LL Comb Run (*L)	0.087	0.546	
+D+Lr, LL Comb Run (L*)	0.326	0.528	
+D+Lr, LL Comb Run (LL)	0.286	0.745	
+D+S	1.884	7.777	
+D+0.750Lr, LL Comb Run (*L)	0.097	0.492	
+D+0.750Lr, LL Comb Run (L*)	0.276	0.479	
+D+0.750Lr, LL Comb Run (LL)	0.246	0.641	

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Wood Beam

Description : B1C - RAFTERS w OVERHANG

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
+D+0.750S	1.445	5.915	
+0.60D	0.076	0.197	
Lr Only, LL Comb Run (*L)	-0.039	0.217	
Lr Only, LL Comb Run (L*)	0.200	0.200	
Lr Only, LL Comb Run (LL)	0.160	0.416	
S Only	1.758	7.448	

JA2

Wood Beam

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

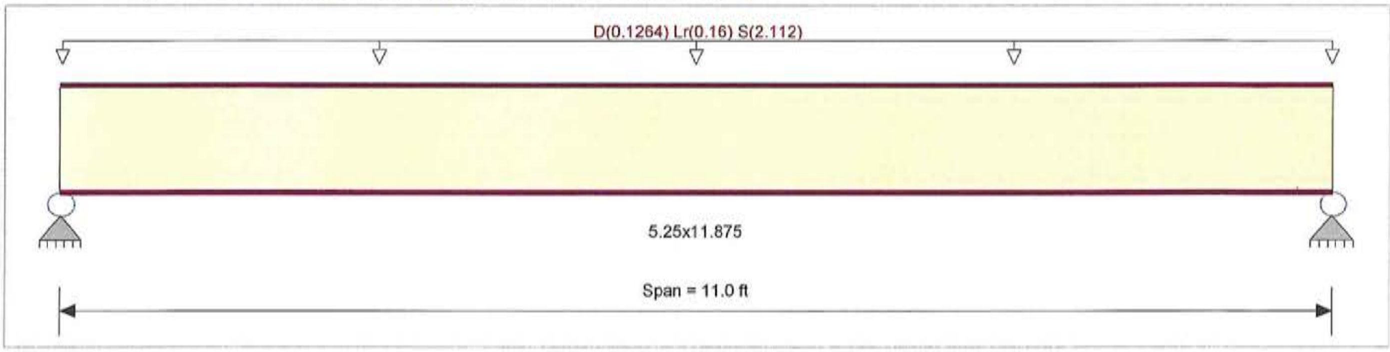
Description : B2 - ROOF BEAM

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-10	Fb -	2,900.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,900.0 psi	Eminbend - xx	1,016.54 ksi
Wood Species	: Trus Joist	Fc - Perp	625.0 psi		
Wood Grade	: Parallam PSL 2.0E	Fv	290.0 psi		
		Ft	2,025.0 psi	Density	45.050 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 8.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.996: 1	Maximum Shear Stress Ratio	=	0.739 : 1
Section used for this span		**5.25x11.875**	Section used for this span		**5.25x11.875**
fb : Actual	=	3,321.29psi	fv : Actual	=	246.45 psi
FB : Allowable	=	3,335.00psi	Fv : Allowable	=	333.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.500ft	Location of maximum on span	=	10.036ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.478 in	Ratio =	276>=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0<240.0
Max Downward Total Deflection	0.511 in	Ratio =	258>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0<240.0

Vertical Reactions

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

Load Combination	Support 1	Support 2
Overall MAXimum	12.418	12.418
Overall MINimum	0.481	0.481
D Only	0.802	0.802
+D+Lr	1.682	1.682
+D+S	12.418	12.418
+D+0.750Lr	1.462	1.462
+D+0.750S	9.514	9.514
+0.60D	0.481	0.481
Lr Only	0.880	0.880
S Only	11.616	11.616

Steel Beam

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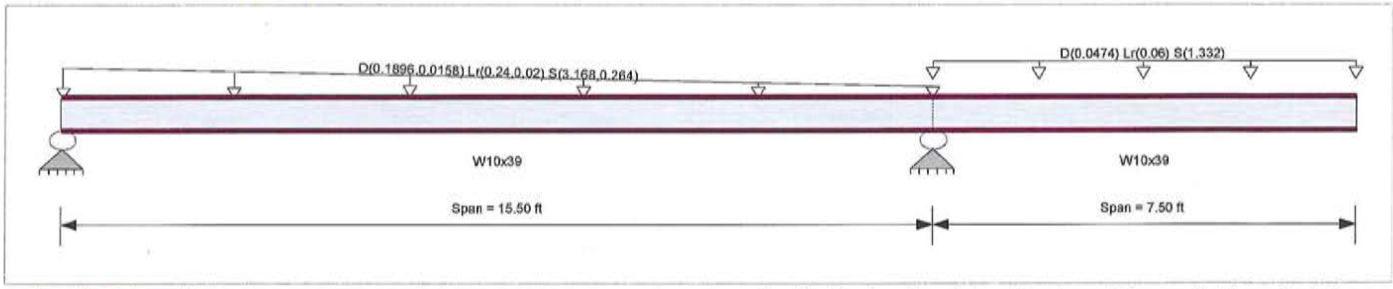
Description : B3 - RIDGE AT OVERHANG

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		


D(0.1896,0.0158) Lr(0.24,0.02) S(3.168,0.264)
D(0.0474) Lr(0.06) S(1.332)
W10x39
W10x39
Span = 15.50 ft
Span = 7.50 ft

Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 0.0 -->> 15.50 ft, Trib Width = 1
Load for Span Number 2
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.4440 ksf, Tributary Width = 3.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.351 : 1	Maximum Shear Stress Ratio =	0.253 : 1
Section used for this span	W10x39	Section used for this span	W10x39
Ma : Applied	41.029 k-ft	Va : Applied	15.799 k
Mn / Omega : Allowable	116.766 k-ft	Vn/Omega : Allowable	62.496 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	5.580ft	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	0.215 in	Ratio =	866 >=360
Max Upward Transient Deflection	-0.041 in	Ratio =	4,422 >=360
Max Downward Total Deflection	0.235 in	Ratio =	791 >=180
Max Upward Total Deflection	-0.049 in	Ratio =	3657 >=180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 15.50 ft	1	0.028	0.019	3.28	-2.43	3.28	195.00	116.77	1.00	1.00	1.17	93.74	62.50
Dsgn. L = 7.50 ft	2	0.021	0.010		-2.43	2.43	195.00	116.77	1.00	1.00	0.65	93.74	62.50
+D+Lr, LL Comb Run (*L)													
Dsgn. L = 15.50 ft	1	0.035	0.018	2.67	-4.12	4.12	195.00	116.77	1.00	1.00	1.14	93.74	62.50
Dsgn. L = 7.50 ft	2	0.035	0.018		-4.12	4.12	195.00	116.77	1.00	1.00	1.10	93.74	62.50
+D+Lr, LL Comb Run (L*)													
Dsgn. L = 15.50 ft	1	0.062	0.039	7.24	-2.43	7.24	195.00	116.77	1.00	1.00	2.46	93.74	62.50
Dsgn. L = 7.50 ft	2	0.021	0.010		-2.43	2.43	195.00	116.77	1.00	1.00	0.65	93.74	62.50
+D+Lr, LL Comb Run (LL)													
Dsgn. L = 15.50 ft	1	0.056	0.038	6.57	-4.12	6.57	195.00	116.77	1.00	1.00	2.35	93.74	62.50
Dsgn. L = 7.50 ft	2	0.035	0.018		-4.12	4.12	195.00	116.77	1.00	1.00	1.10	93.74	62.50
+D+S													
Dsgn. L = 15.50 ft	1	0.351	0.253	41.03	-39.89	41.03	195.00	116.77	1.00	1.00	15.80	93.74	62.50
Dsgn. L = 7.50 ft	2	0.342	0.170		-39.89	39.89	195.00	116.77	1.00	1.00	10.64	93.74	62.50
+D+0.750Lr, LL Comb Run (*L)													
Dsgn. L = 15.50 ft	1	0.032	0.018	2.82	-3.70	3.70	195.00	116.77	1.00	1.00	1.11	93.74	62.50
Dsgn. L = 7.50 ft	2	0.032	0.016		-3.70	3.70	195.00	116.77	1.00	1.00	0.99	93.74	62.50
+D+0.750Lr, LL Comb Run (L*)													
Dsgn. L = 15.50 ft	1	0.054	0.034	6.25	-2.43	6.25	195.00	116.77	1.00	1.00	2.13	93.74	62.50
Dsgn. L = 7.50 ft	2	0.021	0.010		-2.43	2.43	195.00	116.77	1.00	1.00	0.65	93.74	62.50
+D+0.750Lr, LL Comb Run (LL)													

Steel Beam

Description : B3 - RIDGE AT OVERHANG

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 15.50 ft	1	0.049	0.033	5.75	-3.70	5.75	195.00	116.77	1.00	1.00	2.05	93.74	62.50
Dsgn. L = 7.50 ft	2	0.032	0.016		-3.70	3.70	195.00	116.77	1.00	1.00	0.99	93.74	62.50
+D+0.750S													
Dsgn. L = 15.50 ft	1	0.271	0.194	31.59	-30.53	31.59	195.00	116.77	1.00	1.00	12.14	93.74	62.50
Dsgn. L = 7.50 ft	2	0.261	0.130		-30.53	30.53	195.00	116.77	1.00	1.00	8.14	93.74	62.50
+0.60D													
Dsgn. L = 15.50 ft	1	0.017	0.011	1.97	-1.46	1.97	195.00	116.77	1.00	1.00	0.70	93.74	62.50
Dsgn. L = 7.50 ft	2	0.012	0.006		-1.46	1.46	195.00	116.77	1.00	1.00	0.39	93.74	62.50

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.2351	6.758		0.0000	0.000
	2	0.0000	6.758	+D+Lr	-0.0492	7.500

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	15.799	23.633	
Overall MINimum	-0.109	0.559	
D Only	1.166	1.678	
+D+Lr, LL Comb Run (*L)	1.057	2.237	
+D+Lr, LL Comb Run (L*)	2.458	2.402	
+D+Lr, LL Comb Run (LL)	2.349	2.961	
+D+S	15.799	23.633	
+D+0.750Lr, LL Comb Run (*L)	1.084	2.098	
+D+0.750Lr, LL Comb Run (L*)	2.135	2.221	
+D+0.750Lr, LL Comb Run (LL)	2.053	2.640	
+D+0.750S	12.141	18.145	
+0.60D	0.700	1.007	
Lr Only, LL Comb Run (*L)	-0.109	0.559	
Lr Only, LL Comb Run (L*)	1.292	0.723	
Lr Only, LL Comb Run (LL)	1.183	1.282	
S Only	14.633	21.955	

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Steel Beam

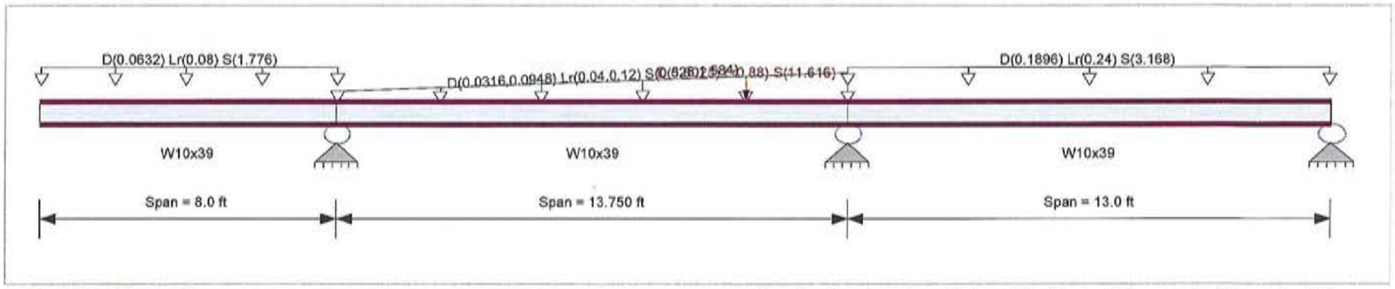
Description : B4 - RIDGE OVER ENTRY

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		


D(0.0632) Lr(0.08) S(1.776)
D(0.0316,0.0948) Lr(0.04,0.12) S(0.528,1.584)
D(0.8025) Lr(0.88) S(11.616)
D(0.1896) Lr(0.24) S(3.168)
W10x39
W10x39
W10x39
Span = 8.0 ft
Span = 13.750 ft
Span = 13.0 ft

Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.4440 ksf, Tributary Width = 4.0 ft, (ROOF)
Load for Span Number 2
Varying Uniform Load : D(S,E) = 0.01580->0.01580, Lr(S,E) = 0.020->0.020, S(S,E) = 0.2640->0.2640 ksf, Extent = 0.0 -->> 13.750 ft, Trib Width =
Point Load : D = 0.8025, Lr = 0.880, S = 11.616 k @ 11.0 ft, (B2)
Load for Span Number 3
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 12.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.515 : 1	Maximum Shear Stress Ratio =	0.411 : 1
Section used for this span	W10x39	Section used for this span	W10x39
Ma : Applied	60.102 k-ft	Va : Applied	25.661 k
Mn / Omega : Allowable	116.766 k-ft	Vn/Omega : Allowable	62.496 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	8.000 ft	Location of maximum on span	13.750 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.667 in	Ratio =	287 >=240.
Max Upward Transient Deflection	-0.105 in	Ratio =	1,571 >=240.
Max Downward Total Deflection	0.699 in	Ratio =	275 >=180
Max Upward Total Deflection	-0.108 in	Ratio =	1525 >=180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 8.00 ft	1	0.028	0.013		-3.27	3.27	195.00	116.77	1.00	1.00	0.82	93.74	62.50
Dsgn. L = 13.75 ft	2	0.031	0.028	0.23	-3.59	3.59	195.00	116.77	1.00	1.00	1.76	93.74	62.50
Dsgn. L = 13.00 ft	3	0.031	0.028	3.20	-3.59	3.59	195.00	116.77	1.00	1.00	1.76	93.74	62.50
+D+Lr, LL Comb Run (**L)													
Dsgn. L = 8.00 ft	1	0.028	0.013		-3.27	3.27	195.00	116.77	1.00	1.00	0.82	93.74	62.50
Dsgn. L = 13.75 ft	2	0.052	0.056	-0.00	-6.05	6.05	195.00	116.77	1.00	1.00	3.51	93.74	62.50
Dsgn. L = 13.00 ft	3	0.061	0.056	7.10	-6.05	7.10	195.00	116.77	1.00	1.00	3.51	93.74	62.50
+D+Lr, LL Comb Run (*L*)													
Dsgn. L = 8.00 ft	1	0.028	0.020		-3.27	3.27	195.00	116.77	1.00	1.00	1.26	93.74	62.50
Dsgn. L = 13.75 ft	2	0.047	0.047	2.42	-5.49	5.49	195.00	116.77	1.00	1.00	2.92	93.74	62.50
Dsgn. L = 13.00 ft	3	0.047	0.031	2.47	-5.49	5.49	195.00	116.77	1.00	1.00	1.91	93.74	62.50
+D+Lr, LL Comb Run (*LL)													
Dsgn. L = 8.00 ft	1	0.028	0.017		-3.27	3.27	195.00	116.77	1.00	1.00	1.08	93.74	62.50
Dsgn. L = 13.75 ft	2	0.068	0.059	1.02	-7.95	7.95	195.00	116.77	1.00	1.00	3.66	93.74	62.50
Dsgn. L = 13.00 ft	3	0.068	0.059	6.32	-7.95	7.95	195.00	116.77	1.00	1.00	3.66	93.74	62.50
+D+Lr, LL Comb Run (L**)													
Dsgn. L = 8.00 ft	1	0.050	0.023		-5.83	5.83	195.00	116.77	1.00	1.00	1.46	93.74	62.50
Dsgn. L = 13.75 ft	2	0.050	0.027	-0.00	-5.83	5.83	195.00	116.77	1.00	1.00	1.71	93.74	62.50

Steel Beam

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Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : B4 - RIDGE OVER ENTRY

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 13.00 ft	3	0.030	0.027	3.47	-2.93	3.47	195.00	116.77	1.00	1.00	1.71	93.74	62.50
+D+Lr, LL Comb Run (L*L)													
Dsgn. L = 8.00 ft	1	0.050	0.023		-5.83	5.83	195.00	116.77	1.00	1.00	1.46	93.74	62.50
Dsgn. L = 13.75 ft	2	0.050	0.055	-0.00	-5.83	5.83	195.00	116.77	1.00	1.00	3.46	93.74	62.50
Dsgn. L = 13.00 ft	3	0.063	0.055	7.38	-5.40	7.38	195.00	116.77	1.00	1.00	3.46	93.74	62.50
+D+Lr, LL Comb Run (LL*)													
Dsgn. L = 8.00 ft	1	0.050	0.024		-5.83	5.83	195.00	116.77	1.00	1.00	1.50	93.74	62.50
Dsgn. L = 13.75 ft	2	0.050	0.043	1.92	-5.83	5.83	195.00	116.77	1.00	1.00	2.69	93.74	62.50
Dsgn. L = 13.00 ft	3	0.041	0.030	2.72	-4.83	4.83	195.00	116.77	1.00	1.00	1.86	93.74	62.50
+D+Lr, LL Comb Run (LLL)													
Dsgn. L = 8.00 ft	1	0.050	0.023		-5.83	5.83	195.00	116.77	1.00	1.00	1.46	93.74	62.50
Dsgn. L = 13.75 ft	2	0.062	0.058	0.32	-7.30	7.30	195.00	116.77	1.00	1.00	3.61	93.74	62.50
Dsgn. L = 13.00 ft	3	0.062	0.058	6.59	-7.30	7.30	195.00	116.77	1.00	1.00	3.61	93.74	62.50
+D+S													
Dsgn. L = 8.00 ft	1	0.515	0.240		-60.10	60.10	195.00	116.77	1.00	1.00	15.03	93.74	62.50
Dsgn. L = 13.75 ft	2	0.515	0.411	-0.00	-60.10	60.10	195.00	116.77	1.00	1.00	25.66	93.74	62.50
Dsgn. L = 13.00 ft	3	0.431	0.411	50.35	-46.59	50.35	195.00	116.77	1.00	1.00	25.66	93.74	62.50
+D+0.750Lr, LL Comb Run (**L)													
Dsgn. L = 8.00 ft	1	0.028	0.013		-3.27	3.27	195.00	116.77	1.00	1.00	0.82	93.74	62.50
Dsgn. L = 13.75 ft	2	0.047	0.049	-0.00	-5.44	5.44	195.00	116.77	1.00	1.00	3.07	93.74	62.50
Dsgn. L = 13.00 ft	3	0.052	0.049	6.13	-5.44	6.13	195.00	116.77	1.00	1.00	3.07	93.74	62.50
+D+0.750Lr, LL Comb Run (*L*)													
Dsgn. L = 8.00 ft	1	0.028	0.018		-3.27	3.27	195.00	116.77	1.00	1.00	1.14	93.74	62.50
Dsgn. L = 13.75 ft	2	0.043	0.041	1.87	-5.02	5.02	195.00	116.77	1.00	1.00	2.55	93.74	62.50
Dsgn. L = 13.00 ft	3	0.043	0.030	2.65	-5.02	5.02	195.00	116.77	1.00	1.00	1.87	93.74	62.50
+D+0.750Lr, LL Comb Run (*LL)													
Dsgn. L = 8.00 ft	1	0.028	0.016		-3.27	3.27	195.00	116.77	1.00	1.00	1.01	93.74	62.50
Dsgn. L = 13.75 ft	2	0.059	0.051	0.81	-6.86	6.86	195.00	116.77	1.00	1.00	3.18	93.74	62.50
Dsgn. L = 13.00 ft	3	0.059	0.051	5.54	-6.86	6.86	195.00	116.77	1.00	1.00	3.18	93.74	62.50
+D+0.750Lr, LL Comb Run (L**)													
Dsgn. L = 8.00 ft	1	0.044	0.021		-5.19	5.19	195.00	116.77	1.00	1.00	1.30	93.74	62.50
Dsgn. L = 13.75 ft	2	0.044	0.028	-0.00	-5.19	5.19	195.00	116.77	1.00	1.00	1.72	93.74	62.50
Dsgn. L = 13.00 ft	3	0.029	0.028	3.40	-3.10	3.40	195.00	116.77	1.00	1.00	1.72	93.74	62.50
+D+0.750Lr, LL Comb Run (L*L)													
Dsgn. L = 8.00 ft	1	0.044	0.021		-5.19	5.19	195.00	116.77	1.00	1.00	1.30	93.74	62.50
Dsgn. L = 13.75 ft	2	0.044	0.049	-0.00	-5.19	5.19	195.00	116.77	1.00	1.00	3.04	93.74	62.50
Dsgn. L = 13.00 ft	3	0.054	0.049	6.34	-4.94	6.34	195.00	116.77	1.00	1.00	3.04	93.74	62.50
+D+0.750Lr, LL Comb Run (LL*)													
Dsgn. L = 8.00 ft	1	0.044	0.021		-5.19	5.19	195.00	116.77	1.00	1.00	1.32	93.74	62.50
Dsgn. L = 13.75 ft	2	0.044	0.038	1.49	-5.19	5.19	195.00	116.77	1.00	1.00	2.38	93.74	62.50
Dsgn. L = 13.00 ft	3	0.039	0.029	2.83	-4.52	4.52	195.00	116.77	1.00	1.00	1.83	93.74	62.50
+D+0.750Lr, LL Comb Run (LLL)													
Dsgn. L = 8.00 ft	1	0.044	0.021		-5.19	5.19	195.00	116.77	1.00	1.00	1.30	93.74	62.50
Dsgn. L = 13.75 ft	2	0.055	0.050	0.29	-6.37	6.37	195.00	116.77	1.00	1.00	3.15	93.74	62.50
Dsgn. L = 13.00 ft	3	0.055	0.050	5.74	-6.37	6.37	195.00	116.77	1.00	1.00	3.15	93.74	62.50
+D+0.750S													
Dsgn. L = 8.00 ft	1	0.393	0.184		-45.89	45.89	195.00	116.77	1.00	1.00	11.47	93.74	62.50
Dsgn. L = 13.75 ft	2	0.393	0.315	-0.00	-45.89	45.89	195.00	116.77	1.00	1.00	19.69	93.74	62.50
Dsgn. L = 13.00 ft	3	0.330	0.315	38.56	-35.84	38.56	195.00	116.77	1.00	1.00	19.69	93.74	62.50
+0.60D													
Dsgn. L = 8.00 ft	1	0.017	0.008		-1.96	1.96	195.00	116.77	1.00	1.00	0.49	93.74	62.50
Dsgn. L = 13.75 ft	2	0.018	0.017	0.14	-2.15	2.15	195.00	116.77	1.00	1.00	1.06	93.74	62.50
Dsgn. L = 13.00 ft	3	0.018	0.017	1.92	-2.15	2.15	195.00	116.77	1.00	1.00	1.06	93.74	62.50

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.6989	0.000		0.0000	0.000
	2	0.0000	0.000	+D+S	-0.1082	4.950
+D+S	3	0.2230	7.020		0.0000	4.950

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum		25.173	43.858	18.494
Overall MINimum		-0.179	-0.285	0.051
D Only		1.585	3.202	1.210
+D+Lr, LL Comb Run (**L)		1.406	5.131	2.580
+D+Lr, LL Comb Run (*L*)		2.081	4.832	1.064
+D+Lr, LL Comb Run (*LL)		1.902	6.761	2.434

Steel Beam

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Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : B4 - RIDGE OVER ENTRY

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
+D+Lr, LL Comb Run (L**)		2.459	2.918	1.260
+D+Lr, LL Comb Run (L*L)		2.280	4.846	2.631
+D+Lr, LL Comb Run (LL*)		2.955	4.548	1.114
+D+Lr, LL Comb Run (LLL)		2.776	6.476	2.485
+D+S		25.173	43.858	18.494
+D+0.750Lr, LL Comb Run (**L)		1.451	4.649	2.238
+D+0.750Lr, LL Comb Run (*L*)		1.957	4.425	1.100
+D+0.750Lr, LL Comb Run (*LL)		1.823	5.871	2.128
+D+0.750Lr, LL Comb Run (L**)		2.241	2.989	1.248
+D+0.750Lr, LL Comb Run (L*L)		2.106	4.435	2.276
+D+0.750Lr, LL Comb Run (LL*)		2.613	4.211	1.138
+D+0.750Lr, LL Comb Run (LLL)		2.478	5.658	2.166
+D+0.750S		19.276	33.694	14.173
+0.60D		0.951	1.921	0.726
Lr Only, LL Comb Run (**L)		-0.179	1.929	1.370
Lr Only, LL Comb Run (*L*)		0.496	1.630	-0.146
Lr Only, LL Comb Run (*LL)		0.317	3.559	1.224
Lr Only, LL Comb Run (L**)		0.874	-0.285	0.051
Lr Only, LL Comb Run (L*L)		0.695	1.644	1.421
Lr Only, LL Comb Run (LL*)		1.370	1.345	-0.096
Lr Only, LL Comb Run (LLL)		1.191	3.274	1.275
S Only		23.588	40.656	17.285

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Steel Beam

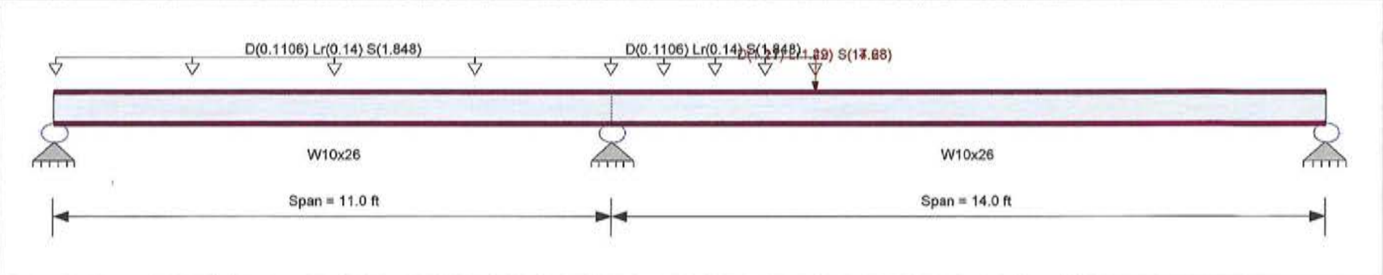
Description : B5 - RIDGE SUPPORT

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 7.0 ft, (ROOF)
Load for Span Number 2
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Extent = 0.0 -->> 4.0 ft, Tributary Width = 7.0 ft, (ROOF)
Point Load : D = 1.170, Lr = 1.290, S = 14.630 k @ 4.0 ft, (B3)
Point Load : D = 1.210, Lr = 1.420, S = 17.280 k @ 4.0 ft, (B4)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.858 : 1	Maximum Shear Stress Ratio =	0.675 : 1
Section used for this span	W10x26	Section used for this span	W10x26
Ma : Applied	67.037 k-ft	Va : Applied	36.178 k
Mn / Omega : Allowable	78.094 k-ft	Vn/Omega : Allowable	53.560 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	11.000ft	Location of maximum on span	11.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.336 in	Ratio =	499 >=240.
Max Upward Transient Deflection	-0.067 in	Ratio =	1,972 >=240.
Max Downward Total Deflection	0.364 in	Ratio =	461 >=180
Max Upward Total Deflection	-0.073 in	Ratio =	1815 >=180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 11.00 ft	1	0.063	0.049	0.34	-4.89	4.89	130.42	78.09	1.00	1.00	2.61	80.34	53.56
Dsgn. L = 14.00 ft	2	0.063	0.049	4.45	-4.89	4.89	130.42	78.09	1.00	1.00	2.61	80.34	53.56
+D+Lr													
Dsgn. L = 11.00 ft	1	0.128	0.101	0.68	-10.00	10.00	130.42	78.09	1.00	1.00	5.39	80.34	53.56
Dsgn. L = 14.00 ft	2	0.128	0.101	9.32	-10.00	10.00	130.42	78.09	1.00	1.00	5.39	80.34	53.56
+D+S													
Dsgn. L = 11.00 ft	1	0.858	0.675	5.86	-67.04	67.04	130.42	78.09	1.00	1.00	36.18	80.34	53.56
Dsgn. L = 14.00 ft	2	0.858	0.675	61.61	-67.04	67.04	130.42	78.09	1.00	1.00	36.18	80.34	53.56
+D+0.750Lr													
Dsgn. L = 11.00 ft	1	0.112	0.088	0.59	-8.73	8.73	130.42	78.09	1.00	1.00	4.70	80.34	53.56
Dsgn. L = 14.00 ft	2	0.112	0.088	8.10	-8.73	8.73	130.42	78.09	1.00	1.00	4.70	80.34	53.56
+D+0.750S													
Dsgn. L = 11.00 ft	1	0.659	0.519	4.48	-51.50	51.50	130.42	78.09	1.00	1.00	27.79	80.34	53.56
Dsgn. L = 14.00 ft	2	0.659	0.519	47.32	-51.50	51.50	130.42	78.09	1.00	1.00	27.79	80.34	53.56
+0.60D													
Dsgn. L = 11.00 ft	1	0.038	0.029	0.21	-2.94	2.94	130.42	78.09	1.00	1.00	1.57	80.34	53.56
Dsgn. L = 14.00 ft	2	0.038	0.029	2.67	-2.94	2.94	130.42	78.09	1.00	1.00	1.57	80.34	53.56

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Steel Beam

Description : B5 - RIDGE SUPPORT

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+S	-0.0727	7.920
+D+S	2	0.3642	6.552		0.0000	7.920

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	4.821	53.188	6.310
Overall MINimum	0.184	2.284	0.345
D Only	0.306	3.807	0.576
+D+Lr	0.612	7.822	1.065
+D+S	4.821	53.188	6.310
+D+0.750Lr	0.536	6.818	0.943
+D+0.750S	3.692	40.843	4.876
+0.60D	0.184	2.284	0.345
Lr Only	0.306	4.015	0.489
S Only	4.515	49.381	5.734

Steel Beam

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Lic. # : KW-06002886
Licensee : RICHMOND HOFFMAYER

Description : B6 - w/ SHEAR WALLS EACH END - STEEL

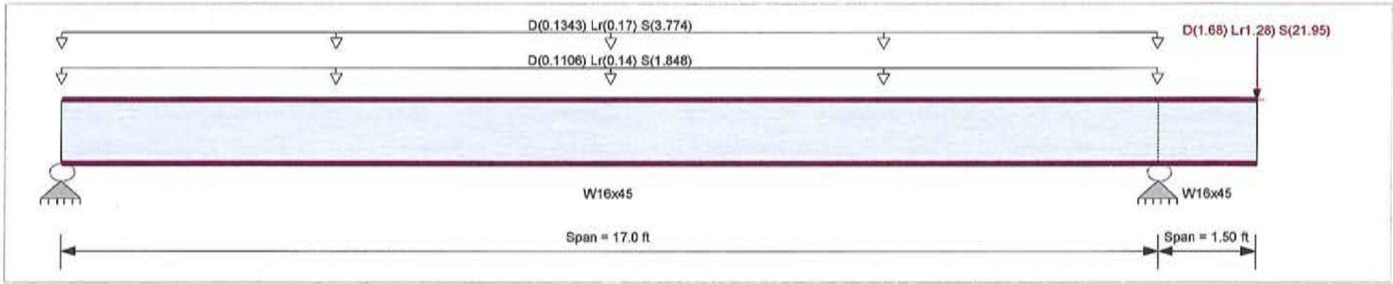
CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 7.0 ft, (ROOF)
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.4440 ksf, Tributary Width = 8.50 ft, (OVERHANG)
Load(s) for Span Number 2
Point Load : D = 1.680, Lr = 1.280, S = 21.950 k @ 1.50 ft, (B4)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.955 : 1	Maximum Shear Stress Ratio =	0.471 : 1
Section used for this span	W16x45	Section used for this span	W16x45
Ma : Applied	196.188 k-ft	Va : Applied	52.339 k
Mn / Omega : Allowable	205.339 k-ft	Vn/Omega : Allowable	111.090 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	8.160ft	Location of maximum on span	17.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.564 in Ratio =	361 >=240.
Max Upward Transient Deflection	-0.144 in Ratio =	249 >=240.
Max Downward Total Deflection	0.593 in Ratio =	344 >=180
Max Upward Total Deflection	-0.151 in Ratio =	238 >=180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 17.00 ft	1	0.045	0.024	9.23	-2.57	9.23	342.92	205.34	1.00	1.00	2.62	166.64	111.09
Dsgn. L = 1.50 ft	2	0.013	0.016		-2.57	2.57	342.92	205.34	1.00	1.00	1.75	166.64	111.09
+D+Lr													
Dsgn. L = 17.00 ft	1	0.095	0.048	19.48	-4.49	19.48	342.92	205.34	1.00	1.00	5.36	166.64	111.09
Dsgn. L = 1.50 ft	2	0.022	0.027		-4.49	4.49	342.92	205.34	1.00	1.00	3.03	166.64	111.09
+D+S													
Dsgn. L = 17.00 ft	1	0.955	0.471	196.19	-35.50	196.19	342.92	205.34	1.00	1.00	52.34	166.64	111.09
Dsgn. L = 1.50 ft	2	0.173	0.213		-35.50	35.50	342.92	205.34	1.00	1.00	23.70	166.64	111.09
+D+0.750Lr													
Dsgn. L = 17.00 ft	1	0.082	0.042	16.92	-4.01	16.92	342.92	205.34	1.00	1.00	4.68	166.64	111.09
Dsgn. L = 1.50 ft	2	0.020	0.024		-4.01	4.01	342.92	205.34	1.00	1.00	2.71	166.64	111.09
+D+0.750S													
Dsgn. L = 17.00 ft	1	0.728	0.359	149.45	-27.26	149.45	342.92	205.34	1.00	1.00	39.91	166.64	111.09
Dsgn. L = 1.50 ft	2	0.133	0.164		-27.26	27.26	342.92	205.34	1.00	1.00	18.21	166.64	111.09
+0.60D													
Dsgn. L = 17.00 ft	1	0.027	0.014	5.54	-1.54	5.54	342.92	205.34	1.00	1.00	1.57	166.64	111.09
Dsgn. L = 1.50 ft	2	0.008	0.009		-1.54	1.54	342.92	205.34	1.00	1.00	1.05	166.64	111.09

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Steel Beam

Description : B6 - w/ SHEAR WALLS EACH END - STEEL

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.5926	8.432		0.0000	0.000
	2	0.0000	8.432	+D+S	-0.1512	1.500

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	48.163	76.037	
Overall MINimum	1.388	2.618	
D Only	2.313	4.363	
+D+Lr	4.835	8.391	
+D+S	48.163	76.037	
+D+0.750Lr	4.204	7.384	
+D+0.750S	36.701	58.118	
+0.60D	1.388	2.618	
Lr Only	2.522	4.028	
S Only	45.850	71.674	

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Wood Beam

Description : B7 - BEARING HEADERS

CODE REFERENCES

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

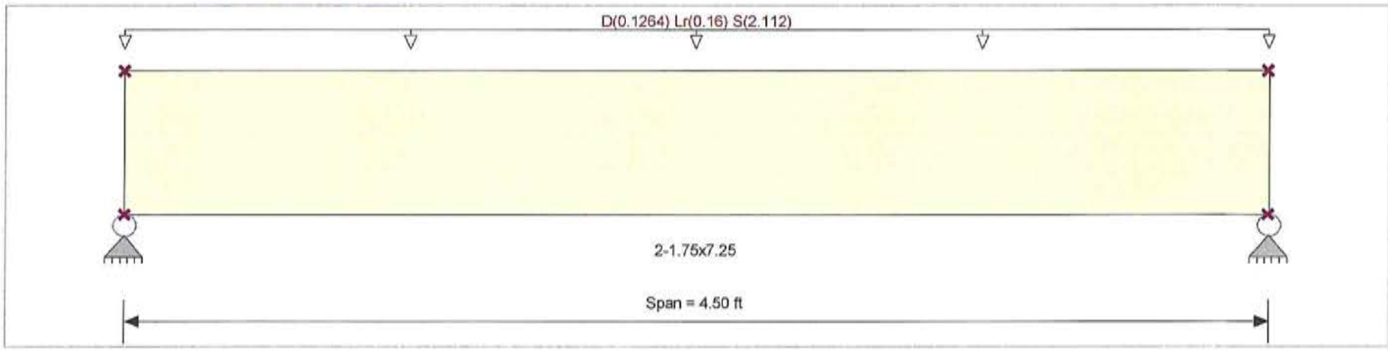
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

Beam Bracing : Completely Unbraced

Fb +	2,600.0 psi	*E : Modulus of Elasticity*	
Fb -	2,600.0 psi	Ebend- xx	1,900.0 ksi
Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Fc - Perp	750.0 psi		
Fv	285.0 psi		
Ft	1,555.0 psi	Density	42.0 pcf



Applied Loads

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

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 8.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.749 : 1	Maximum Shear Stress Ratio	=	0.670 : 1
Section used for this span		**2-1.75x7.25**	Section used for this span		**2-1.75x7.25**
fb : Actual	=	2,217.49psi	fv : Actual	=	219.49 psi
FB : Allowable	=	2,960.54psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	2.250ft	Location of maximum on span	=	3.909 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.093 in	Ratio =	581 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.098 in	Ratio =	548 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	5.036	5.036
Overall MINimum	0.171	0.171
D Only	0.284	0.284
+D+Lr	0.644	0.644
+D+S	5.036	5.036
+D+0.750Lr	0.554	0.554
+D+0.750S	3.848	3.848
+0.60D	0.171	0.171
Lr Only	0.360	0.360
S Only	4.752	4.752

Wood Beam

Description : B9 - GARAGE HEADER

CODE REFERENCES

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

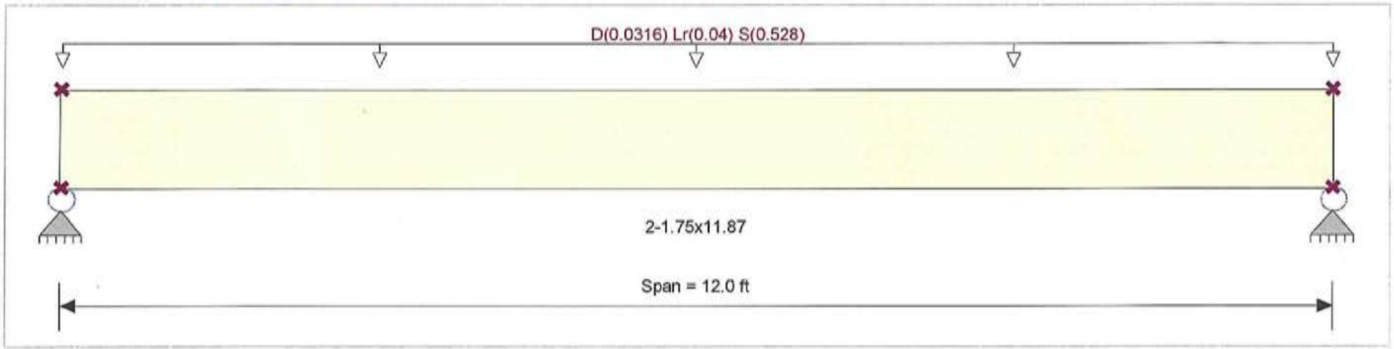
Material Properties

Analysis Method : Allowable Stress Design
Load Combination :ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

Beam Bracing : Completely Unbraced

Fb +	2,600.0 psi	*E : Modulus of Elasticity*	
Fb -	2,600.0 psi	Ebend- xx	1,900.0ksi
Fc - Prll	2,510.0 psi	Eminbend - xx	965.71ksi
Fc - Perp	750.0 psi		
Fv	285.0 psi		
Ft	1,555.0 psi	Density	42.0pcf



Applied Loads

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

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 2.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.532 1	Maximum Shear Stress Ratio	=	0.310 : 1
Section used for this span		2-1.75x11.87	Section used for this span		2-1.75x11.87
fb : Actual	=	1,469.43psi	fv : Actual	=	101.72 psi
FB : Allowable	=	2,760.63psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	6.000ft	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	0.267 in	Ratio =	539>=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0<240.0
Max Downward Total Deflection	0.283 in	Ratio =	508>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0<240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.358	3.358
Overall MINimum	0.114	0.114
D Only	0.190	0.190
+D+Lr	0.430	0.430
+D+S	3.358	3.358
+D+0.750Lr	0.370	0.370
+D+0.750S	2.566	2.566
+0.60D	0.114	0.114
Lr Only	0.240	0.240
S Only	3.168	3.168

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Wood Beam

Description : B8 - BEARING HEADERS

CODE REFERENCES

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

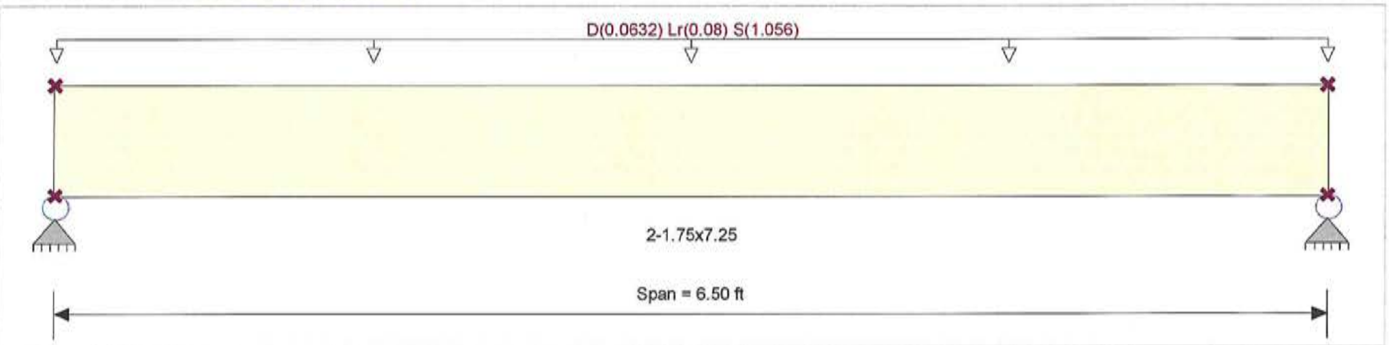
Material Properties

Analysis Method : Allowable Stress Design
Load Combination :ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

Beam Bracing : Completely Unbraced

Fb +	2,600.0 psi	*E : Modulus of Elasticity*	
Fb -	2,600.0 psi	Ebend- xx	1,900.0ksi
Fc - Prll	2,510.0 psi	Eminbend - xx	965.71ksi
Fc - Perp	750.0 psi		
Fv	285.0 psi		
Ft	1,555.0 psi	Density	42.0pcf



Applied Loads

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

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 4.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.785 1	Maximum Shear Stress Ratio	=	0.536 : 1
Section used for this span		2-1.75x7.25	Section used for this span		2-1.75x7.25
fb : Actual	=	2,313.30psi	fv : Actual	=	175.78 psi
FB : Allowable	=	2,947.43psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.250ft	Location of maximum on span	=	5.907 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.202 in	Ratio =	386 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.214 in	Ratio =	364 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.637	3.637
Overall MINimum	0.123	0.123
D Only	0.205	0.205
+D+Lr	0.465	0.465
+D+S	3.637	3.637
+D+0.750Lr	0.400	0.400
+D+0.750S	2.779	2.779
+0.60D	0.123	0.123
Lr Only	0.260	0.260
S Only	3.432	3.432

Wood Beam

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Description : B10 - ENTRY HEADER

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

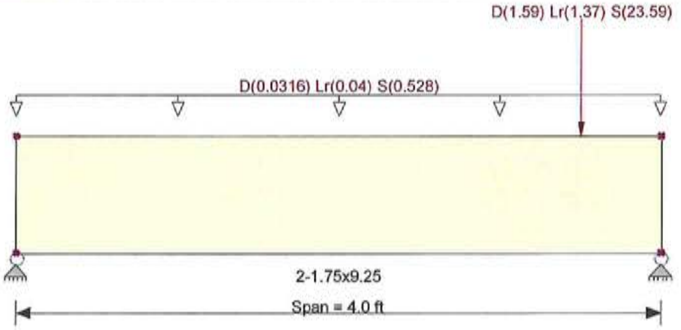
Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

Beam Bracing : Completely Unbraced

Fb +	2,600.0 psi
Fb -	2,600.0 psi
Fc - Prll	2,510.0 psi
Fc - Perp	750.0 psi
Fv	285.0 psi
Ft	1,555.0 psi

E : Modulus of Elasticity

Ebend- xx	1,900.0 ksi
Eminbend - xx	965.71 ksi
Density	42.0 pcf



Applied Loads

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

Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 2.0 ft, (ROOF)
Point Load : D = 1.590, Lr = 1.370, S = 23.590 k @ 3.50 ft, (B4)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.934 : 1	Maximum Shear Stress Ratio	=	0.543 : 1
Section used for this span		2-1.75x9.25	Section used for this span		2-1.75x9.25
fb : Actual	=	2,760.23psi	fv : Actual	=	178.00 psi
FB : Allowable	=	2,955.25psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.489ft	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	0.054 in	Ratio =	895 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.057 in	Ratio =	839 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.267	23.152
Overall MINimum	0.157	0.873
D Only	0.262	1.454
+D+Lr	0.513	2.733
+D+S	4.267	23.152
+D+0.750Lr	0.450	2.414
+D+0.750S	3.266	17.727
+0.60D	0.157	0.873
Lr Only	0.251	1.279
S Only	4.005	21.697

Table 3: Maximum Allowable Uniform Transverse Load (psf) – Type L Panels[1,3]

Panel Core Thickness (in)	Deflection Limit[2]	Panel Span (ft)									
		4[4]	8	10	12	14	16	18	20	22	24
3.5	L/360	103	45	33	24	18	11				
	L/240	225	68	47	34	26	17				
	L/180	297*	91	61	45	34	23				
5.5	L/360	307*	129	57	42	34	25	20	15		
	L/240	307*	182*	87	61	49	37	30	22		
	L/180	307*	182*	112*	80	65	49	39	29		
7.25	L/360	253	171	82	66	54	41	32	23		
	L/240	288*	188*	128	100	81	61	48	35		
	L/180	288*	188*	133*	117*	105	80	63	45		
9.25	L/360	286	188*	117	101	80	58	47	36	32	27
	L/240	326*	188*	147*	134*	120	90	71	52	47	41
	L/180	326*	188*	147*	134*	121	108*	93	68	61	53
11.25	L/360	327*	188*	167*	141	116	91	75	58	47	36
	L/240	327*	188*	167*	153*	132	110*	97	83*	69	53
	L/180	327*	188*	167*	153*	132	110*	97	83*	83	70

[1] Table values assume a simply supported panel with 1.5 in. of continuous bearing on facing at supports. Permanent loads, such as dead load, shall not exceed 0.25 times the tabulated load. Splines consist of #2 or better, Hem-Fir, 1.5 in. wide with a depth equal to the core thickness, spaced to provide not less than two members for every 48 in. of panel width.

[2] Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code.

[3] Tabulated values for 8 ft walls apply to panels constructed with the OSB strength axis oriented either parallel or perpendicular to supports. Tabulated values for other lengths are based on the strong-axis of the facing material oriented parallel to the span direction.

[4] Panels spanning 4 ft shall be a minimum of 8 ft long spanning a minimum of two 4 ft spans. No single span condition is allowed.

*An asterisk (*) indicates the value shown is governed by the average peak load divided by 3.

Table 4: Maximum Allowable Uniform Axial Load (plf) – Type S Panels[1,2,3,4]

Panel Core Thickness (in)	Panel Span (ft)					
	8	10	12	16	20	24
3.5	3500	2553	2453	2117		
5.5	4250	4043	3373	3923	2817	2183
7.25	4917	4327	4473	4197	3497	3067
9.25	4600	4414	4228	4417	3389	3248
11.25	3889	3959	4028	4408	3837*	3333

[1] Splines consist of OSB surface splines not less than 7/16 in. thick inserted below the facing on each side of the panel. Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.

[2] Uniform Axial loads may be applied in accordance with Section 5.5.1. Concentrated point loads shall be addressed in accordance with Section 5.5.2 and Table 6.

[3] Both facings must bear on the supporting foundation or structure.

[4] Tabulated values for 8 ft walls apply to panels constructed with the OSB strength axis oriented either parallel or perpendicular to supports.

* Limited by 1/8 in. deflection (compression)

MAX LOAD
B1B
$W = (16.8 + 20 + 264)\,15/2$
$= 2249$ PLF
OK

Table 5: Maximum Allowable Uniform Axial Loads (plf) – Type L Panels[1,2,3,4]

Panel Core Thickness (in)	Panel Span (ft)					
	8	10	12	16	20	24
3.5	4723	3903	3273	2623		
5.5	5850	5890	4277	4310	2933	2837
7.25	6807	6110	5557	5180	4837	4083
9.25	5473	5709	5946	5948	4729*	4250
11.25	5667	5474	5281	5775*	4729*	4223

[1] Splines consist of #2 or better, Hem-Fir, 1.5 in. wide with a depth equal to the core thickness, spaced to provide not less than two members for every 48 in. of panel width. Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.

[2] Axial loads shall be applied concentrically to the top of the panel through repetitive members spaced not more than 24 in. on center. Such members shall be fastened to a rim board or similar member to distribute along the top of the SIP panel.

[3] Both facings must bear on the supporting foundation or structure.

[4] Tabulated values for 8 ft walls apply to panels constructed with the OSB strength axis oriented either parallel or perpendicular to supports.

* Limited by 1/8 in. deflection (compression)

SINGLE RR DBL RR

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Table 6: Maximum Allowable Axial Compression Point Loads (lbs) – Type S Panels[1,2,3,4]

Top Plate Configuration	1.5" Minimum Bearing Width	3" Minimum Bearing Width
Single 2x4 #2 or Better Hem-Fir Plate	2040	2450
Single 2x4 #2 or Better Hem-Fir Plate with 1-1/8 in. wide, 1.3E Rim Board Cap Plate	4030	4678

[1] Top plate secured to facings as required in Section 6.3
[2] Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.
[3] Concentrated loads shall be applied concentrically to the top of the panel.
[4] Tabulated values are based on the strong-axis of the facing material oriented parallel to the span direction.

$RR_{MAX} = 2249 \times \frac{16}{12} = 2998$ lb

CAP PLATE REQUIRED

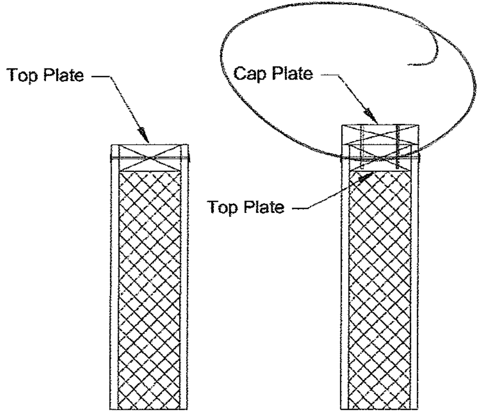

Figure 3: Top Plate Configurations

Table 7: Maximum Allowable Uniform SIP Header Vertical Loads (plf)
3-1/2 in. through 11-1/4 in. Core Thickness[1,2]

Continuous over opening

Header Depth[3] (in)	Deflection Limit[4]	Header Span (ft)			
		4	6	8	10
12	L/480	740	384	228	142
	L/360	740	384	229	142
	L/240	740	384	229	142
18	L/480	798	574	385	311
	L/360	798	574	385	311
	L/240	798	574	385	311
24	L/480	886	629	429	361
	L/360	886	629	429	361
	L/240	886	629	429	361

[1] Vertical loads only. Lateral loads shall be transferred to the edges of the openings through continuous plate(s) designed in accordance with accepted engineering practice. Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.
[2] Tabulated values are based on the strong-axis of the facing material oriented perpendicular to the direction of header span.
[3] Minimum depth of facing above opening.
[4] Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code.

Table 8: Maximum Allowable Uniform Header Loads (plf)
(Panel Splice a minimum of 6 in. from edge of opening) 3-1/2 in. through 11-1/4 in. Core Thickness[1,2]

SIPs wall panel spline minimum 6" from edge of opening

Header Depth[3] (in)	Deflection Limit[4]	Header Span (ft)			
		4	6	8	10
12	L/480	345	243	156	99
	L/360	450	295	190	125
	L/240	630	382	236	153
18	L/480	705	388	254	235
	L/360	750	482	302	281
	L/240	750	482	302	281
24	L/480	698	556	368	350
	L/360	896	556	368	350
	L/240	896	556	368	350

[1] Vertical loads only. Lateral loads shall be transferred to the edges of the openings through continuous plate(s) designed in accordance with accepted engineering practice. Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.
[2] Tabulated values are based on the strong-axis of the facing material oriented perpendicular to the direction of header span.
[3] Minimum depth of facing above opening.
[4] Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code.

Wood Column

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Description : ROOF SUPPORT POSTS - EXTERIOR WALLS - B5 ENDS

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method :	Allowable Stress Design
End Fixities	Top & Bottom Pinned
Overall Column Height	10.50 ft

(Used for non-slender calculations)

Wood Species	Spruce - Pine - Fir		
Wood Grade	No.1		
Fb +	850.0 psi	Fv	125.0 psi
Fb -	850.0 psi	Ft	550.0 psi
Fc - Prll	700.0 psi	Density	26.210 pcf
Fc - Perp	425.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,300.0	1,300.0	1,300.0 ksi
Minimum	470.0	470.0	

Wood Section Name	4x8
Wood Grading/Manuf.	Graded Lumber
Wood Member Type	Sawn

Exact Width	3.50 in
Exact Depth	7.250 in
Area	25.375 in^2
Ix	111.148 in^4
Iy	25.904 in^4

Allow Stress Modification Factors	
Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.050
Cf or Cv for Tension	1.20
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 10 ft, K = 1.0

Applied Loads

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

Column self weight included : 48.495 lbs * Dead Load Factor
AXIAL LOADS . . .
B5 MAX: Axial Load at 10.50 ft, D = 0.5756, Lr = 0.4894, S = 5.734 k
BENDING LOADS . . .
WIND: Lat. Uniform Load creating Mx-x, W = 0.2050 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.3827 : 1
	Load Combination	+D+0.60W
	Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
	Location of max.above base	5.285 ft
	At maximum location values are . . .	
	Applied Axial	0.6241 k
	Applied Mx	1.695 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	881.75 psi
PASS	Maximum Shear Stress Ratio =	0.1909 : 1
	Load Combination	+D+0.60W
	Location of max.above base	0.0 ft
	Applied Design Shear	38.172 psi
	Allowable Shear	200.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	1.076 k	Bottom along Y-Y	1.076 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.3922 in at 5.285 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

			Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
Load Combination	C_D	C_P	Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.877	0.04238	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+Lr	1.250	0.815	0.05858	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+S	1.150	0.834	0.3556	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.750Lr	1.250	0.815	0.05215	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.750S	1.150	0.834	0.2755	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.60W	1.600	0.750	0.3827	PASS	5.285 ft	0.1909	PASS	0.0 ft
+D+0.750Lr+0.450W	1.600	0.750	0.2914	PASS	5.215 ft	0.1431	PASS	0.0 ft
+D+0.750S+0.450W	1.600	0.750	0.3748	PASS	5.285 ft	0.1431	PASS	0.0 ft
+0.60D+0.60W	1.600	0.750	0.3795	PASS	5.285 ft	0.1909	PASS	0.0 ft

Wood Column

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Description : ROOF SUPPORT POSTS - EXTERIOR WALLS - B5 ENDS

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D	1.600	0.750	0.01674	PASS	0.0 ft	0.0	PASS	10.50 ft

Maximum Reactions

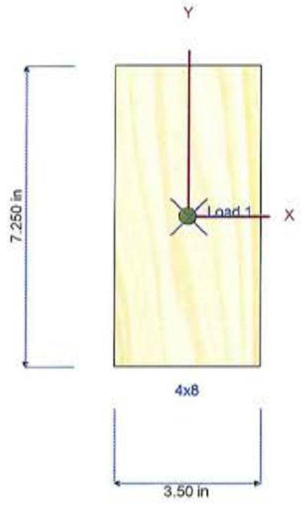
Note: Only non-zero reactions are listed.

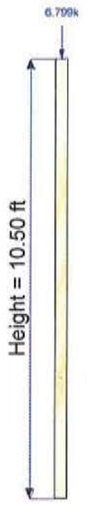
Load Combination	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only					0.624				
+D+Lr					1.113				
+D+S					6.358				
+D+0.750Lr					0.991				
+D+0.750S					4.925				
+D+0.60W			0.646	0.646	0.624				
+D+0.750Lr+0.450W			0.484	0.484	0.991				
+D+0.750S+0.450W			0.484	0.484	4.925				
+0.60D+0.60W			0.646	0.646	0.374				
+0.60D					0.374				
Lr Only					0.489				
S Only					5.734				
W Only			1.076	1.076					

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	0.235 in	5.285 ft
+D+0.750Lr+0.450W	0.0000 in	0.000 ft	0.176 in	5.285 ft
+D+0.750S+0.450W	0.0000 in	0.000 ft	0.176 in	5.285 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.235 in	5.285 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.392 in	5.285 ft

Sketches





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Wood Column

Description : ROOF SUPPORT POSTS - EXTERIOR WALLS - B10 ENDS

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 10.50 ft
(Used for non-slender calculations)

Wood Species	Spruce - Pine - Fir		
Wood Grade	No.1		
Fb +	850.0 psi	Fv	125.0 psi
Fb -	850.0 psi	Ft	550.0 psi
Fc - Prll	700.0 psi	Density	26.210 pcf
Fc - Perp	425.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,300.0	1,300.0	1,300.0 ksi
Minimum	470.0	470.0	

Wood Section Name 6x8
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	5.50 in
Exact Depth	7.50 in
Area	41.250 in^2
Ix	193.359 in^4
Iy	103.984 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0
Cf or Cv for Compression	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 10 ft, K = 1.0

Applied Loads

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

Column self weight included : 78.835 lbs * Dead Load Factor
AXIAL LOADS . . .
B10 MAX: Axial Load at 10.50 ft, D = 1.590, Lr = 1.370, S = 23.590 k
BENDING LOADS . . .
WIND: Lat. Uniform Load creating Mx-x, W = 0.2050 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.8885 : 1
	Load Combination	+D+S
	Governing NDS Forumla	Comp Only, fc/Fc'
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Applied Axial	25.259 k
	Applied Mx	0.0 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	689.16 psi
PASS	Maximum Shear Stress Ratio =	0.1174 : 1
	Load Combination	+D+0.60W
	Location of max.above base	0.0 ft
	Applied Design Shear	23.482 psi
	Allowable Shear	200.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	1.076 k	Bottom along Y-Y	1.076 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.2254 in at 5.285 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

			Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
Load Combination	C_D	C_P	Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.894	0.07186	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+Lr	1.250	0.840	0.1002	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+S	1.150	0.856	0.8885	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.750Lr	1.250	0.840	0.08890	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.750S	1.150	0.856	0.6811	PASS	0.0 ft	0.0	PASS	10.50 ft
+D+0.60W	1.600	0.783	0.3002	PASS	5.285 ft	0.1174	PASS	0.0 ft
+D+0.750Lr+0.450W	1.600	0.783	0.2329	PASS	5.285 ft	0.08806	PASS	0.0 ft
+D+0.750S+0.450W	1.600	0.783	0.6023	PASS	5.215 ft	0.08806	PASS	0.0 ft
+0.60D+0.60W	1.600	0.783	0.2956	PASS	5.285 ft	0.1174	PASS	0.0 ft

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Wood Column

Description : ROOF SUPPORT POSTS - EXTERIOR WALLS - B10 ENDS

Load Combination Results

			Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
Load Combination	C_D	C_P	Stress Ratio	Status	Location	Stress Ratio	Status	Location
+0.60D	1.600	0.783	0.02768	PASS	0.0 ft	0.0	PASS	10.50 ft

Maximum Reactions

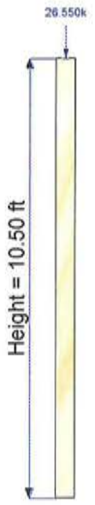
Note: Only non-zero reactions are listed.

	X-X Axis Reaction	k	Y-Y Axis Reaction		Axial Reaction	My - End Moments	k-ft	Mx - End Moments	
Load Combination	@ Base	@ Top	@ Base	@ Top	@ Base	@ Base	@ Top	@ Base	@ Top
D Only					1.669				
+D+Lr					3.039				
+D+S					25.259				
+D+0.750Lr					2.696				
+D+0.750S					19.361				
+D+0.60W			0.646	0.646	1.669				
+D+0.750Lr+0.450W			0.484	0.484	2.696				
+D+0.750S+0.450W			0.484	0.484	19.361				
+0.60D+0.60W			0.646	0.646	1.001				
+0.60D					1.001				
Lr Only					1.370				
S Only					23.590				
W Only			1.076	1.076					

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	0.135 in	5.285 ft
+D+0.750Lr+0.450W	0.0000 in	0.000 ft	0.101 in	5.285 ft
+D+0.750S+0.450W	0.0000 in	0.000 ft	0.101 in	5.285 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.135 in	5.285 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.225 in	5.285 ft

Sketches



RICHMOND HOFFMAYER INC.
STRUCTURAL ENGINEERING

Project Title: **VILLAGE NEST**
Engineer: JJH
Project Descr: MULTI UNIT PROJECT

Project ID: 2017-0610

162

Printed: 7 JUL 2017, 10:36AM

Steel Column

File = d:\ENERCALC Projects\2017-0610.ec6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06002886
Licensee : RICHMOND HOFFMAYER

Description : RIDGE SUPPORT - B4 AND B5

Code References

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Steel Section Name : **Pipe4 Std**
Analysis Method : Allowable Strength
Steel Stress Grade A-36, Carbon Steel, Fy = 36 ksi
Fy : Steel Yield 36.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 11.0 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Fully braced against buckling along X-X Axis
Y-Y (depth) axis :
Fully braced against buckling along Y-Y Axis

Applied Loads

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

Column self weight included : 119.439 lbs * Dead Load Factor
AXIAL LOADS . . .
MAX ROOF: Axial Load at 11.0 ft, D = 3.807, LR = 4.015, S = 49.381 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = 0.8326 : 1
Load Combination +D+S
Location of max.above base 0.0 ft
At maximum location values are . . .
Pa : Axial 53.307 k
Pn / Omega : Allowable 64.024 k
Ma-x : Applied 0.0 k-ft
Mn-x / Omega : Allowable 7.275 k-ft
Ma-y : Applied 0.0 k-ft
Mn-y / Omega : Allowable 7.275 k-ft

PASS Maximum Shear Stress Ratio = 0.0 : 1
Load Combination
Location of max.above base 0.0 ft
At maximum location values are . . .
Va : Applied 0.0 k
Vn / Omega : Allowable 0.0 k

Maximum Load Reactions . .
Top along X-X 0.0 k
Bottom along X-X 0.0 k
Top along Y-Y 0.0 k
Bottom along Y-Y 0.0 k

Maximum Load Deflections . . .
Along Y-Y 0.0 in at 0.0ft above base
for load combination :
Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios Stress Ratio	Status	Location	Maximum Shear Ratios Stress Ratio	Status	Location
D Only	0.061	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+Lr	0.124	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+S	0.833	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750Lr	0.108	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750S	0.640	PASS	0.00 ft	0.000	PASS	0.00 ft
+0.60D	0.037	PASS	0.00 ft	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
D Only	3.926								
+D+Lr	7.941								
+D+S	53.307								
+D+0.750Lr	6.938								
+D+0.750S	40.962								
+0.60D	2.356								
Lr Only	4.015								
S Only	49.381								

Steel Column

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Description : RIDGE SUPPORT - B4 AND B5

Extreme Reactions

		Axial Reaction	X-X Axis Reaction k		Y-Y Axis Reaction		Mx - End Moments k-ft		My - End Moments	
Item	Extreme Value	@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
Axial @ Base	Maximum	53.307								
"	Minimum	2.356								
Reaction, X-X Axis Base	Maximum	3.926								
"	Minimum	3.926								
Reaction, Y-Y Axis Base	Maximum	3.926								
"	Minimum	3.926								
Reaction, X-X Axis Top	Maximum	3.926								
"	Minimum	3.926								
Reaction, Y-Y Axis Top	Maximum	3.926								
"	Minimum	3.926								
Moment, X-X Axis Base	Maximum	3.926								
"	Minimum	3.926								
Moment, Y-Y Axis Base	Maximum	3.926								
"	Minimum	3.926								
Moment, X-X Axis Top	Maximum	3.926								
"	Minimum	3.926								
Moment, Y-Y Axis Top	Maximum	3.926								
"	Minimum	3.926								

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
Lr Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
S Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Steel Section Properties : Pipe4 Std

Depth	=	4.500 in	I xx	=	6.82 in^4	J	=	13.600 in^4
			S xx	=	3.03 in^3			
Diameter	=	4.500 in	R xx	=	1.510 in			
Wall Thick	=	0.237 in	Zx	=	4.050 in^3			
Area	=	2.970 in^2	I yy	=	6.820 in^4			
Weight	=	10.858 plf	S yy	=	3.030 in^3			
			R yy	=	1.510 in			

Ycg = 0.000 in

Steel Column

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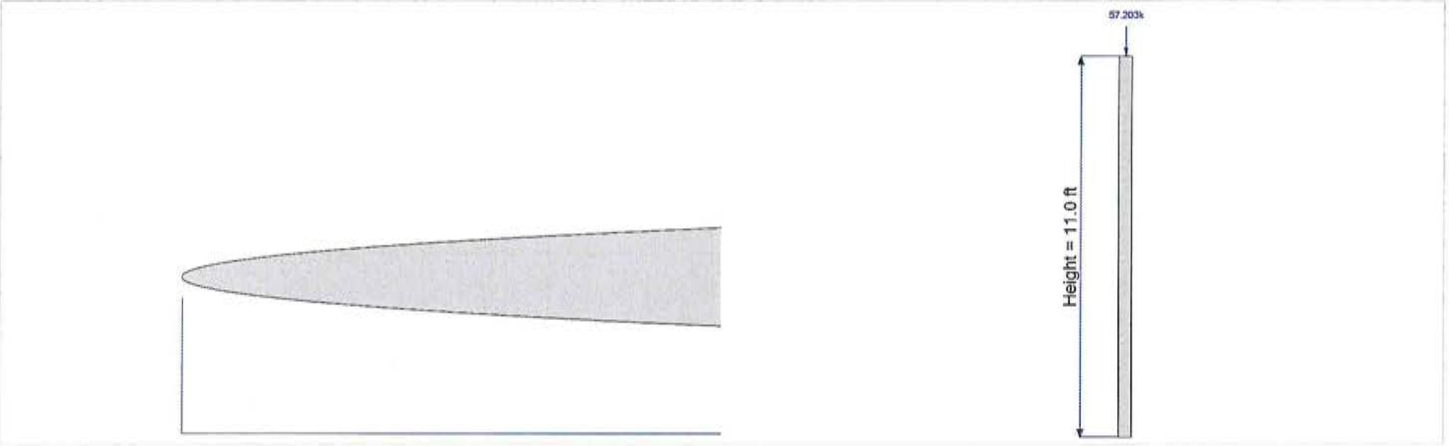
Description : RIDGE SUPPORT - B4 AND B5



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Description : B6 SUPPORT - MAX AT B3 SUPPORT

Code References

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Steel Section Name :	HSS6x6x3/8
Analysis Method :	Allowable Strength
Steel Stress Grade	
Fy : Steel Yield	46.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height	10.0 ft
Top & Bottom Fixity	Top & Bottom Pinned

Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 10.0 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for Y-Y Axis buckling = 10.0 ft, K = 2.1

Applied Loads

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

Column self weight included : 274.80 lbs * Dead Load Factor
AXIAL LOADS . . .
B6: Axial Load at 10.0 ft, Xecc = 0.750 in, D = 4.360, LR = 4.030, S = 71.660 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.9468 : 1
Load Combination	+D+S
Location of max.above base	9.933 ft
At maximum location values are . . .	
Pa : Axial	76.295 k
Pn / Omega : Allowable	91.797 k
Ma-x : Applied	0.0 k-ft
Mn-x / Omega : Allowable	36.267 k-ft
Ma-y : Applied	-4.719 k-ft
Mn-y / Omega : Allowable	36.267 k-ft
PASS Maximum Shear Stress Ratio =	0.008316 : 1
Load Combination	+D+S
Location of max.above base	0.0 ft
At maximum location values are . . .	
Va : Applied	0.4751 k
Vn / Omega : Allowable	57.137 k

Maximum Load Reactions . .

Top along X-X	0.4751 k
Bottom along X-X	0.4751 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum Load Deflections . . .

Along Y-Y 0.0 in at 0.0ft above base
for load combination :
Along X-X -0.04638 in at 5.839ft above base
for load combination : +D+S

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
	Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.050	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+Lr	0.094	PASS	0.00 ft	0.001	PASS	0.00 ft
+D+S	0.947	PASS	9.93 ft	0.008	PASS	0.00 ft
+D+0.750Lr	0.083	PASS	0.00 ft	0.001	PASS	0.00 ft
+D+0.750S	0.724	PASS	9.93 ft	0.006	PASS	0.00 ft
+0.60D	0.030	PASS	0.00 ft	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
D Only	4.635	0.027	0.027						
+D+Lr	8.665	0.052	0.052						
+D+S	76.295	0.475	0.475						
+D+0.750Lr	7.657	0.046	0.046						
+D+0.750S	58.380	0.363	0.363						
+0.60D	2.781	0.016	0.016						
Lr Only	4.030	0.025	0.025						
S Only	71.660	0.448	0.448						

Steel Column

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Description : B6 SUPPORT - MAX AT B3 SUPPORT

Extreme Reactions

		Axial Reaction	X-X Axis Reaction	k	Y-Y Axis Reaction		Mx - End Moments	k-ft	My - End Moments	
Item	Extreme Value	@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
Axial @ Base	Maximum	76.295	0.475	0.475						
"	Minimum	2.781	0.016	0.016						
Reaction, X-X Axis Base	Maximum	76.295	0.475	0.475						
"	Minimum	2.781	0.016	0.016						
Reaction, Y-Y Axis Base	Maximum	4.635	0.027	0.027						
"	Minimum	4.635	0.027	0.027						
Reaction, X-X Axis Top	Maximum	76.295	0.475	0.475						
"	Minimum	2.781	0.016	0.016						
Reaction, Y-Y Axis Top	Maximum	71.660	0.448	0.448						
"	Minimum	4.635	0.027	0.027						
Moment, X-X Axis Base	Maximum	4.635		0.027						
"	Minimum	4.635		0.027						
Moment, Y-Y Axis Base	Maximum	4.635	0.027	0.027						
"	Minimum	4.635	0.027	0.027						
Moment, X-X Axis Top	Maximum	4.635	0.027	0.027						
"	Minimum	4.635	0.027	0.027						
Moment, Y-Y Axis Top	Maximum	4.635	0.027	0.027						
"	Minimum	4.635	0.027	0.027						

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	-0.0027	in	5.839	ft	0.000	in	0.000	ft
+D+Lr	-0.0051	in	5.839	ft	0.000	in	0.000	ft
+D+S	-0.0464	in	5.839	ft	0.000	in	0.000	ft
+D+0.750Lr	-0.0045	in	5.839	ft	0.000	in	0.000	ft
+D+0.750S	-0.0354	in	5.839	ft	0.000	in	0.000	ft
+0.60D	-0.0016	in	5.839	ft	0.000	in	0.000	ft
Lr Only	-0.0025	in	5.839	ft	0.000	in	0.000	ft
S Only	-0.0437	in	5.839	ft	0.000	in	0.000	ft

Steel Section Properties : HSS6x6x3/8

Depth	=	6.000 in	I xx	=	39.50 in^4	J	=	64.600 in^4
Design Thick	=	0.349 in	S xx	=	13.20 in^3			
Width	=	6.000 in	R xx	=	2.280 in			
Wall Thick	=	0.375 in	Zx	=	15.800 in^3			
Area	=	7.580 in^2	I yy	=	39.500 in^4	C	=	22.100 in^3
Weight	=	27.480 plf	S yy	=	13.200 in^3			
			R yy	=	2.280 in			

Ycg = 0.000 in

Steel Column

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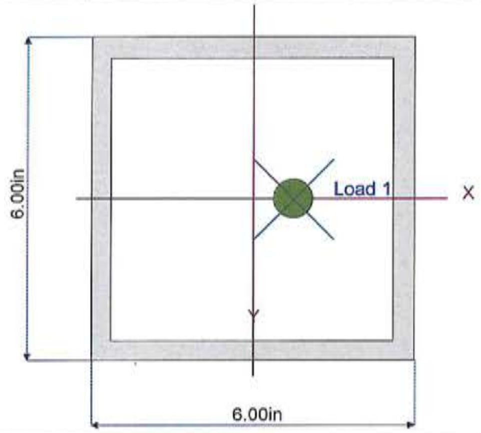
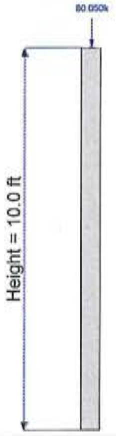
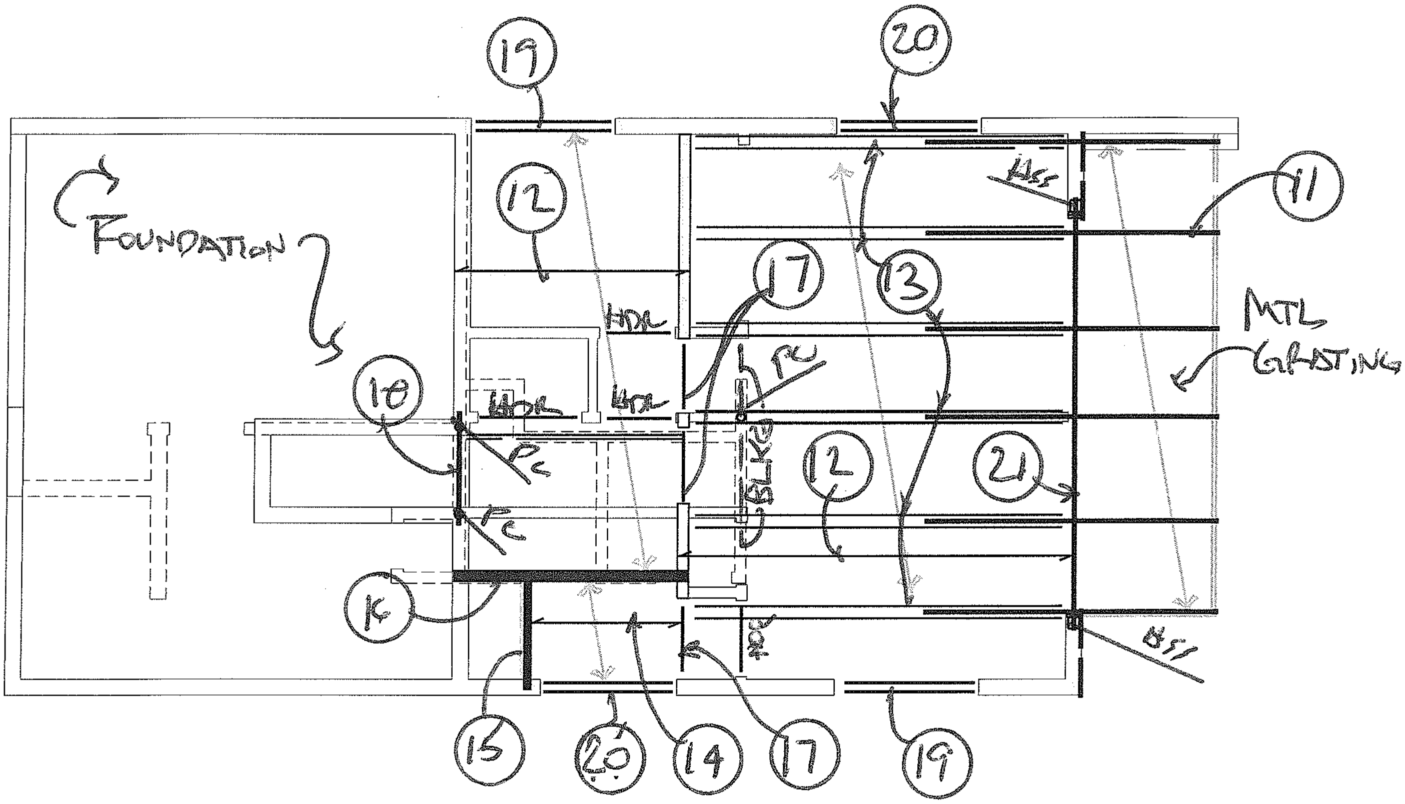
Description : B6 SUPPORT - MAX AT B3 SUPPORT





PROJECT FELDSPAR LOWER JOB 2017-0610
CLIENT VILLAGE NEST
ADDRESS EDEN, UTAH
DESIGN HUUM
ENGINEER J.H. DATE 6/16/2017
RICHMOND PE 49628 HOFFMAYER SE 3935 SHEET 162 OF


19
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FOUNDATION
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2ND FLOOR FRAMING PLAN
FELDSPAR – UPPER
N.T.S.

○ : BEAM #

Wood Beam

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ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : B14 - SHORT FLOOR JOISTS

CODE REFERENCES

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

Material Properties

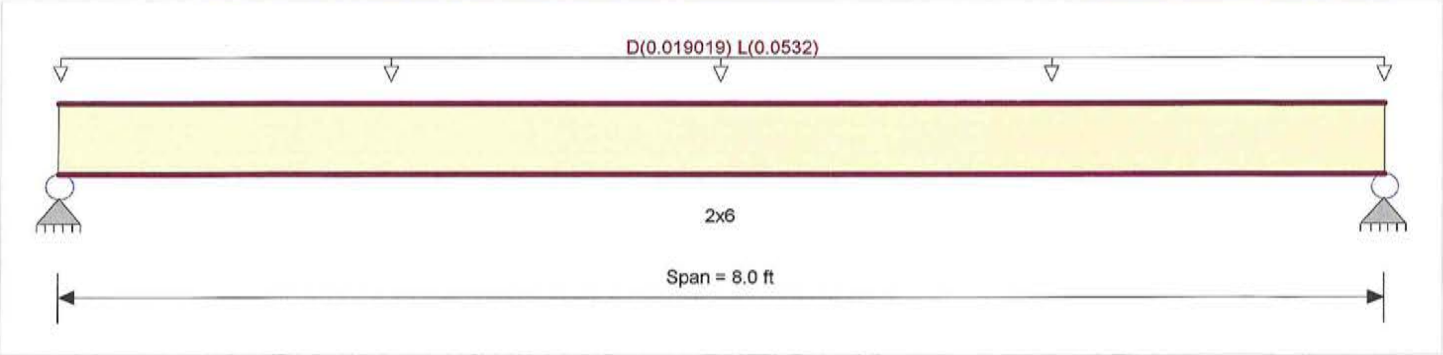
Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Spruce - Pine - Fir
Wood Grade : No. 1/No. 2

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

Fb +	875.0 psi	*E : Modulus of Elasticity*	
Fb -	875.0 psi	Ebend- xx	1,400.0ksi
Fc - Prll	1,150.0 psi	Eminbend - xx	510.0ksi
Fc - Perp	425.0 psi		
Fv	135.0 psi		
Ft	450.0 psi	Density	26.210pcf

Repetitive Member Stress Increase



Applied Loads

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

Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.701: 1	Maximum Shear Stress Ratio	=	0.346 : 1
Section used for this span		2x6	Section used for this span		2x6
fb : Actual	=	916.76psi	fv : Actual	=	46.77 psi
FB : Allowable	=	1,308.13psi	Fv : Allowable	=	135.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	4.000ft	Location of maximum on span	=	7.562ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.169 in	Ratio =	566>=360.
Max Upward Transient Deflection	0.000 in	Ratio =	0<360.0
Max Downward Total Deflection	0.230 in	Ratio =	417>=240.
Max Upward Total Deflection	0.000 in	Ratio =	0<240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.289	0.289
Overall MINimum	0.046	0.046
D Only	0.076	0.076
+D+L	0.289	0.289
+D+0.750L	0.236	0.236
+0.60D	0.046	0.046
L Only	0.213	0.213

Wood Beam

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : B15 - FLOOR SUPPORT

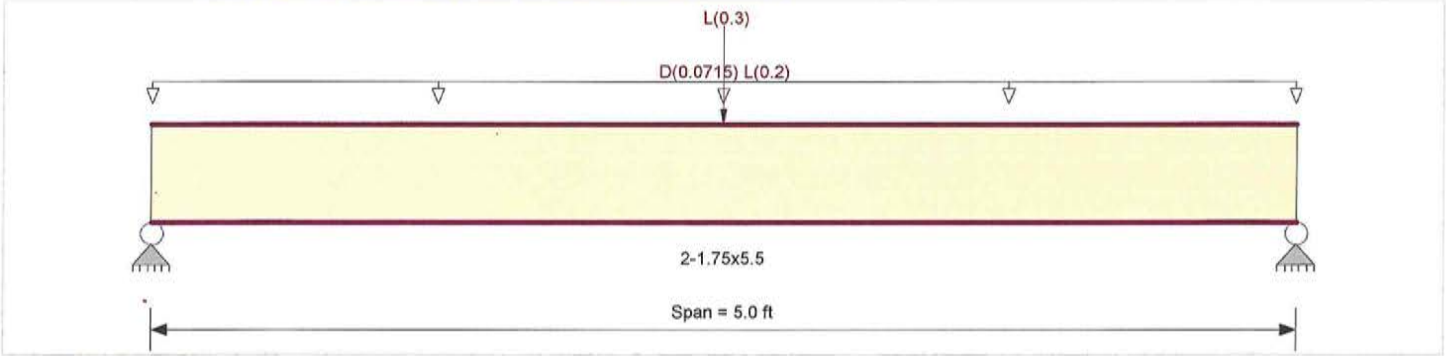
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,600.0 psi	*E : Modulus of Elasticity*	
Load Combination	ASCE 7-10	Fb -	2,600.0 psi	Ebend- xx	1,900.0 ksi
		Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Wood Species	: Trus Joist	Fc - Perp	750.0 psi		
Wood Grade	: MicroLam LVL 1.9 E	Fv	285.0 psi		
		Ft	1,555.0 psi	Density	42.0 pcf

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



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 5.0 ft, (FLOOR)
Point Load : L = 0.30 k @ 2.50 ft, (STAIR POINT LOAD)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.325 1	Maximum Shear Stress Ratio	=	0.196 : 1
Section used for this span		**2-1.75x5.5**	Section used for this span		**2-1.75x5.5**
fb : Actual	=	843.93psi	fv : Actual	=	55.82 psi
FB : Allowable	=	2,600.00psi	Fv : Allowable	=	285.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.500ft	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	0.045 in	Ratio = 1321 >=360
Max Upward Transient Deflection	0.000 in	Ratio = 0 <360
Max Downward Total Deflection	0.057 in	Ratio = 1048 >=240.
Max Upward Total Deflection	0.000 in	Ratio = 0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.843	0.843
Overall MINimum	0.116	0.116
D Only	0.193	0.193
+D+L	0.843	0.843
+D+0.750L	0.680	0.680
+0.60D	0.116	0.116
L Only	0.650	0.650

PROJECT VILLAGE NEST JOB 2017-0610
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RICHMOND PE 44628 HOFFMAYER SE 3455 SHEET 169 OF

METAL GRATING:

D = 12 PSF
L = 60 PSF
S = 264 PSF – (OPENED GRATING – DON'T DBL SNOW)

LOAD COMBINATIONS:

D + L = 72 PSF
D + S = 276 PSF ← GOVERNS
D + 0.75L + 0.75S = 266 PSF

SPAN	$M = w\ell^2/8$ (1-SPAN OR 2-SPANS)
32"	2944 IN-LB
48"	6624 IN-LB
64"	11776 IN-LB

MAT.	ALUMINUM	STEEL
E =	10000 ksi	29000 ksi
F =	12000 psi	18000 psi

MIN

SPAN IN INCHES

$FSK > M$ & $EIK > 5w\ell^3/4608 \times 240$

BARS/FT

276 PSF

$K = 12/(7/16) = 27.4$

$12000(S)27.4 > 2944 \rightarrow S = 0.009\ IN^3$
$12000(S)27.4 > 6624 \rightarrow S = 0.020\ IN^3$
$12000(S)27.4 > 11776 \rightarrow S = 0.035\ IN^3$
} AL

PROJECT VILLAGE NEST JOB 2017-0610
CLIENT -
ADDRESS EDEN, UT
DESIGN HUUM
ENGINEER J.H. DATE 6/14/2017
RICHMOND PE 44628 HOFFMAYER SE 3435 SHEET 170 OF

$$\left.\begin{array}{l} 12/1\text{-}8(0.009) = 0.0061\ \text{IN}^3 \\ 12/1\text{-}8(0.020) = 0.0133\ \text{IN}^3 \\ 12/1\text{-}8(0.035) = 0.0233\ \text{IN}^3 \end{array}\right\}\ \text{STEEL.}$$

$$IK > 5(276)\ell^3/460BE$$

$$I > 0.0109\,\ell^3/E$$

$$\left.\begin{array}{l} 0.0109(32^3)/10E6 = 0.00004\ \text{IN}^4 \\ 0.0109(4\text{-}8^3)/10E6 = 0.0001\ \text{IN}^4 \\ 0.0109(64^3)/10E6 = 0.0003\ \text{IN}^4 \end{array}\right\}\ \text{AL}$$

$$\left.\begin{array}{l} 10/29(0.00004) = 0.000013\ \text{IN}^4 \\ 10/29(0.0001) = 0.000035\ \text{IN}^4 \\ 10/29(0.0003) = 0.0001\ \text{IN}^4 \end{array}\right\}\ \text{STEEL}$$

1" DEEP × 3/16" AL → $S = 0.0313\ \text{IN}^3$
$I = 0.0156\ \text{IN}^4$

* USE UP TO 4-8" SPANS W/ BARS @ 7/16"

1" DEEP × 3/16" STEEL → $S = 0.0313\ \text{IN}^3$
$I = 0.0156\ \text{IN}^4$

* USE UP TO 64" SPANS W/ BARS @ 7/16"

1" × 3/16" AL @ 7/16" GRATING
W/ SPANS @ 4-8"

Steel Beam

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

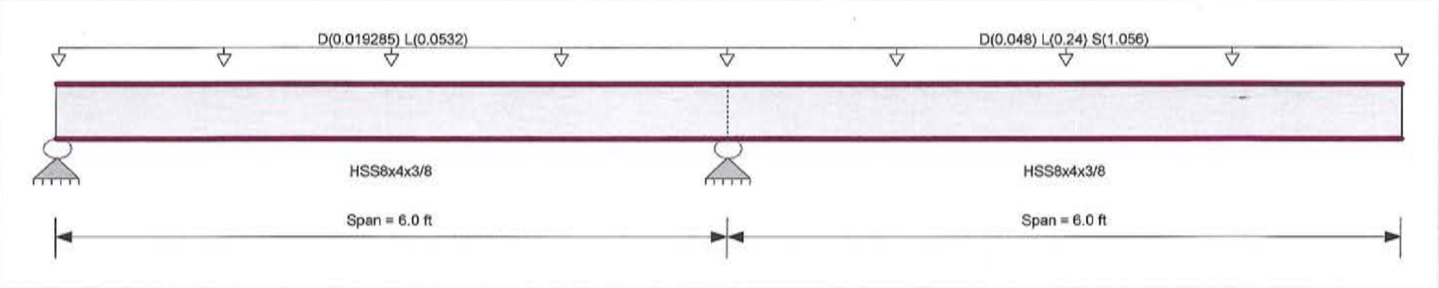
Description : B11 - DECK SUPPORT

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.01450, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)
Load for Span Number 2
Uniform Load : D = 0.0120, L = 0.060, S = 0.2640 ksf, Tributary Width = 4.0 ft, (DECK)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.434 : 1	Maximum Shear Stress Ratio =	0.078 : 1
Section used for this span	HSS8x4x3/8	Section used for this span	HSS8x4x3/8
Ma : Applied	20.367 k-ft	Va : Applied	6.789 k
Mn / Omega : Allowable	46.906 k-ft	Vn/Omega : Allowable	87.183 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	6.000ft	Location of maximum on span	6.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.089 in	Ratio = 1,616 >=360.
Max Upward Transient Deflection	-0.009 in	Ratio = 7,748 >=360.
Max Downward Total Deflection	0.397 in	Ratio = 363 >=240.
Max Upward Total Deflection	-0.043 in	Ratio = 1675 >=240.

Maximum Forces & Stresses for Load Combinations

Load Combination Segment Length	Span #	Max Stress Ratios M	 V	Summary of Moment Values Mmax +	 Mmax -	 Ma Max	 Mnx	 Mnx/Omega	 Cb	 Rm	Summary of Shear Values Va Max	 Vnx	 Vnx/Omega
D Only													
Dsgn. L = 6.00 ft	1	0.029	0.005		-1.36	1.36	78.33	46.91	1.00	1.00	0.45	145.60	87.18
Dsgn. L = 6.00 ft	2	0.029	0.005		-1.36	1.36	78.33	46.91	1.00	1.00	0.45	145.60	87.18
+D+L													
Dsgn. L = 6.00 ft	1	0.121	0.022		-5.68	5.68	78.33	46.91	1.00	1.00	1.89	145.60	87.18
Dsgn. L = 6.00 ft	2	0.121	0.022		-5.68	5.68	78.33	46.91	1.00	1.00	1.89	145.60	87.18
+D+S													
Dsgn. L = 6.00 ft	1	0.434	0.078		-20.37	20.37	78.33	46.91	1.00	1.00	6.79	145.60	87.18
Dsgn. L = 6.00 ft	2	0.434	0.078		-20.37	20.37	78.33	46.91	1.00	1.00	6.79	145.60	87.18
+D+0.750L													
Dsgn. L = 6.00 ft	1	0.098	0.018		-4.60	4.60	78.33	46.91	1.00	1.00	1.53	145.60	87.18
Dsgn. L = 6.00 ft	2	0.098	0.018		-4.60	4.60	78.33	46.91	1.00	1.00	1.53	145.60	87.18
+D+0.750L+0.750S													
Dsgn. L = 6.00 ft	1	0.402	0.072		-18.85	18.85	78.33	46.91	1.00	1.00	6.28	145.60	87.18
Dsgn. L = 6.00 ft	2	0.402	0.072		-18.85	18.85	78.33	46.91	1.00	1.00	6.28	145.60	87.18
+0.60D													
Dsgn. L = 6.00 ft	1	0.017	0.003		-0.82	0.82	78.33	46.91	1.00	1.00	0.27	145.60	87.18
Dsgn. L = 6.00 ft	2	0.017	0.003		-0.82	0.82	78.33	46.91	1.00	1.00	0.27	145.60	87.18

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+0.750L+0.750S	-0.0430	3.504

Steel Beam

Description : B11 - DECK SUPPORT

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S	2	0.3967	6.000		0.0000	3.504

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	-3.254	10.324	
Overall MINimum	-0.052	0.492	
D Only	-0.086	0.820	
+D+L	-0.647	3.139	
+D+S	-3.254	10.324	
+D+0.750L	-0.506	2.559	
+D+0.750L+0.750S	-2.882	9.687	
+0.60D	-0.052	0.492	
L Only	-0.560	2.320	
S Only	-3.168	9.504	

Wood Beam

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Lic. # : KW-06002886
Licensee : RICHMOND HOFFMAYER

Description : B12 - FLOOR JOISTS

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Spruce - Pine - Fir
Wood Grade : No. 1/No. 2

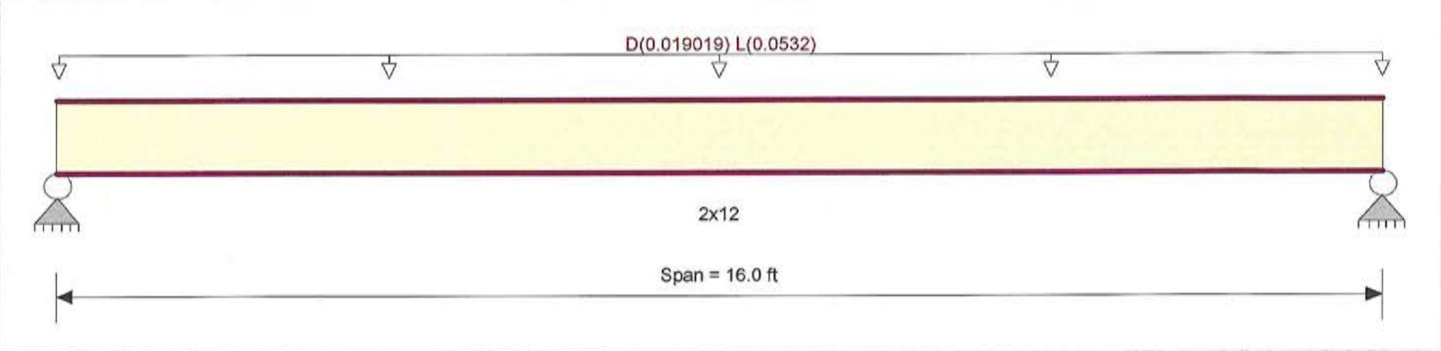
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	875.0 psi
Fb -	875.0 psi
Fc - Prll	1,150.0 psi
Fc - Perp	425.0 psi
Fv	135.0 psi
Ft	450.0 psi

E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	26.210 pcf

Repetitive Member Stress Increase



Applied Loads

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

Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.871 : 1	Maximum Shear Stress Ratio	=	0.336 : 1
Section used for this span		2x12	Section used for this span		2x12
fb : Actual	=	876.47 psi	fv : Actual	=	45.36 psi
FB : Allowable	=	1,006.25 psi	Fv : Allowable	=	135.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	8.000 ft	Location of maximum on span	=	15.066 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.317 in	Ratio =	606 >=360.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360.0
Max Downward Total Deflection	0.430 in	Ratio =	446 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.578	0.578
Overall MINimum	0.091	0.091
D Only	0.152	0.152
+D+L	0.578	0.578
+D+0.750L	0.471	0.471
+0.60D	0.091	0.091
L Only	0.426	0.426

Wood Beam

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Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : B12 - FLOOR JOISTS w/ PARTITION

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Spruce - Pine - Fir
Wood Grade : No. 1/No. 2

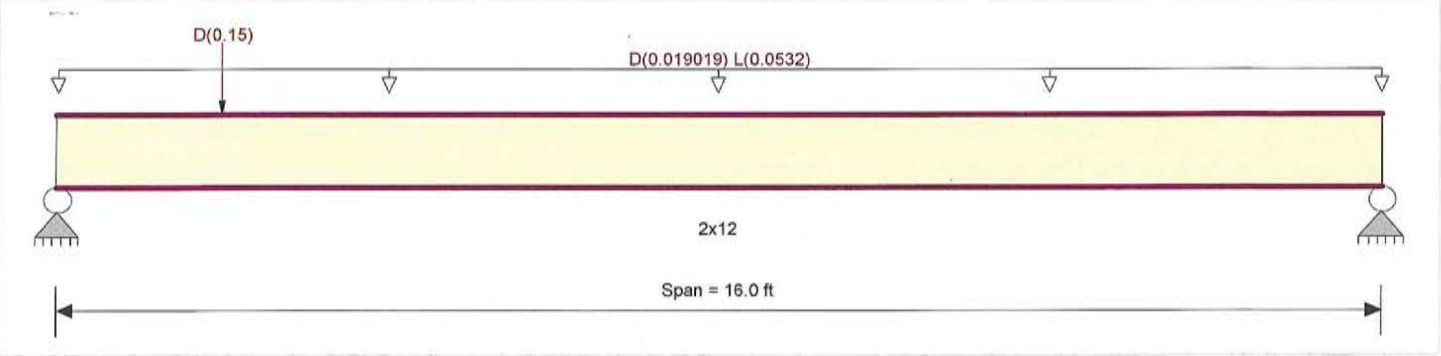
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	875.0 psi
Fb -	875.0 psi
Fc - Prll	1,150.0 psi
Fc - Perp	425.0 psi
Fv	135.0 psi
Ft	450.0 psi

E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	26.210 pcf

Repetitive Member Stress Increase



Applied Loads

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

Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)
Point Load : D = 0.150 k @ 2.0 ft, (WALL)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.928 : 1	Maximum Shear Stress Ratio	=	0.422 : 1
Section used for this span		2x12	Section used for this span		2x12
fb : Actual	=	934.27 psi	fv : Actual	=	57.02 psi
FB : Allowable	=	1,006.25 psi	Fv : Allowable	=	135.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	7.766 ft	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	0.317 in	Ratio =	606 >=360.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360.0
Max Downward Total Deflection	0.463 in	Ratio =	414 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.709	0.597
Overall MINimum	0.170	0.103
D Only	0.283	0.171
+D+L	0.709	0.597
+D+0.750L	0.603	0.490
+0.60D	0.170	0.103
L Only	0.426	0.426

Wood Beam

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Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : B13 - FLOOR JOISTS w/ DECK SUPPORT

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

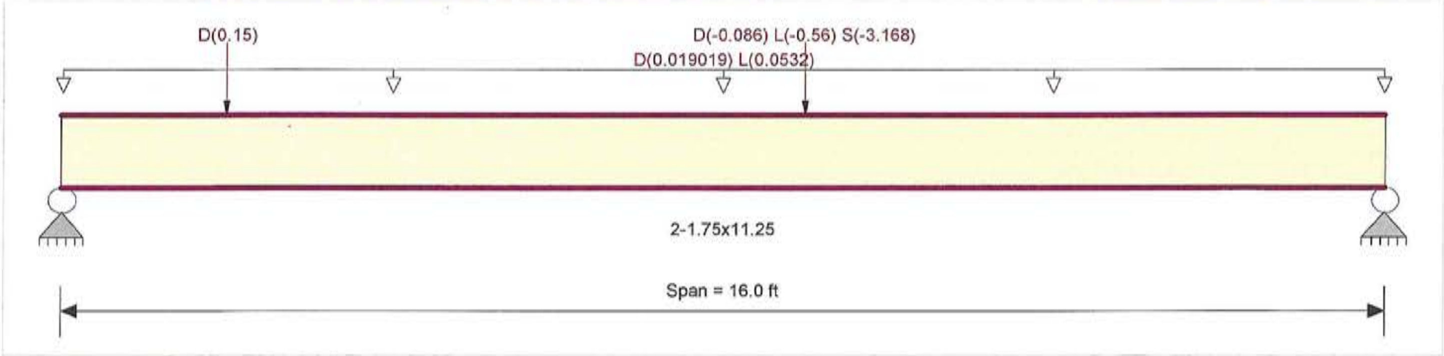
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,600.0 psi
Fb -	2,600.0 psi
Fc - Prll	2,510.0 psi
Fc - Perp	750.0 psi
Fv	285.0 psi
Ft	1,555.0 psi

E : Modulus of Elasticity

Ebend- xx	1,900.0 ksi
Eminbend - xx	965.71 ksi
Density	42.0 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 = 0.01430, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)
Point Load : D = 0.150 k @ 2.0 ft, (WALL)
Point Load : D = -0.0860, L = -0.560, S = -3.168 k @ 9.0 ft, (B9)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.554 : 1	Maximum Shear Stress Ratio	=	0.207 : 1
Section used for this span		2-1.75x11.25	Section used for this span		2-1.75x11.25
fb : Actual	=	1,903.30 psi	fv : Actual	=	67.79 psi
FB : Allowable	=	3,438.50 psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.993 ft	Location of maximum on span	=	9.051 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.003 in	Ratio = 68982 >=360.
Max Upward Transient Deflection	-0.004 in	Ratio = 49011 >=360.
Max Downward Total Deflection	0.052 in	Ratio = 3690 >=240.
Max Upward Total Deflection	-0.388 in	Ratio = 494 >=240.

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	-1.386	-1.782
Overall MINimum	0.181	0.111
D Only	0.338	0.214
+D+L	0.518	0.325
+D+S	-1.048	-1.568
+D+0.750L	0.473	0.297
+D+0.750L+0.750S	-0.566	-1.039
+0.60D	0.203	0.129
L Only	0.181	0.111
S Only	-1.386	-1.782

17e

Wood Beam

Description : B16 - FLOOR SUPPORT

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

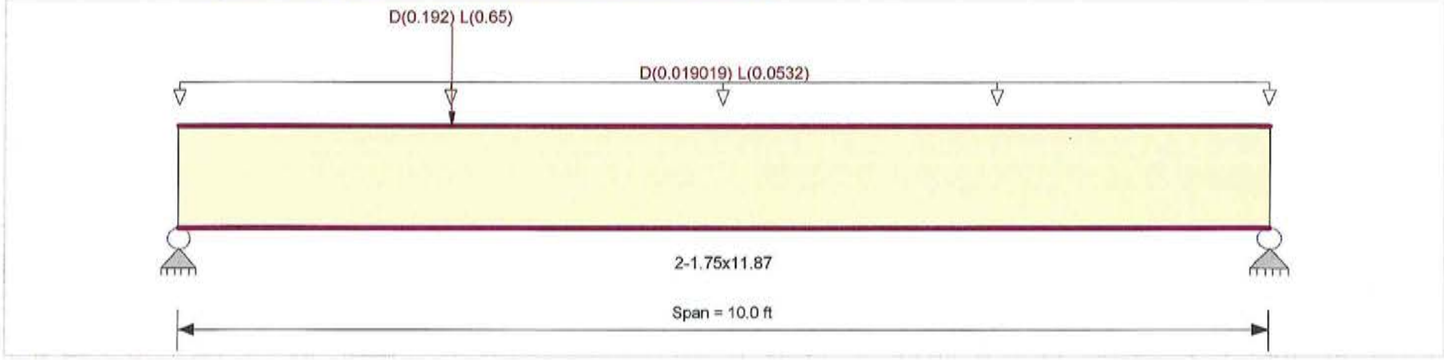
Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

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

Fb +	2,600.0 psi
Fb -	2,600.0 psi
Fc - Prll	2,510.0 psi
Fc - Perp	750.0 psi
Fv	285.0 psi
Ft	1,555.0 psi

E : Modulus of Elasticity

Ebend- xx	1,900.0 ksi
Eminbend - xx	965.71 ksi
Density	42.0 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)
Point Load : D = 0.1920, L = 0.650 k @ 2.50 ft, (B15)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.133 : 1	Maximum Shear Stress Ratio	=	0.123 : 1
Section used for this span		2-1.75x11.87	Section used for this span		2-1.75x11.87
fb : Actual	=	345.66 psi	fv : Actual	=	35.01 psi
FB : Allowable	=	2,600.00 psi	Fv : Allowable	=	285.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.518 ft	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	0.031 in Ratio =	3922 >=360
Max Upward Transient Deflection	0.000 in Ratio =	0 <360
Max Downward Total Deflection	0.043 in Ratio =	2766 >=240.
Max Upward Total Deflection	0.000 in Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.053	0.632
Overall MINimum	0.180	0.122
D Only	0.300	0.204
+D+L	1.053	0.632
+D+0.750L	0.865	0.525
+0.60D	0.180	0.122
L Only	0.754	0.429

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Wood Beam

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Description : B17 - FLOOR SUPPORT - FLUSH w/ FLOOR JOISTS

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

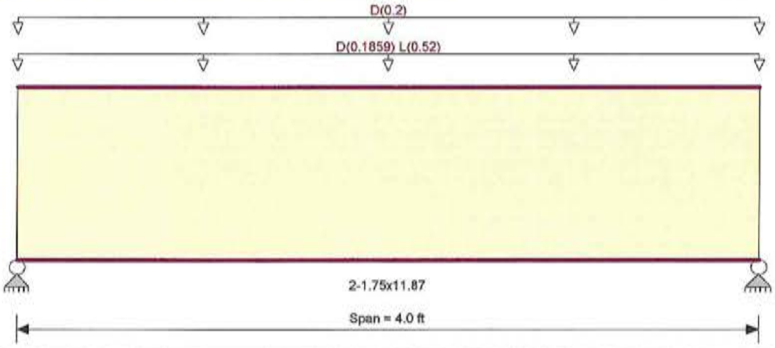
Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

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

Fb +	2,600.0 psi
Fb -	2,600.0 psi
Fc - Prll	2,510.0 psi
Fc - Perp	750.0 psi
Fv	285.0 psi
Ft	1,555.0 psi

E : Modulus of Elasticity

Ebend- xx	1,900.0 ksi
Eminbend - xx	965.71 ksi
Density	42.0 pcf



→ USE AS CONTINUOUS RIM —

Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 13.0 ft, (FLOOR)
Uniform Load : D = 0.20 , Tributary Width = 1.0 ft, (WALL ABV.)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.103 : 1	Maximum Shear Stress Ratio	=	0.119 : 1
Section used for this span		2-1.75x11.87	Section used for this span		2-1.75x11.87
fb : Actual	=	267.84 psi	fv : Actual	=	33.86 psi
FB : Allowable	=	2,600.00 psi	Fv : Allowable	=	285.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.000ft	Location of maximum on span	=	3.022 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.003 in	Ratio = 14785 >=360
Max Upward Transient Deflection	0.000 in	Ratio = 0 <360
Max Downward Total Deflection	0.006 in	Ratio = 8374 >=240.
Max Upward Total Deflection	0.000 in	Ratio = 0 <240.0

Vertical Reactions

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

Load Combination	Support 1	Support 2
Overall MAXimum	1.836	1.836
Overall MINimum	0.478	0.478
D Only	0.796	0.796
+D+L	1.836	1.836
+D+0.750L	1.576	1.576
+0.60D	0.478	0.478
L Only	1.040	1.040

180

Steel Beam

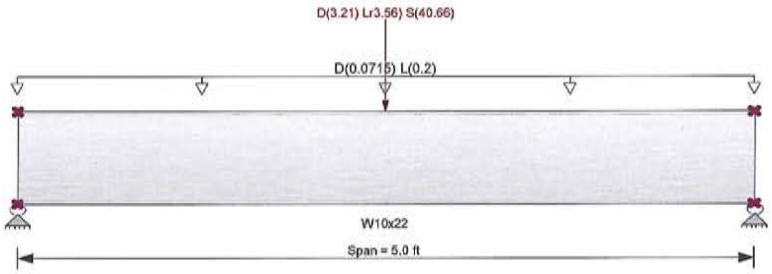
Description : B18 - RIDGE SUPPORT

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 5.0 ft, (FLOOR)
Point Load : D = 3.210, Lr = 3.560, S = 40.660 k @ 2.50 ft, (RIDGE B3 & B4)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.850 : 1	**Maximum Shear Stress Ratio =**	0.453 : 1
Section used for this span	**W10x22**	Section used for this span	**W10x22**
Ma : Applied	55.130 k-ft	Va : Applied	22.169 k
Mn / Omega : Allowable	64.870 k-ft	Vn/Omega : Allowable	48.960 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	2.500ft	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	0.005 in	Ratio =	12,761 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.009 in	Ratio =	6433 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 5.00 ft	1	0.066	0.038	4.30		4.30	108.33	64.87	1.30	1.00	1.84	73.44	48.96
+D+L													
Dsgn. L = 5.00 ft	1	0.076	0.048	4.93		4.93	108.33	64.87	1.28	1.00	2.34	73.44	48.96
+D+Lr													
Dsgn. L = 5.00 ft	1	0.135	0.074	8.75		8.75	108.33	64.87	1.31	1.00	3.62	73.44	48.96
+D+S													
Dsgn. L = 5.00 ft	1	0.850	0.453	55.13		55.13	108.33	64.87	1.32	1.00	22.17	73.44	48.96
+D+0.750Lr+0.750L													
Dsgn. L = 5.00 ft	1	0.125	0.072	8.11		8.11	108.33	64.87	1.30	1.00	3.55	73.44	48.96
+D+0.750L+0.750S													
Dsgn. L = 5.00 ft	1	0.661	0.357	42.89		42.89	108.33	64.87	1.31	1.00	17.46	73.44	48.96
+0.60D													
Dsgn. L = 5.00 ft	1	0.040	0.023	2.58		2.58	108.33	64.87	1.30	1.00	1.10	73.44	48.96

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0093	2.514		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	22.169	22.169
Overall MINimum	0.500	0.500
D Only	1.839	1.839

Steel Beam

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Description : B18 - RIDGE SUPPORT

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D+L	2.339	2.339
+D+Lr	3.619	3.619
+D+S	22.169	22.169
+D+0.750Lr+0.750L	3.549	3.549
+D+0.750L+0.750S	17.461	17.461
+0.60D	1.103	1.103
Lr Only	1.780	1.780
L Only	0.500	0.500
S Only	20.330	20.330

/e2

Steel Column

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : B18 SUPPORT

Code References

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Steel Section Name :	**Pipe4STD**
Analysis Method :	**Allowable Strength**
Steel Stress Grade	
Fy : Steel Yield	36.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height	6.0 ft
Top & Bottom Fixity	**Top & Bottom Pinned**

Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 6.0 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for Y-Y Axis buckling = 6.0 ft, K = 1.0

Applied Loads

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

Column self weight included : 64.80 lbs * Dead Load Factor
AXIAL LOADS . . .
B18: Axial Load at 6.0 ft, D = 1.839, LR = 1.780, L = 0.50, S = 20.330 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.3928 : 1
	Load Combination	+D+S
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Pa : Axial	22.234 k
	Pn / Omega : Allowable	56.610 k
	Ma-x : Applied	0.0 k-ft
	Mn-x / Omega : Allowable	7.275 k-ft
	Ma-y : Applied	0.0 k-ft
	Mn-y / Omega : Allowable	7.275 k-ft
PASS	Maximum Shear Stress Ratio =	0.0 : 1
	Load Combination	
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Va : Applied	0.0 k
	Vn / Omega : Allowable	0.0 k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum Load Deflections . . .

Along Y-Y	0.0 in	at	0.0ft above base
for load combination :			
Along X-X	0.0 in	at	0.0ft above base
for load combination :			

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios Stress Ratio	Status	Location	Maximum Shear Ratios Stress Ratio	Status	Location
D Only	0.034	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+L	0.042	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+Lr	0.065	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+S	0.393	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750Lr+0.750L	0.064	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750L+0.750S	0.310	PASS	0.00 ft	0.000	PASS	0.00 ft
+0.60D	0.020	PASS	0.00 ft	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
D Only	1.904								
+D+L	2.404								
+D+Lr	3.684								
+D+S	22.234								
+D+0.750Lr+0.750L	3.614								
+D+0.750L+0.750S	17.526								
+0.60D	1.142								

Steel Column

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Description : B18 SUPPORT

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
Lr Only	1.780								
L Only	0.500								
S Only	20.330								

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	22.234								
"	Minimum	0.500								
Reaction, X-X Axis Base	Maximum	1.904								
"	Minimum	1.904								
Reaction, Y-Y Axis Base	Maximum	1.904								
"	Minimum	1.904								
Reaction, X-X Axis Top	Maximum	1.904								
"	Minimum	1.904								
Reaction, Y-Y Axis Top	Maximum	1.904								
"	Minimum	1.904								
Moment, X-X Axis Base	Maximum	1.904								
"	Minimum	1.904								
Moment, Y-Y Axis Base	Maximum	1.904								
"	Minimum	1.904								
Moment, X-X Axis Top	Maximum	1.904								
"	Minimum	1.904								
Moment, Y-Y Axis Top	Maximum	1.904								
"	Minimum	1.904								

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+L	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr+0.750L	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750L+0.750S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
Lr Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
L Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
S Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Steel Section Properties : Pipe4STD

Depth	=	4.500 in	I xx	=	6.82 in^4	J	=	13.600 in^4
			S xx	=	3.03 in^3			
Diameter	=	4.500 in	R xx	=	1.510 in			
Wall Thick	=	0.237 in	Zx	=	4.050 in^3			
Area	=	2.960 in^2	I yy	=	6.820 in^4			
Weight	=	10.800 plf	S yy	=	3.030 in^3			
			R yy	=	1.510 in			

Ycg = 0.000 in

Steel Column

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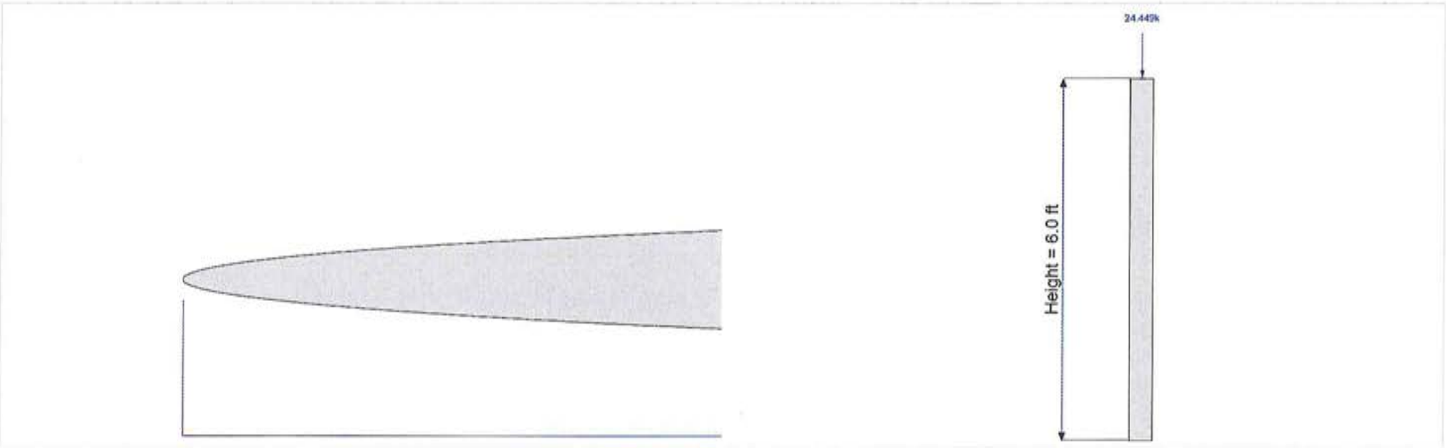
Description : B18 SUPPORT



Wood Beam

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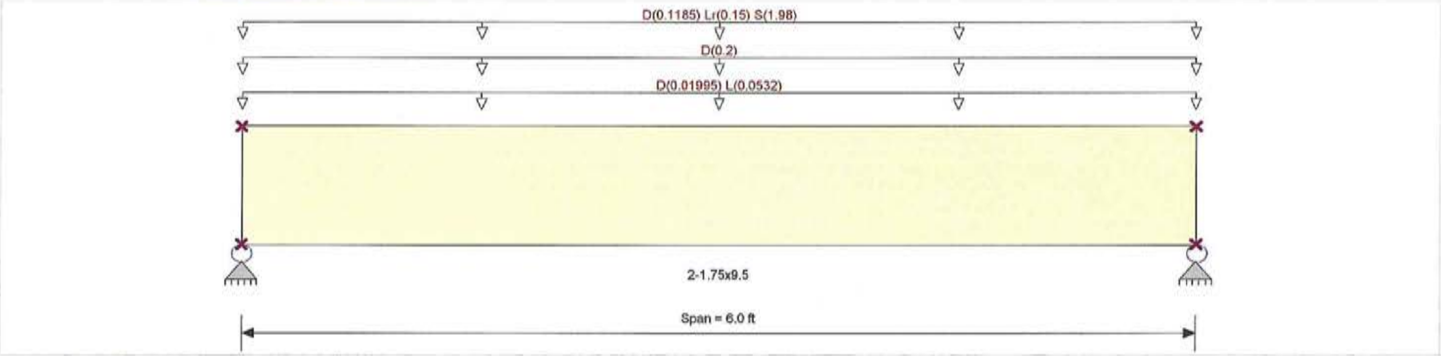
Description : B19 - HEADERS

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,600.0 psi	*E : Modulus of Elasticity*	
Load Combination	ASCE 7-10	Fb -	2,600.0 psi	Ebend- xx	1,900.0 ksi
		Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Wood Species	: Trus Joist	Fc - Perp	750.0 psi		
Wood Grade	: MicroLam LVL 1.9 E	Fv	285.0 psi		
		Ft	1,555.0 psi	Density	42.0 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, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)
Uniform Load : D = 0.20 , Tributary Width = 1.0 ft, (WALL)
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Tributary Width = 7.50 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.815 : 1	Maximum Shear Stress Ratio	=	0.709 : 1
Section used for this span		**2-1.75x9.5**	Section used for this span		**2-1.75x9.5**
fb : Actual	=	2,388.03 psi	fv : Actual	=	232.29 psi
FB : Allowable	=	2,931.26 psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.000 ft	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	0.122 in	Ratio =	589 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.144 in	Ratio =	500 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	6.984	6.984
Overall MINimum	0.160	0.160
D Only	1.044	1.044
+D+L	1.204	1.204
+D+Lr	1.494	1.494
+D+S	6.984	6.984
+D+0.750Lr+0.750L	1.502	1.502
+D+0.750L+0.750S	5.619	5.619
+0.60D	0.627	0.627
Lr Only	0.450	0.450

Wood Beam

Description : **B19 - HEADERS**

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
L Only	0.160	0.160
S Only	5.940	5.940

1-e7

Wood Beam

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Description : B20 - HEADERS

CODE REFERENCES

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

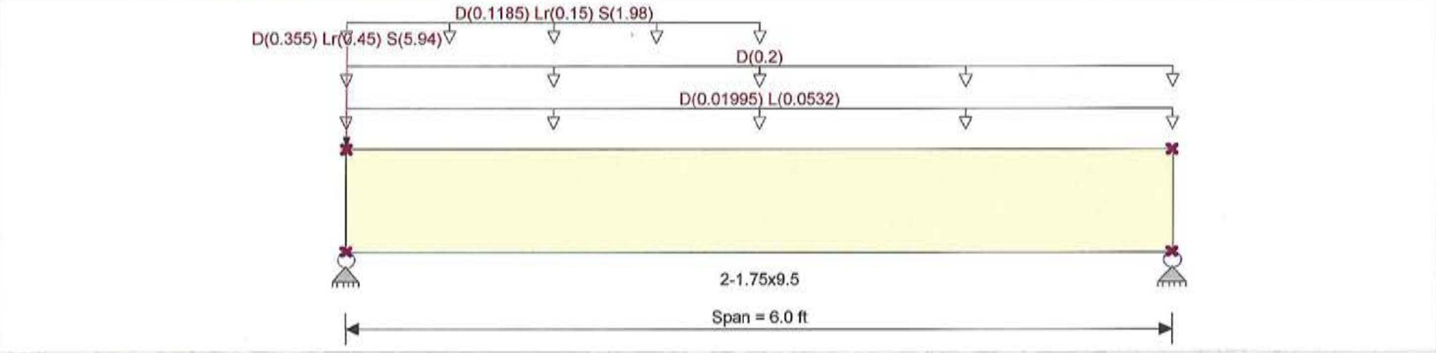
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Trus Joist
Wood Grade : MicroLam LVL 1.9 E

Beam Bracing : Completely Unbraced

Fb +	2,600.0 psi	*E : Modulus of Elasticity*	
Fb -	2,600.0 psi	Ebend- xx	1,900.0 ksi
Fc - Prll	2,510.0 psi	Eminbend - xx	965.71 ksi
Fc - Perp	750.0 psi		
Fv	285.0 psi		
Ft	1,555.0 psi	Density	42.0 pcf



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, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)
Uniform Load : D = 0.20 , Tributary Width = 1.0 ft, (WALL)
Uniform Load : D = 0.01580, Lr = 0.020, S = 0.2640 ksf, Extent = 0.0 -->> 3.0 ft, Tributary Width = 7.50 ft, (ROOF)
Point Load : D = 0.3550, Lr = 0.450, S = 5.940 k @ 0.0 ft, (ROOF HEADER)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.489 : 1	Maximum Shear Stress Ratio	=	0.492 : 1
Section used for this span		2-1.75x9.5	Section used for this span		2-1.75x9.5
fb : Actual	=	1,433.05psi	fv : Actual	=	161.29 psi
FB : Allowable	=	2,931.26psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	2.321ft	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	0.062 in	Ratio =	1167 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.079 in	Ratio =	906 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	11.706	2.263
Overall MINimum	0.160	0.113
D Only	1.311	0.778
+D+L	1.470	0.937
+D+Lr	2.098	0.890
+D+S	11.706	2.263
+D+0.750Lr+0.750L	2.021	0.982
+D+0.750L+0.750S	9.227	2.011
+0.60D	0.786	0.467

Wood Beam

Description : **B20 - HEADERS**

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Lr Only	0.788	0.113
L Only	0.160	0.160
S Only	10.395	1.485

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Steel Beam

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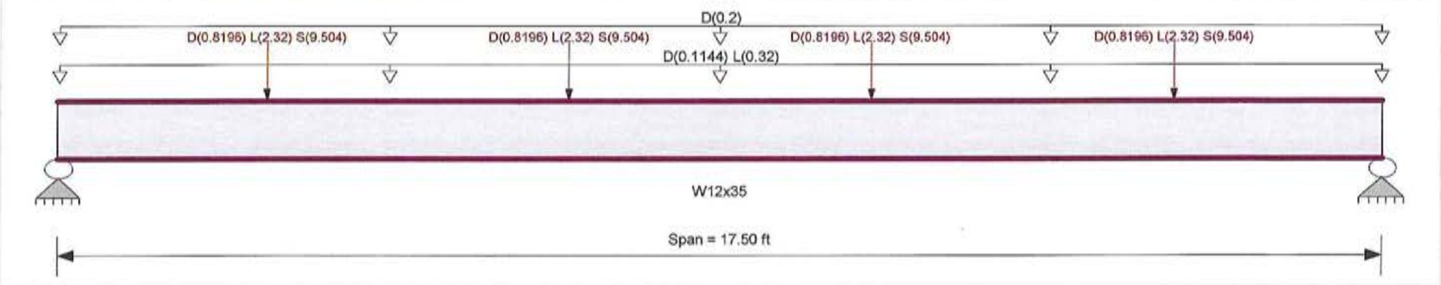
Description : B21 - BALCONY SUPPORT

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		


D(0.8196) L(2.32) S(9.504)
D(0.8196) L(2.32) S(9.504)
D(0.2)
D(0.1144) L(0.32)
D(0.8196) L(2.32) S(9.504)
D(0.8196) L(2.32) S(9.504)
W12x35
Span = 17.50 ft

Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.01430, L = 0.040 ksf, Tributary Width = 8.0 ft, (FLOOR)
Uniform Load : D = 0.20 k/ft, Tributary Width = 1.0 ft, (GLASS)
Point Load : D = 0.8196, L = 2.320, S = 9.504 k @ 2.750 ft, (BALCONY - B13)
Point Load : D = 0.8196, L = 2.320, S = 9.504 k @ 6.750 ft, (BALCONY - B13)
Point Load : D = 0.8196, L = 2.320, S = 9.504 k @ 10.750 ft, (BALCONY - B13)
Point Load : D = 0.8196, L = 2.320, S = 9.504 k @ 14.750 ft, (BALCONY - B13)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.897 : 1**	**Maximum Shear Stress Ratio =**	**0.327 : 1**
Section used for this span	**W12x35**	Section used for this span	**W12x35**
Ma : Applied	114.595 k-ft	Va : Applied	24.532 k
Mn / Omega : Allowable	127.745 k-ft	Vn/Omega : Allowable	75.0 k
Load Combination	+D+0.750L+0.750S	Load Combination	+D+0.750L+0.750S
Location of maximum on span	8.750ft	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	0.233 in	Ratio =	902 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.375 in	Ratio =	559 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 17.50 ft	1	0.166	0.063	21.16		21.16	213.33	127.74	1.00	1.00	4.70	112.50	75.00
+D+L													
Dsgn. L = 17.50 ft	1	0.434	0.162	55.45		55.45	213.33	127.74	1.00	1.00	12.14	112.50	75.00
+D+S													
Dsgn. L = 17.50 ft	1	0.872	0.316	111.45		111.45	213.33	127.74	1.00	1.00	23.70	112.50	75.00
+D+0.750L													
Dsgn. L = 17.50 ft	1	0.367	0.137	46.88		46.88	213.33	127.74	1.00	1.00	10.28	112.50	75.00
+D+0.750L+0.750S													
Dsgn. L = 17.50 ft	1	0.897	0.327	114.60		114.60	213.33	127.74	1.00	1.00	24.53	112.50	75.00
+0.60D													
Dsgn. L = 17.50 ft	1	0.099	0.038	12.70		12.70	213.33	127.74	1.00	1.00	2.82	112.50	75.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.3754	8.800		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	24.532	24.532

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Steel Beam

Description : B21 - BALCONY SUPPORT

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MINimum	2.818	2.818
D Only	4.696	4.696
+D+L	12.136	12.136
+D+S	23.704	23.704
+D+0.750L	10.276	10.276
+D+0.750L+0.750S	24.532	24.532
+0.60D	2.818	2.818
L Only	7.440	7.440
S Only	19.008	19.008

Steel Column

Description : B21 SUPPORT

Code References

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Steel Section Name :	HSS6x6x3/8
Analysis Method :	Allowable Strength
Steel Stress Grade	
Fy : Steel Yield	36.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height 10.0 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 10.0 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for Y-Y Axis buckling = 10.0 ft, K = 1.0

Applied Loads

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

Column self weight included : 274.80 lbs * Dead Load Factor
AXIAL LOADS . . .
B21: Axial Load at 10.0 ft, D = 4.696, L = 7.440, S = 19.0 k
B6 MAX: Axial Load at 10.0 ft, Xecc = 0.50 in, Yecc = 0.50 in, D = 4.363, LR = 4.028, S = 71.674 k
BENDING LOADS . . .
WIND: Lat. Uniform Load creating Mx-x, W = 1.0 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.9052 : 1
Load Combination	+D+S
Location of max.above base	9.933 ft
At maximum location values are . . .	
Pa : Axial	100.008 k
Pn / Omega : Allowable	141.228 k
Ma-x : Applied	-3.147 k-ft
Mn-x / Omega : Allowable	28.383 k-ft
Ma-y : Applied	-3.147 k-ft
Mn-y / Omega : Allowable	28.383 k-ft
PASS Maximum Shear Stress Ratio =	0.06750 : 1
Load Combination	+D+0.60W
Location of max.above base	10.0 ft
At maximum location values are . . .	
Va : Applied	3.018 k
Vn / Omega : Allowable	44.716 k

Maximum Load Reactions . .

Top along X-X	0.3168 k
Bottom along X-X	0.3168 k
Top along Y-Y	5.0 k
Bottom along Y-Y	5.0 k

Maximum Load Deflections . . .

Along Y-Y 0.1985 in at 5.034ft above base
for load combination : W Only

Along X-X -0.03093 in at 5.839ft above base
for load combination : +D+S

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios Stress Ratio	Status	Location	Maximum Shear Ratios Stress Ratio	Status	Location
D Only	0.066	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+L	0.119	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+Lr	0.095	PASS	0.00 ft	0.001	PASS	0.00 ft
+D+S	0.905	PASS	9.93 ft	0.007	PASS	0.00 ft
+D+0.750Lr+0.750L	0.127	PASS	0.00 ft	0.001	PASS	0.00 ft
+D+0.750L+0.750S	0.738	PASS	9.93 ft	0.005	PASS	0.00 ft
+D+0.60W	0.297	PASS	5.03 ft	0.067	PASS	10.00 ft
+D+0.750Lr+0.750L+0.450W	0.262	PASS	5.03 ft	0.051	PASS	10.00 ft
+D+0.750L+0.750S+0.450W	0.763	PASS	4.97 ft	0.056	PASS	10.00 ft
+0.60D+0.60W	0.284	PASS	5.03 ft	0.067	PASS	10.00 ft
+0.60D	0.040	PASS	0.00 ft	0.000	PASS	0.00 ft

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Steel Column

Description : B21 SUPPORT

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
D Only	9.334	0.018	0.018	-0.018	0.018				
+D+L	16.774	0.018	0.018	-0.018	0.018				
+D+Lr	13.362	0.035	0.035	-0.035	0.035				
+D+S	100.008	0.317	0.317	-0.317	0.317				
+D+0.750Lr+0.750L	17.935	0.031	0.031	-0.031	0.031				
+D+0.750L+0.750S	82.919	0.242	0.242	-0.242	0.242				
+D+0.60W	9.334	0.018	0.018	2.982	3.018				
+D+0.750Lr+0.750L+0.450W	17.935	0.031	0.031	2.219	2.281				
+D+0.750L+0.750S+0.450W	82.919	0.242	0.242	2.008	2.492				
+0.60D+0.60W	5.600	0.011	0.011	2.989	3.011				
+0.60D	5.600	0.011	0.011	-0.011	0.011				
Lr Only	4.028	0.017	0.017	-0.017	0.017				
L Only	7.440								
S Only	90.674	0.299	0.299	-0.299	0.299				
W Only				5.000	5.000				

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	100.008	0.317	0.317	-0.317	0.317				
"	Minimum				5.000	5.000				
Reaction, X-X Axis Base	Maximum	100.008	0.317	0.317	-0.317	0.317				
"	Minimum	7.440								
Reaction, Y-Y Axis Base	Maximum				5.000	5.000				
"	Minimum	100.008	0.317	0.317	-0.317	0.317				
Reaction, X-X Axis Top	Maximum	100.008	0.317	0.317	-0.317	0.317				
"	Minimum	7.440								
Reaction, Y-Y Axis Top	Maximum	100.008	0.317	0.317	-0.317	0.317				
"	Minimum	7.440								
Moment, X-X Axis Base	Maximum	9.334		0.018	-0.018	0.018				
"	Minimum	9.334		0.018	-0.018	0.018				
Moment, Y-Y Axis Base	Maximum	9.334	0.018	0.018	-0.018	0.018				
"	Minimum	9.334	0.018	0.018	-0.018	0.018				
Moment, X-X Axis Top	Maximum	9.334	0.018	0.018	-0.018	0.018				
"	Minimum	9.334	0.018	0.018	-0.018	0.018				
Moment, Y-Y Axis Top	Maximum	9.334	0.018	0.018	-0.018	0.018				
"	Minimum	9.334	0.018	0.018	-0.018	0.018				

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	-0.0018	in	5.839	ft	-0.002	in	5.839	ft
+D+L	-0.0018	in	5.839	ft	-0.002	in	5.839	ft
+D+Lr	-0.0034	in	5.839	ft	-0.003	in	5.839	ft
+D+S	-0.0309	in	5.839	ft	-0.031	in	5.839	ft
+D+0.750Lr+0.750L	-0.0030	in	5.839	ft	-0.003	in	5.839	ft
+D+0.750L+0.750S	-0.0236	in	5.839	ft	-0.024	in	5.839	ft
+D+0.60W	-0.0018	in	5.839	ft	0.117	in	5.034	ft
+D+0.750Lr+0.750L+0.450W	-0.0030	in	5.839	ft	0.086	in	5.034	ft
+D+0.750L+0.750S+0.450W	-0.0236	in	5.839	ft	0.066	in	4.832	ft
+0.60D+0.60W	-0.0011	in	5.839	ft	0.118	in	5.034	ft
+0.60D	-0.0011	in	5.839	ft	-0.001	in	5.839	ft
Lr Only	-0.0016	in	5.839	ft	-0.002	in	5.839	ft
L Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
S Only	-0.0292	in	5.839	ft	-0.029	in	5.839	ft
W Only	0.0000	in	0.000	ft	0.199	in	5.034	ft

Steel Section Properties : HSS6x6x3/8

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Steel Column

Description : B21 SUPPORT

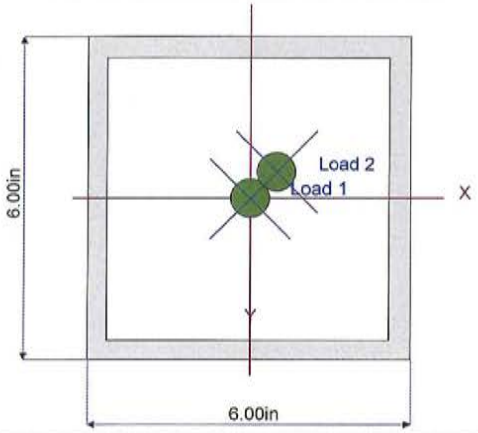
Steel Section Properties : HSS6x6x3/8

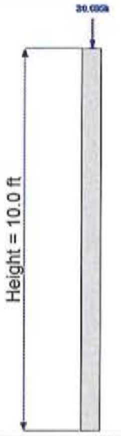
Depth = 6.000 in
Design Thick = 0.349 in
Width = 6.000 in
Wall Thick = 0.375 in
Area = 7.580 in^2
Weight = 27.480 plf

I xx = 39.50 in^4
S xx = 13.20 in^3
R xx = 2.280 in
Zx = 15.800 in^3
I yy = 39.500 in^4
S yy = 13.200 in^3
R yy = 2.280 in

J = 64.600 in^4
C = 22.100 in^3

Ycg = 0.000 in





SIP WALL INFORMATION:

ICC-ES ESR-1882

TRY TYPE "S" PANELS

SEISMIC DESIGN CATEGORY D -
PERIODIC SPECIAL INSPECTION REQUIRED FOR FASTENING AND ANCHORING OF SHEAR WALLS.

TRY 7-1/4" PANEL CORE THICKNESS.

→ MAX HEIGHT = 10'-6"

$W_{\text{UNIF ROOF}} = (15.8_D + 20_{L_r} + 264_S)\,14'/2 = \underline{\underline{2099 \text{ PLF}}}$

$W_{\text{UNIF 2ND}} = (14.3_D + 40_L)\,8'/2 = \underline{\underline{217 \text{ PLF}}}$

$W_{\text{MAX}} = 2099 + 217 = \underline{\underline{2316 \text{ PLF}}}$

SEE TABLE 5 NEXT PAGE, CAP = $\underline{\underline{4326 \text{ PLF}}}$ ✓

→ MAX HEADER SPAN W/O POINT LOAD = 4.5'

$W_{\text{MAX}} \le \underline{\underline{2099 \text{ PLF}}}$

SEE TABLE 14, PG #196, CAP = $\underline{\underline{2620 \text{ PLF}}}$ ✓

DBL TRIMMER, PREMIER INSUL-BEAM II

TABLE 4—UNIFORM TRANSVERSE LOADS FOR FACE SUPPORTED PREMIER TYPE L PANELS[1,2,3] (psf)

PANEL CORE THICKNESS (inches)	DEFLECTION	PANEL SPAN									
		4 ft[4]	8 ft	10 ft	12 ft	14 ft	16 ft	18 ft	20 ft	22 ft	24 ft
$3^{1}/_{2}$[5]	$^{L}/_{360}$	98	45	32	24	16	11	----	----	----	----
	$^{L}/_{240}$	215	67	47	34	24	16	----	----	----	----
	$^{L}/_{180}$	298*	90	61	44	34	22	----	----	----	----
$5^{1}/_{2}$[6]	$^{L}/_{360}$	241	128	57	41	33	25	20	15	----	----
	$^{L}/_{240}$	288*	182*	86	60	49	37	29	22	----	----
	$^{L}/_{180}$	288*	182*	112*	79	65	49	39	29	----	----
$7^{1}/_{4}$[7]	$^{L}/_{360}$	241	168	80	65	54	42	33	24	----	----
	$^{L}/_{240}$	288*	188*	126	99	81	61	49	34	----	----
	$^{L}/_{180}$	288*	188*	133*	117*	105	80	62	44	----	----
$9^{1}/_{4}$[8]	$^{L}/_{360}$	274	188*	116	100	80	62	47	35	32	28
	$^{L}/_{240}$	326*	188*	147*	134*	120	92	70	52	46	41
	$^{L}/_{180}$	326*	188*	147*	134*	121*	108*	93	68	61	53
$11^{1}/_{4}$[8]	$^{L}/_{360}$	327*	188*	167*	140	116	90	75	57	47	36
	$^{L}/_{240}$	327*	188*	167*	153*	132*	110*	97*	83*	69	53
	$^{L}/_{180}$	327*	188*	167*	153*	132*	110*	97*	83*	83*	70

For **SI:** 1 inch = 25.4 mm, 1 psf = 47.9 Pa, 1 foot = 304.8 mm.

[1] Floor panels must have a minimum $^{3}/_{4}$-inch-thick top skin or a minimum $^{7}/_{16}$-inch-thick top skin overlaid with minimum $^{7}/_{16}$-inch-thick finish flooring perpendicular to the panels.

[2] The tabulated values are for roof and floor panels installed with simply supported single span conditions with panels supported at each end on a minimum $1^{1}/_{2}$-inch-wide continuous support in contact with the panel face. Tabulated values are applicable to panels installed with the strong axis of the OSB panel facers parallel to the panel span.

[3] Allowable loads with an asterisk, *, indicates a capacity based on the average peak test load divided by 3.

[4] Panels spanning 4 feet shall be a minimum of 8 feet long spanning a minimum of two 4 foot spans. No single span conditions must be permitted.

[5] $3^{1}/_{2}$-inch thick core panels must be limited to a maximum span of 10 feet when used in roof applications.

[6] $5^{1}/_{2}$-inch thick core panels must be limited to a maximum span of 14 feet when used in roof applications.

[7] $7^{1}/_{4}$-inch thick core panels must be limited to a maximum span of 18 feet when used in roof applications.

[8] $9^{1}/_{4}$ and $11^{1}/_{4}$-inch thick core panels shall be limited to a maximum span of 20 feet when used in roof applications.

TABLE 5—ALLOWABLE UNIFORM AXIAL LOADS FOR PREMIER TYPE S PANELS (plf)[1,2,3,4]

PANEL CORE THICKNESS (inches)	PANEL SPAN					
	8 ft	10 ft	12 ft	16 ft	20 ft	24 ft
$3^{1}/_{2}$	3,500	2,555	2,450	2,120	----	----
$5^{1}/_{2}$	4,250	4,040	3,375	3,920	2,815	----
$7^{1}/_{4}$	4,915	4,325	4,475	4,195	3,495	3,065
$9^{1}/_{4}$	4,200	4,200	4,200	4,200	3,389	3,247
$11^{1}/_{4}$	3,890	3,890	3,890	3,890	3,890	3,333

For **SI:** 1 inch = 25.4 mm, 1 plf = 14.6 N/m, 1 foot = 304.8 mm.

[1] For the allowable axial load on the fire-resistance-rated assembly, see Section 4.2.6.

[2] For combined loads; requirements in Section 4.1 must be applied.

[3] The tabulated loads are uniform axial loads applied concentrically to the full thickness of the panels, including panel facings.

[4] The tabulated values are for panels installed with strong axis of the OSB panel facers parallel to the wall height (panel span) and on concrete foundations. The member, element, or structure supporting the bearing wall panels, as shown in Figures 7, 8 and 9, must be designed for the bearing stress of the wall panels to the satisfaction to the code official.

TABLE 6—ALLOWABLE UNIFORM AXIAL LOADS FOR PREMIER TYPE L PANELS (plf)[1,2,3,4]

PANEL CORE THICKNESS (inches)	PANEL SPAN					
	8 ft	10 ft	12 ft	16 ft	20 ft	24 ft
$3^{1}/_{2}$	4,725	3,905	3,095	2,350	----	----
$5^{1}/_{2}$	5,850	5,890	4,280	4,310	2,933	----
$7^{1}/_{4}$	6,850	6,110	5,555	5,180	4,835	4,080
$9^{1}/_{4}$	5,470	5,470	5,470	5,470	5,470	4,250
$11^{1}/_{4}$	4,500	4,333	4,167	3,750	3,750	3,333

For **SI:** 1 inch = 25.4 mm, 1 plf = 14.6 N/m, 1 foot = 304.8 mm.

[1] For the allowable axial load on fire-resistance-rated assembly, see Section 4.2.7.

[2] For combined loads; requirements in Section 4.1 must be applied.

[3] The tabulated loads are uniform axial loads applied concentrically to the full thickness of the panels, including panel facings.

[4] The tabulated values are for panels installed with strong axis of the OSB panel facers parallel to the wall height (panel span) on concrete foundations. The member, element, or structure supporting the bearing wall panels, as shown in Figures 7, 8 and 9, must be designed for the bearing stress of the wall panels to the satisfaction to the code official.

Table 13: Wind Speed vs. Pressure

Wall Loads (psf) - End Zone (Zone 5) for 100sf to 500 sf effective wind area												
Mean Roof Height (ft)	90 MPH			100 MPH			110 MPH			120 MPH		
	Exp B	Exp C	Exp D	Exp B	Exp C	Exp D	Exp B	Exp C	Exp D	Exp B	Exp C	Exp D
15	-15.1	-18.3	-22.2	-18.7	-22.6	-27.5	-22.6	-27.3	-33.2	-26.9	-32.5	-39.5
20	-15.1	-19.5	-23.4	-18.7	-24.1	-29.0	-22.6	-29.2	-35.0	-26.9	-34.7	-41.7
25	-15.1	-20.4	-24.3	-18.7	-25.2	-30.1	-22.6	-30.5	-36.4	-26.9	-36.3	-43.3
30	-15.1	-21.1	-25.1	-18.7	-26.2	-31.0	-22.6	-31.6	-37.5	-26.9	-37.7	-44.7
35	-15.9	-21.9	-25.7	-19.6	-27.1	-31.8	-23.7	-32.8	-38.4	-28.2	-39.0	-45.7
40	-16.5	-22.5	-26.3	-20.4	-27.9	-32.5	-24.6	-33.7	-39.3	-29.3	-40.1	-46.8
45	-16.9	-23.1	-26.9	-20.9	-28.6	-33.3	-25.3	-34.6	-40.2	-30.1	-41.2	-47.9
50	-17.5	-23.6	-27.3	-21.7	-29.2	-33.8	-26.2	-35.3	-40.9	-31.2	-42.0	-48.7
55	-18.0	-24.0	-27.8	-22.3	-29.7	-34.4	-26.9	-35.9	-41.6	-32.0	-42.8	-49.5
60	-18.4	-24.5	-28.2	-22.8	-30.3	-35.0	-27.6	-36.6	-42.3	-32.8	-43.6	-50.3
Net Design wind pressure	-15.1			-18.7			-22.6			-26.9		

Wall Loads (psf) - End Zone (Zone 5) for 100sf to 500sf effective wind area												
Mean Roof Height (ft)	130 MPH			140 MPH			150 MPH			170 MPH		
	Exp B	Exp C	Exp D	Exp B	Exp C	Exp D	Exp B	Exp C	Exp D	Exp B	Exp C	Exp D
15	-31.6	-38.2	-46.5	-36.7	-44.4	-53.9	-42.1	-50.9	-61.9	-54.1	-65.5	-79.5
20	-31.6	-40.8	-49.0	-36.7	-47.3	-56.9	-42.1	-54.3	-65.3	-54.1	-69.8	-83.9
25	-31.6	-42.7	-50.9	-36.7	-49.5	-59.1	-42.1	-56.8	-67.8	-54.1	-73.0	-87.1
30	-31.6	-44.2	-52.5	-36.7	-51.4	-60.9	-42.1	-58.9	-69.9	-54.1	-75.7	-89.8
35	-33.2	-45.8	-53.7	-38.5	-53.2	-62.4	-44.2	-61.1	-71.6	-56.8	-78.4	-92.0
40	-34.4	-47.1	-55.0	-40.0	-54.7	-63.9	-45.9	-62.7	-73.3	-59.0	-80.6	-94.1
45	-35.4	-48.3	-56.2	-41.1	-56.2	-65.3	-47.2	-64.4	-74.9	-60.6	-82.8	-96.3
50	-36.7	-49.3	-57.2	-42.6	-57.3	-66.4	-48.8	-65.7	-76.2	-62.8	-84.4	-97.9
55	-37.6	-50.2	-58.1	-43.7	-58.4	-67.5	-50.1	-66.9	-77.5	-64.4	-86.0	-99.5
60	-38.6	-51.2	-59.1	-44.8	-59.5	-68.6	-51.4	-68.2	-78.7	-66.0	-87.6	-101.2
Net Design wind pressure	-31.6			-36.7			-42.1			-54.1		

More information on this chart can be found in Technical Bulletin #15 (www.premiersips.com).

Table 14: Premier Insul-Beam II Header Loads (plf)

No. of Trimmer Studs	Deflection	Header Span (ft.)						
		2'	3'	4'	5'	6'	7'	8'
1	L/480	3150	2100	1575	1260	1050	900	788
	L/360	3150	2100	1575	1260	1050	900	788
	L/240	3150	2100	1575	~~1260~~	1050	900	788
2	L/480	6300	4200	3150	2520	2100	1800	1545
	L/360	6300	4200	3150	2520	2100	1800	1575
	L/240	6300	4200	3150	2520	2100	1800	1575

No. of Trimmer Studs	Deflection	Header Span (ft.)							
		9'	10'	11'	12'	13'	14'	15'	16'
1	L/480	700	630	573	458	360	288	234	193
	L/360	700	630	573	525	480	384	313	257
	L/240	700	630	573	525	485	450	420	386
2	L/480	1085	791	594	458	360	288	234	193
	L/360	1400	1055	792	610	480	384	313	257
	L/240	1400	1245	792	864	720	577	469	386

Values listed for each deflection represent the least value of the bearing capacity of the trimmer, shear or bending capacity of the header or the actual deflection at the design load.
Refer to Technical Bulletin #30 for supporting headers in Premier SIPs wall panels (www.premiersips.com).
Note: Trimmer stud design capacities must be reviewed.

SINGLE TOP PLATE W/ CAP

Table 6: Maximum Allowable Axial Compression Point Loads (lbs) – Type S Panels[1,2,3,4]

Top Plate Configuration	1.5" Minimum Bearing Width	3" Minimum Bearing Width
Single 2x4 #2 or Better Hem-Fir Plate	2040	2450
Single 2x4 #2 or Better Hem-Fir Plate with 1-1/8 in. wide, 1.3E Rim Board Cap Plate	4030	4678

[1] Top plate secured to facings as required in Section 6.3

[2] Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.

[3] Concentrated loads shall be applied concentrically to the top of the panel.

[4] Tabulated values are based on the strong-axis of the facing material oriented parallel to the span direction.

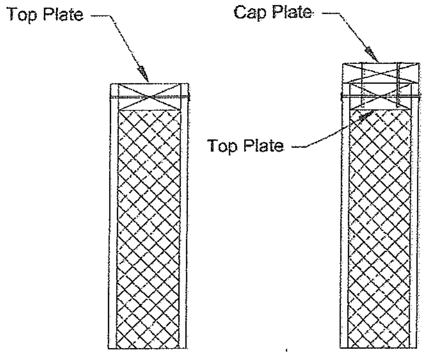


Figure 3: Top Plate Configurations

2099 PLF × 16/12

= 2799 lb LOAD FROM RF

Table 7: Maximum Allowable Uniform SIP Header Vertical Loads (plf)
3-1/2 in. through 11-1/4 in. Core Thickness[1,2]

Continuous over opening

Header Depth[3] (in)	Deflection Limit[4]	Header Span (ft)			
		4	6	8	10
12	L/480	740	384	228	142
	L/360	740	384	229	142
	L/240	740	384	229	142
18	L/480	798	574	385	311
	L/360	798	574	385	311
	L/240	798	574	385	311
24	L/480	886	629	429	361
	L/360	886	629	429	361
	L/240	886	629	429	361

[1] Vertical loads only. Lateral loads shall be transferred to the edges of the openings through continuous plate(s) designed in accordance with accepted engineering practice. Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.

[2] Tabulated values are based on the strong-axis of the facing material oriented perpendicular to the direction of header span.

[3] Minimum depth of facing above opening.

[4] Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code.

Table 8: Maximum Allowable Uniform Header Loads (plf)
(Panel Splice a minimum of 6 in. from edge of opening) 3-1/2 in. through 11-1/4 in. Core Thickness[1,2]

SIPs wall panel spline minimum 6" from edge of opening

Header Depth[3] (in)	Deflection Limit[4]	Header Span (ft)			
		4	6	8	10
12	L/480	345	243	156	99
	L/360	450	295	190	125
	L/240	630	382	236	153
18	L/480	705	388	254	235
	L/360	750	482	302	281
	L/240	750	482	302	281
24	L/480	698	556	368	350
	L/360	896	556	368	350
	L/240	896	556	368	350

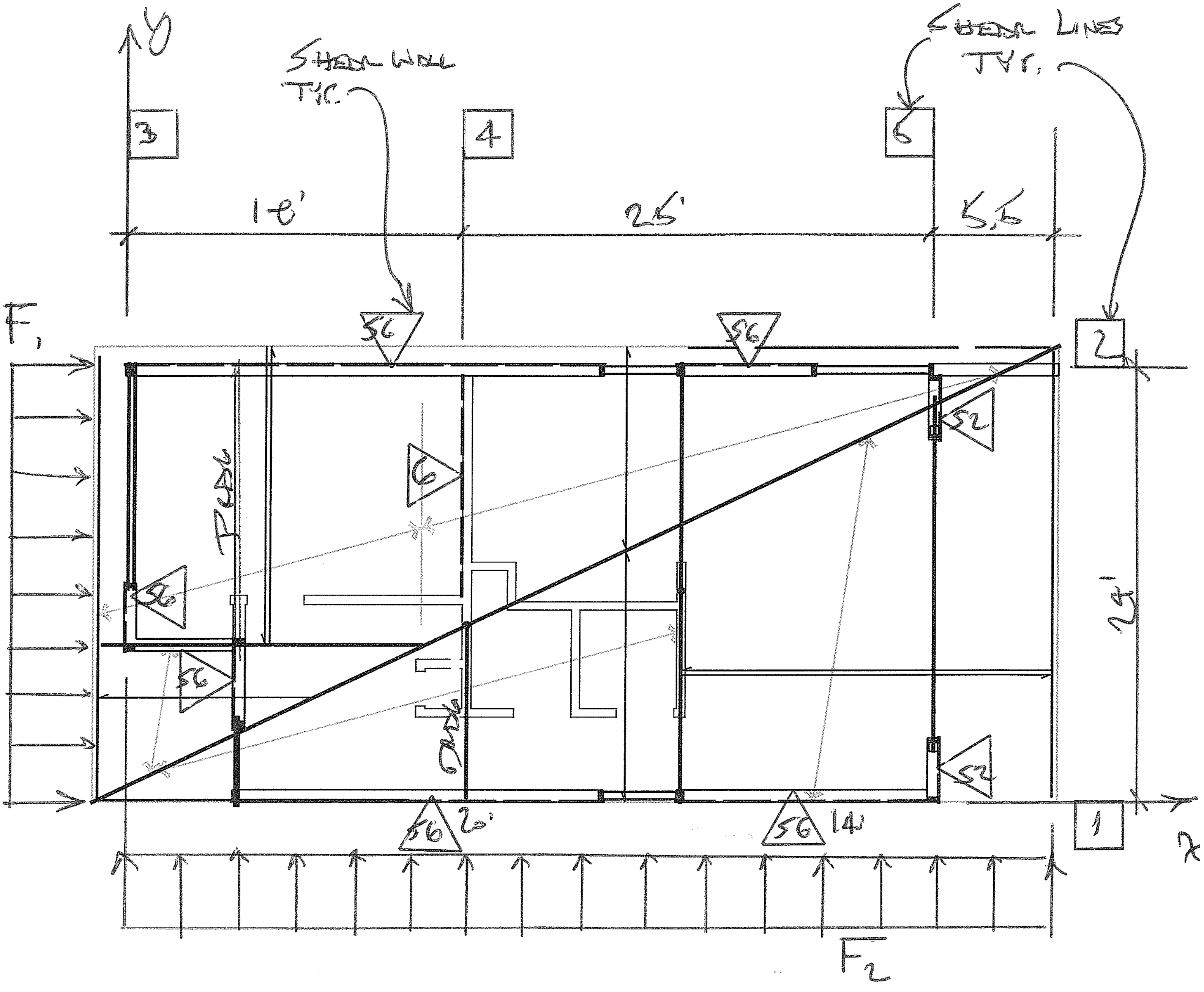
[1] Vertical loads only. Lateral loads shall be transferred to the edges of the openings through continuous plate(s) designed in accordance with accepted engineering practice. Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.

[2] Tabulated values are based on the strong-axis of the facing material oriented perpendicular to the direction of header span.

[3] Minimum depth of facing above opening.

[4] Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code.

PROJECT	FELDSPAR LOWER	JOB	2017-0610
CLIENT	VILLAGE NEST		
ADDRESS	EDEN, UTAH		
DESIGN	HUUM		
ENGINEER	J.H. RICHMOND PE 49628 HOFFMAYER SE 3935	DATE	6/16/2017
		SHEET	198 OF

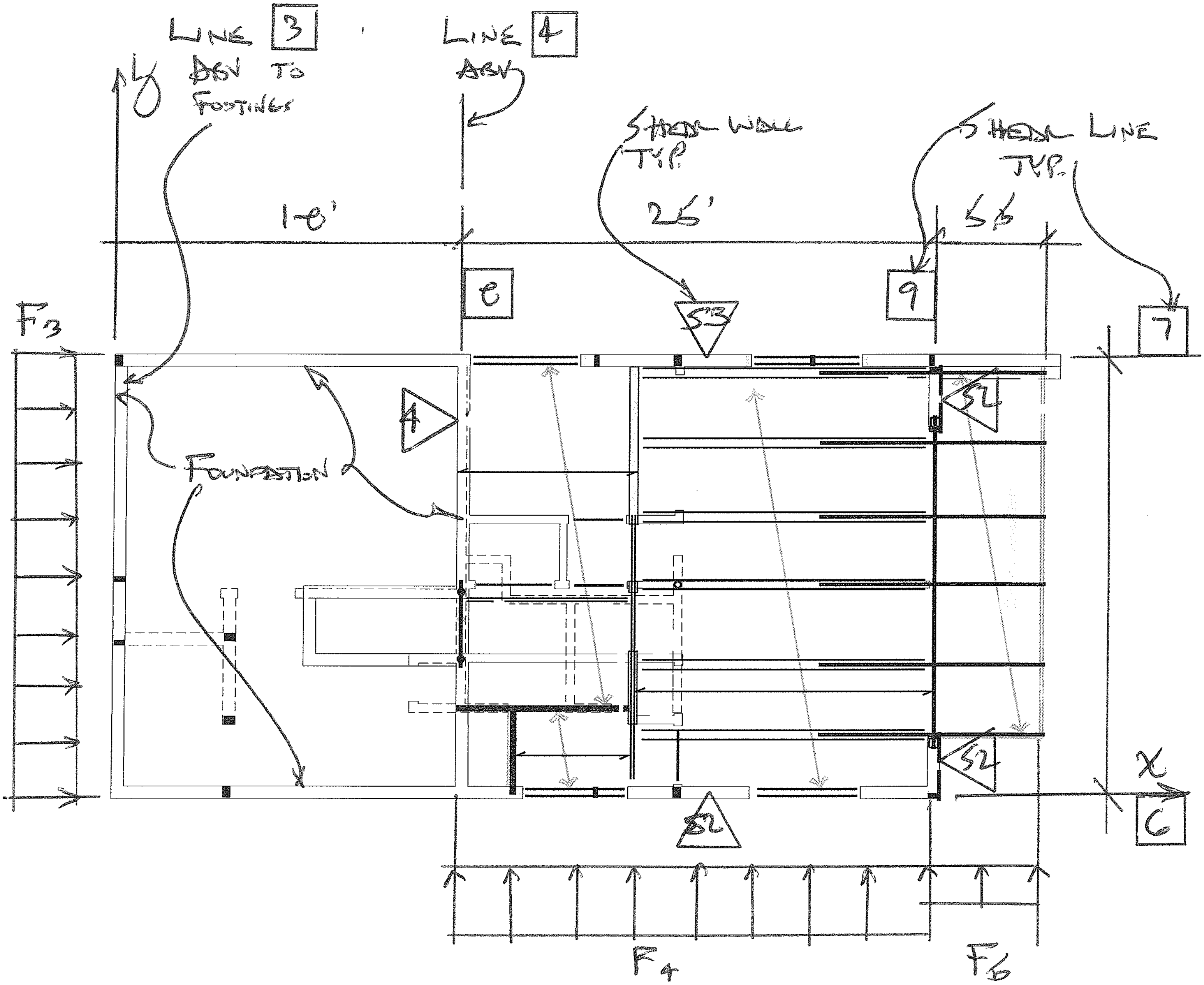

y
SHEAR WALL TYP.
SHEAR LINES TYP.
3
4
6
18'
25'
5.5
F1
56
56
2
6
52
56
56
52
24'
56
20'
56
14'
1
x
F2

2ND FLOOR SHEAR WALLS

FELDSPAR UVN
N.T.S.

RICHMOND HOFFMAYER INC.
STRUCTURAL ENGINEERING

PROJECT FELDSPAR LOWER
JOB 2017-0610
CLIENT VILLAGE NEST
ADDRESS EDEN, UTAH
DESIGN HUUM
ENGINEER J.H.
RICHMOND PE 49628 HOFFMAYER SE 3935
DATE 6/16/2017
SHEET 199 OF



LINE 3
y
ABV TO FOOTINGS
LINE 4
ABV
SHEAR WALL TYP.
SHEAR LINE TYP.
1-0'
25'
5'6
8
9
7
S3
4
S2
S2
F3
FOUNDATION
S2
X
6
F4
F6

1st FLOOR SHEAR WALLS

FELDSPAR UVN
N.T.S.

SEISMIC DESIGN:

ADDRESS	5754 DAYBREAK RIDGE, EDEN, UTAH				
	GOOGLE EARTH				
	LATITUDE	N	= 41.3600		
	LONGTITUDE	W	= -111.7400		
	USGS NSHMP - 2012 IBC				
	BASED ON SITE CLASS B, Fa = 1.0 AND Fv = 1.0	Ss	= 0.806		
		S1	= 0.267		

2016CBC / ASCE 7-10

1603.1.5 EARTHQUAKE DESIGN DATA

No.	Item	Symbol	Value		Reference
1	SEISMIC IMPORTANCE FACTOR AND OCCUPANCY	I	= 1.00		
	OCCUPANCY		= II		*ASCE 7-10 TABLE 11.5-1*
2	MAPPED SPECTRAL RESPONSE ACCELERATIONS	Ss	= 0.81		*USGS MAPS OF SPECTRAL RESPONSE*
		S1	= 0.27		
3	SITE CLASS		D		IBC DEFAULT
	SITE COEFFICIENTS	1.00	= 1.18		*TABLE 11.4-1*
		Fv	= 1.87		*TABLE 11.4-2*
4	SPECTRAL RESPONSE COEFFICIENTS	SMS	= 0.949		
		SM1	= 0.498		
		SDS	= 0.633	= 2/3 SMS	*ASCE 7-10 (11.4-3)*
		SD1	= 0.332	= 2/3 SM1	*ASCE 7-10 (11.4-4)*
5	SEISMIC DESIGN CATEGORY		D		*IBC2012 TABLES 1613.5.6(1) & 1613.5.6(2)*

MAIN BUILDING DIRECTION - X

No.	Item	Symbol	Value		Reference
6	BASIC SEISMIC-FORCE-RESISTING SYSTEM A. BEARING WALL SYSTEMS A-LIGHT-FRAMED WALLS SHEATHED WITH WOOD PANELS				
		R	= 6.50		*ASCE 7-10 TABLE 12.2-1*
		Ωo	= 3.00		*ASCE 7-10 TABLE 12.2-1*
		Cd	= 4.00		*ASCE 7-10 TABLE 12.2-1*
7	DESIGN BASE SHEAR	V	= Cs W		
8	SEISMIC RESPONSE COEFFICIENT	Cs	= 0.097	= SDS / (R / I)	*ASCE 7-10 (12.8-2)*
		T	= Cu Ta		
		Cu	= 1.40		*ASCE 7-10 TABLE 12.8-1*
		hn	= 20.00	ft	
		Ct	= 0.02		*ASCE 7-10 TABLE 12.8-2*
		x	= 0.75		*ASCE 7-10 TABLE 12.8-2*
		Ta	= 0.189	= Ct hn^x	*ASCE 7-10 (12.8-7)*
		T	= 0.265	s	*ASCE 7-10 12.8.2*
		TL	= 8.00	s	*ASCE 7-10 FIG.22-16*
		Cs'	= 0.193	MAX	*ASCE 7-10 (12.8-3)*
		Cs"	= 0.010	MIN	*ASCE 7-10 (12.8-5)*
		Cs"'	= 0.010	MIN'	*ASCE 7-10 (12.8-6)*
		Cs USE	= 0.097		Cs OVERIDE 0.000
9	RESPONSE MODIFICATION FACTOR	R	= 6.50		
10	ANALYSIS PROCEDURE USED		EQUIVALENT LATERAL FORCE PROCEDURE		
		V	= 0.0973	W	*X-DIRECTION BASE SHEAR FORCE*

MAIN BUILDING DIRECTION - Y

No.	Item	Symbol	Value		Reference
6	BASIC SEISMIC-FORCE-RESISTING SYSTEM A. BEARING WALL SYSTEMS A-LIGHT-FRAMED WALLS SHEATHED WITH WOOD PANELS				
		R	= 6.50		*ASCE 7-10 TABLE 12.2-1*
		Ωo	= 3.00		*ASCE 7-10 TABLE 12.2-1*
		Cd	= 4.00		*ASCE 7-10 TABLE 12.2-1*
7	DESIGN BASE SHEAR	V	= Cs W		
8	SEISMIC RESPONSE COEFFICIENT	Cs	= 0.097	= SDS / (R / I)	*ASCE 7-10 (12.8-2)*
		T	= Cu Ta		
		Cu	= 1.40		*ASCE 7-10 TABLE 12.8-1*
		hn	= 20.00	ft	
		Ct	= 0.02		*ASCE 7-10 TABLE 12.8-2*
		x	= 0.75		*ASCE 7-10 TABLE 12.8-2*
		Ta	= 0.189	= Ct hn^x	*ASCE 7-10 (12.8-7)*
		T	= 0.265	s	*ASCE 7-10 12.8.2*
		TL	= 8.00	s	*ASCE 7-10 FIG.22-16*
		Cs'	= 0.193	MAX	*ASCE 7-10 (12.8-3)*
		Cs"	= 0.010	MIN	*ASCE 7-10 (12.8-5)*
		Cs"'	= 0.010	MIN'	*ASCE 7-10 (12.8-6)*
		Cs USE	= 0.097		Cs OVERIDE 0.000
9	RESPONSE MODIFICATION FACTOR	R	= 6.50		
10	ANALYSIS PROCEDURE USED		EQUIVALENT LATERAL FORCE PROCEDURE		
		V	= 0.0973	W	*Y-DIRECTION BASE SHEAR FORCE*

DATE 7/7/2017
SHEET ______

AREA WEIGHTS - 2ND LEVEL:

ROOF

9.00 ft WALL HT.

WEIGHT TABULATIONS:

2X = 88628.16 lb

2Y = 91676.91 lb

WEIGHT TYPES:

A	METAL ROOF	15.76 psf	U	EXTERIOR	15.23 psf
B	OPENED BALCONY	7.00 psf	V	INTERIOR	8.85 psf
C	2nd FLOOR	14.25 psf	W	-	0.00 psf
D	DRIVEWAY LOADS	85.38 psf	X	-	0.00 psf
E	20% SNOW LOAD FOR SEISM	52.80 psf	Y	-	0.00 psf
F	GARAGE LOADS	63.50 psf	Z	-	0.00 psf

W#	F#		UNIT WEIGHT	TYPE		LENGTH		UNIT WEIGHT	TYPE		HEIGHT		NUMBER		UNIT WEIGHT	TYPE		HEIGHT		NUMBER		TRIB. WEIGHT		WIDTH		TOTAL WEIGHT
2-1	1	2X	15.76 psf	A	x	51.00 ft	+	15.23 psf	U	x	5.00 ft	x	2.00	+	8.85 psf	V	x	5.00 ft	x	1.00	=	1000.04 plf	x	24.00 ft	=	24000.96 lb
		+	52.80 psf	E	x	51.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	2692.80 plf	x	24.00 ft	=	64627.20 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
																										88628.16 lb
2-2	2	2Y	15.76 psf	A	x	24.00 ft	+	15.23 psf	U	x	5.00 ft	x	2.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	530.39 plf	x	51.00 ft	=	27049.71 lb
		+	52.80 psf	E	x	24.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	1267.20 plf	x	51.00 ft	=	64627.20 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
																										91676.91 lb
2--	-	-	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
																										0.00 lb
2--	-	-	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
																										0.00 lb
2--	-	-	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
																										0.00 lb
2--	-	-	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
		+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
																										0.00 lb

201

AREA WEIGHTS - 1ST LEVEL:

2ND FLOOR

9.00 ft WALL HT.

WEIGHT TABULATIONS:

1X = 18564.00 lb

1Y = 18564.00 lb

WEIGHT TYPES:

A	METAL ROOF	15.76 psf	U	EXTERIOR	15.23 psf
B	OPENED BALCONY	7.00 psf	V	INTERIOR	8.85 psf
C	2nd FLOOR	14.25 psf	W	-	0.00 psf
D	DRIVEWAY LOADS	85.38 psf	X	-	0.00 psf
E	20% SNOW LOAD FOR SEISM	52.80 psf	Y	-	0.00 psf
F	GARAGE LOADS	63.50 psf	Z	-	0.00 psf

W#	F#			UNIT WEIGHT	TYPE		LENGTH		UNIT WEIGHT	TYPE		HEIGHT		NUMBER		UNIT WEIGHT	TYPE		HEIGHT		NUMBER		TRIB. WEIGHT		WIDTH		TOTAL WEIGHT
1-3	3	1X		14.25 psf	C	x	24.00 ft	+	15.23 psf	U	x	10.00 ft	x	2.00	+	8.85 psf	V	x	10.00 ft	x	1.00	=	735.00 plf	x	24.00 ft	=	17640.00 lb
			+	7.00 psf	B	x	5.50 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	38.50 plf	x	24.00 ft	=	924.00 lb
			+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
			+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
																											18564.00 lb
1-4	4	1Y		14.25 psf	C	x	24.00 ft	+	15.23 psf	U	x	10.00 ft	x	2.00	+	8.85 psf	V	x	10.00 ft	x	1.00	=	735.00 plf	x	24.00 ft	=	17640.00 lb
			+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
			+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
			+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
																											17640.00 lb
1-5	5	1Y		7.00 psf	B	x	24.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	168.00 plf	x	5.50 ft	=	924.00 lb
			+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
			+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
			+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
																											924.00 lb
1--	-	-		0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
			+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
			+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
			+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
																											0.00 lb
1--	-	-		0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
			+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
			+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
			+	0.00 psf	-	x	0.00 ft	+	0.00 psf	-	x	0.00 ft	x	0.00	+	0.00 psf	-	x	0.00 ft	x	0.00	=	0.00 plf	x	0.00 ft	=	0.00 lb
																											0.00 lb

202

EQUIVALENT LATERAL FORCE PROCEDURE:

BASE SHEAR	V	=	Cs W		*ASCE 7-10 (12.8-1)*	
TOTAL BUILDING DEAD WEIGHT - X DIRECTION	Wx	=	107192.16	lb		
X-DIRECTION SEISMIC RESPONSE COEFICIENT	Csx	=	0.0973		SEE PREVIOUS PAGES ... Rx =	6.50
STRENGTH LEVEL BASE SHEAR FORCE - X	Vx	=	10434.97	lb		
TOTAL BUILDING DEAD WEIGHT - Y DIRECTION	Wy	=	110240.91	lb		
Y-DIRECTION SEISMIC RESPONSE COEFICIENT	Csy	=	0.0973		SEE PREVIOUS PAGES ... Ry =	6.50
STRENGTH LEVEL BASE SHEAR FORCE - Y	Vy	=	10731.76	lb		

ASCE 7-10

12.8.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES

PER EACH ORTHOGONAL DIRECTION x & y

F = Cv V *ASCE 7-10 (12.8-11)*

$C_v = \dfrac{w\,h^k}{\Sigma\, w_i h_i^k}$ *ASCE 7-10 (12.8-12)*

X-DIRECTION BUILDING PERIOD T = 0.2648 s SEE PREVIOIUS PAGES
k = 1.00 *ASCE 7-10 12.8.3*

LEVEL	wx	hx	hx ^ k	wx hx ^ k	Cvx	STRENGTH SHEAR FORCE PER LEVEL Fx	Wx Csx	DISTRIBUTION FACTOR Fx / (Wx Csx)
-	0.00	0.00	0.000	0.00	0.000	0.00	0.00	0.000
-	0.00	0.00	0.000	0.00	0.000	0.00	0.00	0.000
-	0.00	0.00	0.000	0.00	0.000	0.00	0.00	0.000
-	0.00	0.00	0.000	0.00	0.000	0.00	0.00	0.000
2	88628.16	20.00	20.000	1772563.12	0.905	9445.72	8627.80	1.095
1	18564.00	10.00	10.000	185640.00	0.095	989.25	1807.17	0.547
SUM	107192.16			1958203.12	1.000	10434.97		

Y-DIRECTION BUILDING PERIOD T = 0.2648 s SEE PREVIOIUS PAGES
k = 1.00 *ASCE 7-10 12.8.3*

LEVEL	wy	hy	hxy ^ k	wy hy ^ k	Cvy	STRENGTH SHEAR FORCE PER LEVEL Fy	Wy Csy	DISTRIBUTION FACTOR Fy / (Wy Csy)
-	0.00	0.00	0.000	0.00	0.000	0.00	0.00	0.000
-	0.00	0.00	0.000	0.00	0.000	0.00	0.00	0.000
-	0.00	0.00	0.000	0.00	0.000	0.00	0.00	0.000
-	0.00	0.00	0.000	0.00	0.000	0.00	0.00	0.000
2	91676.91	20.00	20.000	1833538.12	0.908	9745.10	8924.59	1.092
1	18564.00	10.00	10.000	185640.00	0.092	986.66	1807.17	0.546
SUM	110240.91			2019178.12	1.000	10731.76		

DATE 7/7/2017
SHEET 209

SEISMIC FORCES:

4TH LEVEL FORCES -

F#		TOTAL TRIB. WEIGHT		DIST. FACTOR		Cs		DESIGN SHEAR FORCE (STRENGTH LEVEL)			DISTRIBUTION OVERIDE	R OVERIDE
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		

3RD LEVEL FORCES -

F#		TOTAL TRIB. WEIGHT		DIST. FACTOR		Cs		DESIGN SHEAR FORCE (STRENGTH LEVEL)			DISTRIBUTION OVERIDE	R OVERIDE
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		

2ND LEVEL FORCES ROOF

F#		TOTAL TRIB. WEIGHT		DIST. FACTOR		Cs		DESIGN SHEAR FORCE (STRENGTH LEVEL)			DISTRIBUTION OVERIDE	R OVERIDE
1	2X	3692.84 plf	x	1.09	x	0.0973	=	393.57 plf	R =	6.50		
2	2Y	1797.59 plf	x	1.09	x	0.0973	=	191.08 plf	R =	6.50		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		

1ST LEVEL FORCES 2ND FLOOR

F#		TOTAL TRIB. WEIGHT		DIST. FACTOR		Cs		DESIGN SHEAR FORCE (STRENGTH LEVEL)			DISTRIBUTION OVERIDE	R OVERIDE
3	1X	773.50 plf	x	0.55	x	0.0973	=	41.22 plf	R =	6.50		
4	1Y	735.00 plf	x	0.55	x	0.0973	=	39.06 plf	R =	6.50		
5	1Y	168.00 plf	x	0.55	x	0.0973	=	8.93 plf	R =	6.50		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		
-	-	0.00 plf	x	0.00	x	0.0000	=	0.00 plf	R =	0.00		

SEISMIC FORCES TO SHEAR LINES:

2ND LEVEL ROOF

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
1	F1	-	(	393.57 plf	)x	24.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	4722.86 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(-	)x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
												6.50								4722.86 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
2	F1	-	(	393.57 plf	)x	24.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	4722.86 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(-	)x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
												6.50								4722.86 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
3	F2	-	(	191.08 plf	)x	18.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	1719.72 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(-	)x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
												6.50								1719.72 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
4	F2	-	(	191.08 plf	)x	18.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	1719.72 lb
+	F2	-	(	191.08 plf	)x	25.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	2388.50 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(-	)x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
												6.50								4108.23 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
5	F2	-	(	191.08 plf	)x	25.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	2388.50 lb
+	F2	-	(	191.08 plf	)x	5.50	x	27.75	/	24.00	x	6.50	/	6.50	x	1.00	/	1.00	=	1215.15 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		)x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(-	)x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
												6.50								3603.66 lb

SEISMIC FORCES TO SHEAR LINES:

1ST LEVEL 2ND FLOOR

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
6	F3	-	(	41.22 plf	) x	24.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	494.62 lb
+	-	LINE1	(		4722.86 lb) x	1.00	x	1.00	/	1.00	x	6.50	/	6.50	x	1.00	/	1.00	=	4722.86 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(-	) x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
												6.50								5217.49 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
7	F3	-	(	41.22 plf	) x	24.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	494.62 lb
+	-	LINE2	(		4722.86 lb) x	1.00	x	1.00	/	1.00	x	6.50	/	6.50	x	1.00	/	1.00	=	4722.86 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(-	) x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
												6.50								5217.49 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
8	F4	-	(	39.06 plf	) x	25.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	488.31 lb
+	-	LINE4	(		4108.23 lb) x	1.00	x	1.00	/	1.00	x	6.50	/	6.50	x	1.00	/	1.00	=	4108.23 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(-	) x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
												6.50								4596.54 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
9	F4	-	(	39.06 plf	) x	25.00	x	1.00	/	2.00	x	6.50	/	6.50	x	1.00	/	1.00	=	488.31 lb
+	F5	-	(	8.93 plf	) x	5.50	x	27.75	/	25.00	x	6.50	/	6.50	x	1.00	/	1.00	=	54.51 lb
+	-	LINE5	(		3603.66 lb) x	1.00	x	1.00	/	1.00	x	6.50	/	6.50	x	1.00	/	1.00	=	3603.66 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(-	) x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
												6.50								4146.48 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		R DESIGN		R REQ'D		% LOAD EFFECT				
-	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
+	-			(-	) x			0.00	/	0.00	x	0.00	/	0.00	x	1.00	/	1.00	=	0.00 lb
												100.00								0.00 lb

REDUNDANCY CHECK:

IN LIEU OF CHECKING REDUNDANCY ON EACH SHEAR LINE, USE DEFAULT VALUE OF rho = 1.3

STRENGTH DESIGN SHEAR FORCE SUMMARY:

	LINE	FORCE		rho		V SEISMIC (STRENGTH)		φ		V SEISMIC (ASD)
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
ROOF	1	4722.86 lb	x	1.30	=	6139.72 lb	x	0.70	=	4297.80 lb
	2	4722.86 lb	x	1.30	=	6139.72 lb	x	0.70	=	4297.80 lb
	3	1719.72 lb	x	1.30	=	2235.64 lb	x	0.70	=	1564.95 lb
	4	4108.23 lb	x	1.30	=	5340.70 lb	x	0.70	=	3738.49 lb
	5	3603.66 lb	x	1.30	=	4684.75 lb	x	0.70	=	3279.33 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
2ND FLOOR	6	5217.49 lb	x	1.30	=	6782.73 lb	x	0.70	=	4747.91 lb
	7	5217.49 lb	x	1.30	=	6782.73 lb	x	0.70	=	4747.91 lb
	8	4596.54 lb	x	1.30	=	5975.50 lb	x	0.70	=	4182.85 lb
	9	4146.48 lb	x	1.30	=	5390.42 lb	x	0.70	=	3773.29 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb	x	1.30	=	0.00 lb	x	0.70	=	0.00 lb

DATE 7/7/2017
SHEET 10e

WIND LOAD DESIGN:

2016CBC / ASCE 7-10								
1609.6.4	**DIRECTIONAL PROCEDURE - MWFRS**							
	ENCLOSED BUILDING						ASCE 7-10 27.4.1	
26.5-1A	**BASIC WIND SPEEDS FOR RISK CATEGORY II BUILDINGS**							
1	BASIC WIND SPEED	V =	115.00 MPH				ASCE 7-10 FIG 26.5-1	
	WIND DIRECTIONALLY FACTOR	Kd =	0.85				ASCE 7-10 TABLE 26.6-1	
2	IMPORTANCE FACTOR, CATEGORY II	I =	1.00				ASCE 7-10 TABLE 6-1	

BUILDING DIRECTION - X								
3	EXPOSURE CATEGORY		C				ASCE 7-10 26.7.3	
			CASE 1				ASCE 7-10 TABLE 6-3	
	HEIGHT ABOVE GROUND LEVEL	z =	20.00 ft				MEAN ROOF HEIGHT	
	TERRAIN EXPOSURE CONSTRANT	α =	9.50				ASCE 7-10 TABLE 26.9-1	
	TERRAIN EXPOSURE CONSTRANT	zg =	900.00 ft				ASCE 7-10 TABLE 26.9-1	
	VELOCITY PRESSURE	Kz =	0.90				ASCE 7-10 TABLE 27.3-1	
4	TOPOGRAPHY FACTOR	Kzt =	1.000				ASCE 7-10 26.8.2	
5	VELOCITY PRESSURE	qz =	.00256 Kz Kzt Kd V^2 I				ASCE 7-10 (27.3-1)	
		qz =	25.954 psf				PRESSURE AT MEAN ROOF HEIGHT	
6	GUST FACTOR	G =	0.85				ASCE 7-10 26.9.1	
7	ENCLOSURE CLASSIFICATION		ENCLOSED				ASCE 7-10 26.10	
8	INTERNAL PRESSURE	GCpi =	0.18 + / -				ASCE 7-10 TABLE 26.11-1	
9	EXTERNAL PRESSURE COEFFICIENTS	L =	51.00				ASCE 7-10 FIG 27.4.1	
		B =	24.00				ASCE 7-10 FIG 27.4.1	
		L/B =	2.125				ASCE 7-10 FIG 27.4.1	
		h/L =	0.392				ASCE 7-10 FIG 27.4.1	
		Cp 1 =	0.800	WINDWARD WALL			ASCE 7-10 FIG 27.4.1	
		Cp 2 =	-0.294	LEEWARD WALL			ASCE 7-10 FIG 27.4.1	
		Cp 3 =	-0.700	SIDE WALLS			ASCE 7-10 FIG 27.4.1	
	PARALLEL TO RIDGE	Cp 4 =	-0.900	ROOF 1	0.00 deg		ASCE 7-10 FIG 27.4.1	PITCH 0.00 :12
		Cp 5 =	-0.900	ROOF 2	0.00 deg			PITCH 0.00 :12
10	DESIGN WIND LOAD	p =	qGCp - qi (Gcpi)				ASCE 7-10 TABLE 26.11-1	
	WINDWARD q = qz & qi = qh	p =	17.649 -	-4.672 =		22.321 psf		LATERAL FORCE
	LEEWARD q = qh	p =	-6.480 -	4.672 =		-11.152 psf		P WIND 33.47 psf
	SIDE WALLS qi = qh	p =	-15.443 -	4.672 =		-20.114 psf		
	ROOF 1 qi = qh	p =	-19.855 +	-4.672 =		-24.527 psf		WIND -24.53 psf
	ROOF 2 qi = qh	p =	-19.855 +	-4.672 =		-24.527 psf		LEE -24.53 psf
	DESIGN WIND LOAD x DIRECTION	p WALL =	22.32 -	-11.15 =		33.47 psf		UPLIFT FORCE
		p ROOF	0.00 -	0.00 =		0.00 psf		

BUILDING DIRECTION - Y								
3	EXPOSURE CATEGORY		C				ASCE 7-10 26.7.3	
			CASE 1				ASCE 7-10 TABLE 6-3	
	HEIGHT ABOVE GROUND LEVEL	z =	20.00 ft				MEAN ROOF HEIGHT	
	TERRAIN EXPOSURE CONSTRANT	α =	9.50				ASCE 7-10 TABLE 26.9-1	
	TERRAIN EXPOSURE CONSTRANT	zg =	900.00 ft				ASCE 7-10 TABLE 26.9-1	
	VELOCITY PRESSURE	Kz =	0.90				ASCE 7-10 TABLE 27.3-1	
4	TOPOGRAPHY FACTOR	Kzt =	1.000				ASCE 7-10 26.8.2	
5	VELOCITY PRESSURE	qz =	.00256 Kz Kzt Kd V^2 I				ASCE 7-10 (27.3-1)	
		qz =	25.954 psf				PRESSURE AT MEAN ROOF HEIGHT	
6	GUST FACTOR	G =	0.85				ASCE 7-10 26.9.1	
7	ENCLOSURE CLASSIFICATION		ENCLOSED				ASCE 7-10 26.10	
8	INTERNAL PRESSURE	GCpi =	0.18 + / -				ASCE 7-10 TABLE 26.11-1	
9	EXTERNAL PRESSURE COEFFICIENTS	L =	24.00				ASCE 7-10 FIG 27.4.1	
		B =	51.00				ASCE 7-10 FIG 27.4.1	
		L/B =	0.471				ASCE 7-10 FIG 27.4.1	
		h/L =	0.833				ASCE 7-10 FIG 27.4.1	
		Cp 1 =	0.800	WINDWARD WALL			ASCE 7-10 FIG 27.4.1	
		Cp 2 =	-0.500	LEEWARD WALL			ASCE 7-10 FIG 27.4.1	
		Cp 3 =	-0.700	SIDE WALLS			ASCE 7-10 FIG 27.4.1	
	NORMAL TO RIDGE (> 10 deg)	Cp 4 =	-0.900	ROOF 1	0.00 deg		ASCE 7-10 FIG 27.4.1	PITCH 0.00 :12
		Cp 4 =	-0.900	ROOF 2	0.00 deg			PITCH 0.00 :12
10	DESIGN WIND LOAD	p =	qGCp - qi (Gcpi)				ASCE 7-10 TABLE 26.11-1	
	WINDWARD q = qz & qi = qh	p =	17.649 -	-4.672 =		22.321 psf		LATERAL FORCE
	LEEWARD q = qh	p =	-11.030 -	4.672 =		-15.702 psf		P WIND 38.02 psf
	SIDE WALLS qi = qh	p =	-15.443 -	4.672 =		-20.114 psf		
	ROOF 1 qi = qh	p =	-19.855 +	-4.672 =		-24.527 psf		WIND -24.53 psf
	ROOF 2 qi = qh	p =	-19.855 +	-4.672 =		-24.527 psf		LEE -24.53 psf
	DESIGN WIND LOAD y DIRECTION	P WALL =	22.32 -	-15.70 =		38.02 psf		UPLIFT FORCE
		P ROOF	0.00 -	0.00 =		0.00 psf		

WIND FORCES:

	P#		WIND FORCE WALL	TRIB. HEIGHT	WIND FORCE ROOF		TRIB. HT.	LEE FORCE ROOF		TRIB. HT.	V WIND (STRENGTH)
- = 0.00ft	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
- = 0.00ft	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
ROOF = 10.00ft	1	2X	33.47 psf x	5.00 + (	-24.53 psf x	SIN 0.00 x	0.00 ft -	-24.53 psf x	SIN 0.00 x	0.00 ft) =	167.36 plf
	2	2Y	38.02 psf x	5.00 + (	-24.53 psf x	SIN 0.00 x	0.00 ft -	-24.53 psf x	SIN 0.00 x	0.00 ft) =	190.11 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
2ND FLOOR = 10.00ft	3	1X	33.47 psf x	10.00 + (	-24.53 psf x	SIN 0.00 x	0.00 ft -	-24.53 psf x	SIN 0.00 x	0.00 ft) =	334.73 plf
	4	1Y	38.02 psf x	10.00 + (	-24.53 psf x	SIN 0.00 x	0.00 ft -	-24.53 psf x	SIN 0.00 x	0.00 ft) =	380.23 plf
	5	1Y	38.02 psf x	10.00 + (	-24.53 psf x	SIN 0.00 x	0.00 ft -	-24.53 psf x	SIN 0.00 x	0.00 ft) =	380.23 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	7.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	7.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf
	-	-	0.00 psf x	0.00 + (	0.00 psf x	SIN 0.00 x	0.00 ft -	0.00 psf x	SIN 0.00 x	0.00 ft) =	0.00 plf

WIND FORCES TO SHEAR LINES:

2ND LEVEL ROOF

SHEAR LINE FORCES

LINE	F#	FORCE	WIDTH	X BAR	L	FACTORS		% LOAD EFFECT		
1	F1	167.36 plf	24.00	1.00	2.00	1.00	1.00	1.00	1.00	2008.36 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-			0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
										2008.36 lb

SHEAR LINE FORCES

LINE	F#	FORCE	WIDTH	X BAR	L	FACTORS		% LOAD EFFECT		
2	F1	167.36 plf	24.00	1.00	2.00	1.00	1.00	1.00	1.00	2008.36 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-			0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
										2008.36 lb

SHEAR LINE FORCES

LINE	F#	FORCE	WIDTH	X BAR	L	FACTORS		% LOAD EFFECT		
3	F2	190.11 plf	18.00	1.00	2.00	1.00	1.00	1.00	1.00	1711.02 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-			0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
										1711.02 lb

SHEAR LINE FORCES

LINE	F#	FORCE	WIDTH	X BAR	L	FACTORS		% LOAD EFFECT		
4	F2	190.11 plf	18.00	1.00	2.00	1.00	1.00	1.00	1.00	1711.02 lb
+	F2	190.11 plf	25.00	1.00	2.00	1.00	1.00	1.00	1.00	2376.42 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-			0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
										4087.45 lb

SHEAR LINE FORCES

LINE	F#	FORCE	WIDTH	X BAR	L	FACTORS		% LOAD EFFECT		
5	F2	190.11 plf	25.00	1.00	2.00	1.00	1.00	1.00	1.00	2376.42 lb
+	F2	190.11 plf	5.50	27.75	24.00	1.00	1.00	1.00	1.00	1209.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-		0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
+	-			0.00	0.00	1.00	1.00	1.00	1.00	0.00 lb
										3585.43 lb

WIND FORCES TO SHEAR LINES:

1ST LEVEL 2ND FLOOR

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		FACTORS				% LOAD EFFECT				
6	F3	-	(	334.73 plf	) x	24.00	x	1.00	/	2.00	x	1.00	/	1.00	x	1.00	/	1.00	=	4016.72 lb
+	-	F1	(	2008.36 lb	) x	1.00	x	1.00	/	1.00	x	1.00	/	1.00	x	1.00	/	1.00	=	2008.36 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-			(	) x			0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
																				6025.08 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		FACTORS				% LOAD EFFECT				
7	F3	-	(	334.73 plf	) x	24.00	x	1.00	/	2.00	x	1.00	/	1.00	x	1.00	/	1.00	=	4016.72 lb
+	-	F2	(	2008.36 lb	) x	1.00	x	1.00	/	1.00	x	1.00	/	1.00	x	1.00	/	1.00	=	2008.36 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-			(	) x			0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
																				6025.08 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		FACTORS				% LOAD EFFECT				
8	F4	-	(	380.23 plf	) x	25.00	x	1.00	/	2.00	x	1.00	/	1.00	x	1.00	/	1.00	=	4752.84 lb
+	-	F4	(	4087.45 lb	) x	1.00	x	1.00	/	1.00	x	1.00	/	1.00	x	1.00	/	1.00	=	4087.45 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-			(	) x			0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
																				8840.29 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		FACTORS				% LOAD EFFECT				
9	F4	-	(	380.23 plf	) x	25.00	x	1.00	/	2.00	x	1.00	/	1.00	x	1.00	/	1.00	=	4752.84 lb
+	F5	-	(	380.23 plf	) x	5.50	x	27.75	/	25.00	x	1.00	/	1.00	x	1.00	/	1.00	=	2321.29 lb
+	-	F5	(	3585.43 lb	) x	1.00	x	1.00	/	1.00	x	1.00	/	1.00	x	1.00	/	1.00	=	3585.43 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-			(	) x			0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
																				10659.56 lb

SHEAR LINE FORCES

LINE	F#			FORCE		WIDTH		X BAR		L		FACTORS				% LOAD EFFECT				
-	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
+	-			(	) x			0.00	/	0.00	x	1.00	/	1.00	x	1.00	/	1.00	=	0.00 lb
																				0.00 lb

WIND FORCE SUMMARY:

ALLOWABLE STRESS DESIGN SHEAR FORCE SUMMARY:

	LINE		V WIND (STRENGTH)		ϕ		V WIND (ASD)
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
ROOF	1	=	2008.36 lb	x	0.60	=	1205.02 lb
	2	=	2008.36 lb	x	0.60	=	1205.02 lb
	3	=	1711.02 lb	x	0.60	=	1026.61 lb
	4	=	4087.45 lb	x	0.60	=	2452.47 lb
	5	=	3585.43 lb	x	0.60	=	2151.26 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
2ND FLOOR	6	=	6025.08 lb	x	0.60	=	3615.05 lb
	7	=	6025.08 lb	x	0.60	=	3615.05 lb
	8	=	8840.29 lb	x	0.60	=	5304.17 lb
	9	=	10659.56 lb	x	0.60	=	6395.74 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb

2/3

Table 10: Allowable In-Plane Shear Strength (Pounds per Foot) for SIP Shear Walls (Seismic Loads in Seismic Design Categories A, B, C, D, E and F)[1,2]

Spline Type[3]	Framing Minimum SG[4]	Minimum Facing Connections[2]			Shear Strength[5] (plf)	
		Chord[2]	Plate[2]	Spline[3]		
Block, Surface, or Lumber Spline (Type S, Type L)	0.50	0.113"x 2-1/4" nails, 6" on center	0.113"x 2-1/4" nails, 3" on center	(7/16" thick, 3" wide spline) 0.113"x 2-1/4" nails, 6" on center	360	△S6
	0.50	0.113"x 2-1/4" nails, 6" on center	0.113"x 2-1/4" nails, 6" on center	(3/4" thick, 3" wide spline) 0.113"x 2-1/4" nails, 6" on center	360	
	0.50	0.113" x 2-3/8" nails, 3" on center Staggered (3/8" edge distance and 3/4" edge distance)	0.113" x 2-3/8" round head nails, 3" on center Staggered (3/8", 3/4" edge distance)	(23/32" thick, 3" wide spline) 0.113" x 2-3/8" nails, 3" on center Staggered (3/8" edge distance and 3/4" edge distance)	720	△S3
	0.50	0.113" x 2-3/8" nails, 2" on center Staggered (3/8" edge distance and 3/4" edge distance)	0.113" x 2-3/8" round head nails, 2" on center Staggered (3/8", 3/4" edge distance)	(23/32" thick, 3" wide spline) 0.113" x 2-3/8" nails, 2" on center Staggered (3/8" edge distance and 3/4" edge distance)	920	△S2

[1] Shear strength values, as published in this table, are limited to assemblies resisting wind or seismic forces where the aspect ratio (height:width) does not exceed 1:1 for Type S panel connections or 2:1 for Type L panel connections. (IM 014 ACU17)

[2] Chords, hold-downs and connections to other structural elements must be designed by a registered design professional in accordance with accepted engineering practice.

[3] Spline type at interior panel-to-panel joints only, solid chord members are required at each end of each shear wall segment.

[4] Required connections must be made on each side of the panel. Dimensional or engineered lumber shall have an equivalent specific gravity not less than specified.

[5] For design to resist seismic forces, shear wall height-width ratios greater than 2:1, but not exceeding 3.5:1, are permitted for assemblies using lumber splines provided the allowable shear strength values in this table are multiplied by 2w/h.

USE FOR DESIGN NEXT PAGES

SIP 6 = △6
SIP 3 = △3
SIP 2 = △2

2013CBC IBC SILL PLATE ANCHOR BOLT DESIGN:

ACI 318-14 CHAPTER 17				CAST-IN-PLACE ANCHOR RODS UNDER TENSION AND SHEAR ONLY - NO TENSION LOAD
	phi Vn	>	Vua	ACI 318-11 D.4.1.1 - MODERATE OR HIGH SEISMIC RISK REGION

APPLIED LOADS

→	Vs	=	1332.00 lb	O.K.	SERVICE LEVEL SHEAR FORCE BASED ON 0.7 ρE FROM A TIMBER SILL PLATE
	ρE	=	1902.86 lb		
SHEAR	Vua	=	1902.86 lb		STRENGTH LEVEL DESIGN FORCE TO ANCHORS
NOTE: TENSION	Nua	=	0.00 lb		NO TENSION LOAD IN SILL PLATE ATTACHMENT

ANCHOR BOLT AND CONCRETE

da	=	5/8 in	ANCHOR BOLT DIAMETER
n	=	1.00	NUMBER OF BOLTS IN GROUP
nt	=	6.00	THREADS PER INCH PER BOLT
f'c	=	2500.00 psi	CONCRETE COMPRESSIVE STRENGTH AT 28 DAYS
hef	=	5.00 in	EMBED DEPTH OF J PORTION OF ANCHOR BOLT LIMITED TO 8da
Ca1	=	1.75 in	EDGE DISTANCE TO FROM BOLT TO SIDE OF CONCRETE - PERPENDICULAR TO LOAD
spc	=	12.00 in	MINIMUM ANCHOR BOLT SPACING
Ca2	=	5.00 in	SPACE FROM BOLT CENTER TO EDGE OF CONCRETE IN THE DIRECTION OF THE LOAD

STEEL STRENGTH OF ANCHOR IN SHEAR

ACI 318-11 D.6.1	DUCTILE STEEL ELEMENT phi			A307 ANCHOR BOLTS REQUIRED
	phi	=	0.65	FOR SHEAR LOADS
	Vsa	=	n 0.6 Ase futa	CAST-IN HOOKED BOLT ANCHORS
	Ase	=	0.17 in^2	INDIVIDUAL ANCHOR ROD AREA = pi/4 x (da - 0.9743 / nt)^2
	futa	=	68400.00 psi	MIN OF 125 ksi vs. 1.9 x 36 ksi
	Vsa	=	6898.26 lb	ACI 318-11 (D-29)
	phi Vsa	=	4483.87 lb	

CONCRETE BREAKOUT STRENGTH OF ANCHOR IN SHEAR

ACI 318-11 D.6.2	Vcb	=	Avc / Avco Ψ_{ed},V $\Psi_{c,V}$ V_b	ACI 318-11 (D-30)
	phi	=	0.70	NO SUPPLEMENTARY REINFORCEMENT USED - CONDITION B
	Avc	=	13.78 in^2	Avc = 1.5 Ca1 x (1.5 Ca1 + 1.5 Ca1) WITH Ca2 > 1.5 Ca1
	Avco	=	13.78 in^2	Avco = 4.5 Ca1^2 PER ACI 318-11 (D-32)
	Vb	=	7 (le / do)^0.2 SQRT(do) SQRT(f'c) (Ca1)^1.5	
	le	=	5.00 in	le = hef FOR ANCHORS WITH CONSTANT STIFFNESS < 8do MAX.
	Ψ ed,V	=	1.00	SHEAR IS PARALLEL TO EDGE AND Ca2 > 1.5 Ca1 PER ACI 318-11
	Ψ c,V	=	1.40	CAST-IN ANCHORS IN UNCRACKED CONCRETE WITH REBAR PER ACI 318-11 D.6.2.7
	Vb	=	970.92 lb	ACI 318-11 (D-35)
	Vcb	=	1359.29 lb	ACI 318-11 (D-21)
NOTE:	2 Vcb	=	2718.57 lb	SHEAR PARALLEL TO EDGE PER ACI 318-11 D.6.2.1
	phi 2 Vcb	=	1903.00	

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

ACI 318-05 D.6.3	Vcp	=	kcp Ncb	ACI 318-11 (D-40)
	phi	=	0.70	NO SUPPLEMENTARY REINFORCEMENT USED - CONDITION B
	kcp	=	2.00	FOR hef > 2.5 in PER ACI 318-11 D.6.3.1
	Ncb	=	Anc / Anco $\Psi_{ed,N}$ $\Psi_{c,N}$ $\Psi_{cp,N}$ N_b	ACI 318-11 (D-3)
	Ca,min	=	1.75 in	MIN. DISTANCE TO EDGE OF CONCRETE ... Ca1 OR Ca2 FROM ABOVE
	Anc	=	138.75 in^2	Anc = (Ca1+ 1.5 hef) (2 x 1.5 hef) PER ACI 318-11 RD5.2.1 (b) WHERE Ca1 = Ca,min
	Anco	=	225.00 in^2	Anco = 9 hef^2 PER ACI 318-11 (D-5)
	Nb	=	kc SQRT (f'c) hef^1.5	ACI 318-11 (D-6)
	kc	=	24.00	CAST-IN ANCHOR FACTOR
	Nb	=	13416.41 lb	BASIC CRACKED CONCRETE BREAKOUT STRENGTH OF ANCHOR IN TENSION
	Ψ ed,N	=	0.77	EDGE MODIFICATION FACTOR w/ Ca,min < 1.5 hef PER ACI 318-11 D.5.2.5
	Ψ c,N	=	1.00	ASSUME CONCRETE IS CRACKED AT SERVICE LOAD LEVELS PER ACI 318-11 D.5.2.6
	Ψ cp,N	=	1.00	CAST-IN ANCHOR FACTOR PER ACI 318-11 D.5.2.7
	Ncb	=	6370.56 lb	ACI 318-11 (D-4)
	Vcp	=	12741.12 lb	ACI 318-11 (D-30)
	phi Vcp	=	8918.78 lb	

SHEAR CAPACITY SUMMARY

phi Vn MIN.	=	1903.00 lb	BREAKOUT CONTROLS DESIGN
phi Vn	=	1903.00 lb	ANCHOR BOLT STRENGTH ASSUMING TIMBER PLATE DUCTILITY > Vua ... O.K.

NOTE: ENSURE THAT THE SILL PLATE WILL YIELD BELOW THE STRENGTH LEVEL FORCE phi Vn

SHEAR WALL SCHEDULE ANCHOR VALUES:

BOLT / NAIL / VALUES - SEE TABLES BELOW FOR CALCULATIONS

1.) BOLT CAPACITIES:					
5/8" DIAMETER BOLTS IN CONCRETE, Vs	1332 lbs / BOLT IN CONCRETE NEAR FOUNDATION EDGE - 1-3/4" MIN. - ACI 318-14				
5/8" DIAMETER BOLTS IN CONCRETE, Vw	1554 lbs / BOLT IN CONCRETE NEAR FOUNDATION EDGE - 1-3/4" MIN. - ACI 318-14 - NORMALIZED FOR WIND (0.7E / 0.6W ASD FORCES)				
2x TIMBER AGAINST CONCRETE, Z' =	1488 lbs / BOLT - SINGLE SHEAR w/ CONCRETE SIDE - tm = 6" MIN NDS 2012 TABLE 11E w/ CD =	1.60	... USE MIN:	1,332 lbs S	1,488 lbs W
3x TIMBER AGAINST CONCRETE, Z' =	1888 lbs / BOLT - SINGLE SHEAR w/ CONCRETE SIDE - tm = 6" MIN NDS 2005 TABLE 11E w/ CD =	1.60	... USE MIN:	1,332 lbs S	1,554 lbs W
2.) NAIL CAPACITIES FOR 3/4" PLY:					
2x w/ 16d SILL NAIL CAPACITY =	145 lbs / NAIL - THROUGH 3/4" in THICK PLY - BASED ON REDUCED EMBEDMENT CAPACITIES &	1.60			
2x w/ 20d SILL NAIL CAPACITY =	207 lbs / NAIL - THROUGH 3/4" in THICK PLY - BASED ON REDUCED EMBEDMENT CAPACITIES &	1.60			
3x w/ 30d SILL NAIL CAPACITY =	150 lbs / NAIL - THROUGH 3/4" in THICK PLY - BASED ON REDUCED EMBEDMENT CAPACITIES &	1.60			
3.) NAIL CAPACITIES FOR 1-1/8" PLY:					
2x w/ 20d SILL NAIL CAPACITY =	118 lbs / NAIL - THROUGH 3/4" in THICK PLY - BASED ON REDUCED EMBEDMENT CAPACITIES &	1.60			
4.) LAG CAPACITIES:					
2x w/ 1/4" DIA. LAG x 4" L CAPACITY =	240 lbs / LAG - ts = 1-1/2" NDS 2012 TABLE 11J w/ CD =	1.60			
3x w/ 3/8" DIA. LAG x 6" L CAPACITY =	320 lbs / LAG - ts = 2-1/2" NDS 2012 TABLE 11J w/ CD =	1.60			
5.) SIMPSON SDS1/4 x 6" SCREWS =	560 lbs / SCREW - BASED ON 100% VALUES PER LARR 25711 & ICC-ES ESR-2236 x CD =	1.60			
6.) SIMPSON A35 CLIPS w/ 8d x 1-1/2 =	695 lbs / CLIP - BASED ON 160% VALUES FOR F1 ORIENTATION PER LARR 25716				
7.) SIMPSON LTP4 CLIPS =	670 lbs / CLIP - BASED ON 160% VALUES FOR ORIENTATION G PER LARR 25717				
8.) SIMPSON LTP5 CLIPS =	545 lbs / CLIP - BASED ON 160% VALUES FOR H ORIENTATION				

COMMON NAIL SINGLE SHEAR (Z) VALUES FOR SILL PLATES - 2005 NDS TABLE 11N (G=0.50) AND 3/4" PLY

NAIL d	NAIL dia.	REQ'D EMBED = 10 d	NAIL LENGTH	PLATE THK.	PLY THICKNESS	EMBED REMAINDER	DIAMTER CHECK	SHEAR VALUE	SHORT TERM	REDUCED
16.00	0.162 in	1.62 in	3.50 in	1.50 in	0.75 in	1.25 in	7.72	141.00 lbs	225.60 lbs	145.06 lbs
20.00	0.192 in	1.92 in	4.00 in	1.50 in	0.75 in	1.75 in	9.11	170.00 lbs	272.00 lbs	206.60 lbs
20.00	0.192 in	1.92 in	4.00 in	2.50 in	0.75 in	0.75 in	LESS THAN 6d	170.00 lbs	272.00 lbs	0.00 lbs
30.00	0.207 in	2.07 in	4.50 in	2.50 in	0.75 in	1.25 in	6.04	186.00 lbs	297.60 lbs	149.76 lbs

COMMON NAIL SINGLE SHEAR (Z) VALUES FOR SILL PLATES - 2005 NDS TABLE 11N (G=0.50) AND 1 1/8" PLY

NAIL d	NAIL dia.	REQ'D EMBED = 10 d	NAIL LENGTH	PLATE THK.	PLY THICKNESS	EMBED REMAINDER	DIAMTER CHECK	SHEAR VALUE	SHORT TERM	REDUCED
16.00	0.162 in	1.62 in	3.50 in	1.50 in	1.125 in	0.88 in	LESS THAN 6d	105.00 lbs	168.00 lbs	0.00 lbs
20.00	0.192 in	1.92 in	4.00 in	1.50 in	1.125 in	1.38 in	7.16	124.00 lbs	198.40 lbs	118.40 lbs
20.00	0.192 in	1.92 in	4.00 in	2.50 in	1.125 in	0.38 in	LESS THAN 6d	124.00 lbs	198.40 lbs	0.00 lbs
30.00	0.207 in	2.07 in	4.50 in	2.50 in	1.125 in	0.88 in	LESS THAN 6d	134.00 lbs	214.40 lbs	0.00 lbs

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SHEAR WALL SCHEDULE CALCULATIONS:

STRUCT I PLY OR STRUCT I OSB - MORE THAN 3 PLY REQUIRED
10d COMMON NAILS FOR SHEAR WALLS
SIMPSON SDS SCREWS AND SIMPSON CLIPS
LAG SCREWS PER NDS
A307 ANCHOR BOLTS x 7" MIN. EMBED
WIND LOAD RESISTANCE VALUES ARE 40% INCREASE OVER SEISMIC VALUES
ALL LOAD DURATION INCREASES FOR SEISMIC LOADS ARE LIMITED TO 1.60
ALL LOAD NAIL AND SCREW DURATION INCREASES FOR WIND LOADS ARE PER NDS 20(1.60
ALL SIMPSON CLIP LOAD DURATION INCREASES FOR WIND LOADS ARE LIMITED TO 1.60

SEISMIC RESISTANCE SHEAR WALL SUMMARY

TYPE	SEISMIC	SILL	SILL NAILS FOR UP TO 3/4" SHTG	SILL NAILS FOR UP TO 1-1/8" SHTG	SILL ATTACHMENT w/ SDS SCREWS	SILL ATTACHMENT w/ LAGS	A35 CLIPS	LTP4 CLIPS	LTP5 CLIPS	5/8" ANCHOR BOLTS
6	340 plf	2x	16d @ 5.12 in o.c.	20d @ 4.18 in o.c.	SDS 1/4 x 4-1/2 @ 19.76 in o.c.	1/4 x 4-1/2 @ 8.47 in o.c.	24.53 in	23.65 in	19.24 in	47.01 in
4	510 plf	3x	30d @ 3.52 in o.c.	-	SDS 1/4 x 6 @ 13.18 in o.c.	3/8 x 6 @ 7.53 in o.c.	16.35 in	15.76 in	12.82 in	31.34 in
3	665 plf	3x	30d @ 2.70 in o.c.	-	SDS 1/4 x 6 @ 10.11 in o.c.	3/8 x 6 @ 5.77 in o.c.	12.54 in	12.09 in	9.83 in	24.04 in
2	870 plf	3x	-	-	SDS 1/4 x 6 @ 7.72 in o.c.	3/8 x 6 @ 4.41 in o.c.	9.59 in	9.24 in	7.52 in	18.37 in
66	680 plf	3x	30d @ 2.64 in o.c.	-	SDS 1/4 x 6 @ 9.88 in o.c.	3/8 x 6 @ 5.65 in o.c.	12.26 in	11.82 in	9.62 in	23.51 in
44	1020 plf	3x	-	-	SDS 1/4 x 6 @ 6.59 in o.c.	3/8 x 6 @ 3.76 in o.c.	8.18 in	7.88 in	6.41 in	15.67 in
33	1330 plf	3x	-	-	SDS 1/4 x 6 @ 5.05 in o.c.	3/8 x 6 @ 2.89 in o.c.	6.27 in	6.05 in	4.92 in	12.02 in
22	1740 plf	3x	-	-	SDS 1/4 x 6 @ 3.86 in o.c.	-	4.79 in	4.62 in	3.76 in	9.19 in

WIND RESISTANCE SHEAR WALL SUMMARY

TYPE	WIND	SILL	SILL NAILS FOR UP TO 3/4" SHTG	SILL NAILS FOR UP TO 1-1/8" SHTG	SILL ATTACHMENT w/ SDS SCREWS	SILL ATTACHMENT w/ LAGS	A35 CLIPS	LTP4 CLIPS	LTP5 CLIPS	5/8" ANCHOR BOLTS
6	349 plf	2x	16d @ 4.99 in o.c.	20d @ 4.07 in o.c.	SDS 1/4 x 6 @ 19.26 in o.c.	1/4 x 4-1/2 @ 8.25 in o.c.	23.90 in	23.04 in	18.74 in	51.16 in
4	714 plf	3x	30d @ 2.52 in o.c.	-	SDS 1/4 x 6 @ 9.41 in o.c.	3/8 x 6 @ 5.38 in o.c.	11.68 in	11.26 in	9.16 in	26.12 in
3	931 plf	3x	-	-	SDS 1/4 x 6 @ 7.22 in o.c.	3/8 x 6 @ 4.12 in o.c.	8.96 in	8.64 in	7.02 in	20.03 in
2	1218 plf	3x	-	-	SDS 1/4 x 6 @ 5.52 in o.c.	3/8 x 6 @ 3.15 in o.c.	6.85 in	6.60 in	5.37 in	15.31 in
66	952 plf	3x	-	-	SDS 1/4 x 6 @ 7.06 in o.c.	3/8 x 6 @ 4.03 in o.c.	8.76 in	8.45 in	6.87 in	19.59 in
44	1428 plf	3x	-	-	SDS 1/4 x 6 @ 4.71 in o.c.	3/8 x 6 @ 2.69 in o.c.	5.84 in	5.63 in	4.58 in	13.06 in
33	1862 plf	3x	-	-	SDS 1/4 x 6 @ 3.61 in o.c.	3/8 x 6 @ 2.06 in o.c.	4.48 in	4.32 in	3.51 in	10.02 in
22	2436 plf	3x	-	-	SDS 1/4 x 6 @ 2.76 in o.c.	-	3.42 in	3.30 in	2.68 in	7.66 in

NOTE: VALUES IN TABLE NOT LISTED INDICATE REQUIRED FASTENER SPACING OF LESS THAN 2" ON CENTER AND ARE NOT RECOMMENDED
NOTE: WHERE SIMPSON CLIPS ARE LESS THAN 5" ON CENTER, CLIPS MAY BE PLACED ON BOTH SIDES OF THE SHEAR WALL WITH DOUBLE THE REQUIRED SPACING AND STAGGERED
COMBINE RESULTS AND CHOOSE SMALLEST SPACING FOR ANCHORAGE AND NAILS (TYPE 6 CAPACITY FOR WIND REDUCED TO LESS THAN 350 plf FOR TABLES) ... SEE NEXT PAGE FOR RESULTS

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SHEAR WALL SCHEDULE:

MARK	MATERIAL THICKNESS STRUCT 1 PLY., UNO	SHEAR WALL NAIL SIZE[1]	SPACING AT PLYWOOD EDGES[2]	SILL PLATE TO ANCHOR BOLTS	ANCHOR BOLT SPACING [3 & 4]	FRAMING @ ADJOINING EDGES [6]	SILL NAIL SPACING [5] FOR UP TO 3/4" THK. SHEATHING	SILL NAIL SPACING [5] FOR UP TO 1-1/8" THK. SHEATHING	SILL LAG SPACING (OPTION)	SDS1/4" x 6" SIMPSON WOOD SCREWS FOR SILL (OPTION) [9]	A35 CLIP SPACING - TOP PLATE TO FRMG. ABV. [10]	LTP4 CLIP SPACING - TOP PLATE TO FRMG. ABV. [10]	WIND SHEAR WALL CAPACITY	SEISMIC SHEAR WALL CAPACITY [8]
SINGLE SIDED SHEAR WALLS														
6	15/32" PLY	10d	6" o.c.	2x	5/8" in DIA. @ 47.0 in o.c.	2x	16d @ 5.0 in o.c.	20d @ 4.0 in o.c.	1/4 in DIA @ 8.0 in o.c.	18.0 in o.c.	18.00 in o.c.	20.0 in o.c.	349 plf	340 plf
4	15/32" PLY	10d	4" o.c.	3x	5/8" in DIA. @ 24.0 in o.c.	3x	30d @ 3.5 in o.c.	-	3/8 in DIA @ 5.0 in o.c.	9.0 in o.c.	8.00 in o.c.	10.0 in o.c.	714 plf	510 plf
3[6]	15/32" PLY	10d	3" o.c.	3x	5/8" in DIA. @ 16.0 in o.c.	3x	30d @ 2.7 in o.c.	-	3/8 in DIA @ 4.0 in o.c.	6.0 in o.c.	8.00 in o.c.	8.5 in o.c.	931 plf	665 plf
2[6]	15/32" PLY	10d	2" o.c.	3x	5/8" in DIA. @ 15.3 in o.c.	3x	-	-	3/8 in DIA @ 3.0 in o.c.	5.5 in o.c.	6.50 in o.c.	6.5 in o.c.	1218 plf	870 plf
DOUBLE SIDED SHEAR WALLS [7]														
66	15/32" PLY	10d	6" o.c.	3x	5/8" in DIA. @ 16.0 in o.c.	3x	30d @ 2.6 in o.c.	-	3/8 in DIA @ 4.0 in o.c.	7.0 in o.c.	8.50 in o.c.	8.0 in o.c.	952 plf	680 plf
44[6]	15/32" PLY	10d	4" o.c.	3x	5/8" in DIA. @ 12.0 in o.c.	3x	-	-	3/8 in DIA @ 2.5 in o.c.	4.5 in o.c.	5.50 in o.c.	5.5 in o.c.	1428 plf	1020 plf
33[6]	15/32" PLY	10d	3" o.c.	3x	5/8" in DIA. @ 8.0 in o.c.	3x	-	-	3/8 in DIA @ 2.0 in o.c.	3.5 in o.c.	4.00 in o.c.	4.0 in o.c.	1862 plf	1330 plf
22[6]	15/32" PLY	10d	2" o.c.	3x	5/8" in DIA. @ 7.7 in o.c.	3x	-	-	-	2.5 in o.c.	3.00 in o.c.	3.0 in o.c.	2436 plf	1740 plf

FOOTNOTES:

1.) ALL NAILS ARE TO BE COMMON NAILS WITH 10d HAVING A 1-5/8" MINIMUM PENETRATION INTO FRAMING AND 8d HAVING A 1-1/2" MINIMUM PENETRATION INTO FRAMING.
2.) ALL NAILS ARE TO HAVE 1/2" MINIMUM EDGE DISTANCE FROM PANEL ENDS AND EDGES. DO NOT BREAK SURFACE LAM OF PLY WITH NAIL HEAD.
3.) 5/8" DIAMETER ANCHOR BOLTS x 7 in MIN INTO CONCRETE FOOTINGS. NOTE: ADDITIONAL THREAD LENGTH IS REQUIRED AT 3x SILLS.
4.) 5/8" DIAMETER A36 THREADED RODS x 10 in MIN EMBED INTO SIMPSON SET-XP EPOXY IN CONCRETE FOOTINGS MAY BE USED FOR REPAIR AND RETROFIT UNDER SPECIAL INSPECTION
5.) USE 16d FOR 2x AND 30d FOR 3x - COMMON NAILS FOR CONNECTING PLATES TO JOISTS AND BLOCKING. USE 2x NOMINAL BLOCKING OR RIM MIN. FOR ALL 16d SILL NAILS AND 3x BLOCKING OR RIM FOR ALL 30d SILL NAILING. NAILS SHALL BE AT LEAST 1/2" FROM ALL EDGES OF SILL AND BLOCKING. WHERE MULTIPLE ROWS ARE REQUIRED, SPACE ROWS 1/2" MIN. AND TAKE CARE NOT TO SPLIT THE WOOD.
6.) MIN. 3x NOMINAL FRAMING SHALL BE USED AT ALL ADJOINING PANEL EDGES AT ALL WALLS WITH PLY TWO SIDES OR SINGLE SIDED WALL WITH 10d NAILS @ 3" o.c. OR LESS.
7.) WHERE PANELS ARE APPLIED TO BOTH SIDES OF THE STUDS, PANEL JOINTS SHALL BE OFFSET TO FALL ON DIFFERENT FRAMING MEMBERS OR FRAMING SHALL BE 3x NOMINAL AND ALL NAILS SHALL BE STAGGERED.
8.) LOAD VALUES ARE BASED ON CBC AND LABC TABLE 2306.4.1.
9.) SDS1/4x6" WOOD SCREWS BY SIMPSON SHALL COMPLY WITH LARR 25711 AND ICC-ES ESR-2236. NOTE: FOR 2x SILL AND 3/4" MAX. DIAPHRAGM SHEATHING, USE SDS1/4x4-1/2" O.K.
10.) WHERE CLIPS SPACING PREVENTS CLIPS ON A SINGLE SIDE FROM FITTING, ALTERNATE CLIPS ON EACH SIDE OF TOP PLATE TO RIM/BLOCKING ABOVE AND DOUBLE REQUIRED SPACING ABOVE FOR EACH LINE OF CLIPS.

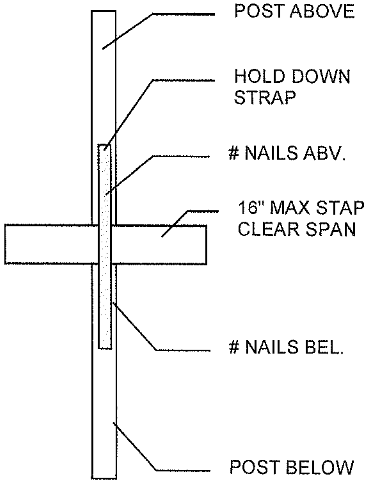
GENERAL NOTES:

A.) SHEAR WALL VALUES ARE FROM AWC NDS AND ASC SDPWS.
B.) ALL PLYWOOD IS TO BE STRUCTURAL I GRADE w/ (4) PLIES MINIMUM, AND ALL SHALL BE APPLIED DIRECTLY TO FRAMING.
C.) PLY MAY BE APPLIED EITHER VERTICALLY OR HORIZONTALLY ACROSS STUDS.
D.) WHERE STUDS ARE SPACED AT 16" o.c., INTERMEDIATE STUDS ARE TO BE NAILED AT 12" o.c. WHERE STUDS ARE SPACED FARTHER THAN 16" o.c., INTERMEDIATE STUDS ARE TO BE NAILED AT 6" o.c.
E.) ALL PLYWOOD JOINT NAILING AND SILL NAILING IS TO BE STAGGERED.
F.) ALL ANCHOR BOLTS ARE TO USE 3"x3"x.229" PLATE WASHERS w/ DIAGONALLY SLOTTED HOLES.
G.) ANCHOR BOLTS ARE TO BE INSTALLED INTO 2500 psi MINIMUM CONCRETE AT 28 DAYS.
H.) PRE-DRILL PILOT HOLES FOR SILL PLATE LAG SCREWS 40%-70% OF THREADED SHANK DIA. & FULL DIA. FOR SMOOTH SHANK PORTION. LAG INTO CENTERLINE OF BLOCK OR RIM BELOW PLY DIAPHRAGM.
J.) PROVIDE PREDRILLED HOLES 65% TO 75% OF THE NAIL DIAMETER FOR NAILS LARGER THAN 20d.
K.) STRUCTURAL OBSERVATION IS REQUIRED FOR ALL PANELS.

STRAP HOLD DOWN CAPACITY SCHEDULE:

UPLIFT CAPACITY IS BASED ON THE MINIMUM RATING x 0.75
SIMPSON STRAPS

HOLD DOWN	MFG RATING	POST	TEST RATING	LA REPORT	ICC REPORT	LARR 25910 25% REDUCTION	MAX DEFLECTION	NAIL SIZE	TOTAL NAILS
MST37	2710.00 lbs	4x	2710.00 lbs	RR25713	ESR-2105	2032.50 lbs	0.030	16d	22
MST48	4205.00 lbs	4x	4205.00 lbs	RR25713	ESR-2105	3153.75 lbs	0.030	16d	34
MST60	4605.00 lbs	4x	4605.00 lbs	RR25713	ESR-2105	3453.75 lbs	0.030	16d	34
MST72	6505.00 lbs	4x	6505.00 lbs	RR25713	ESR-2105	4878.75 lbs	0.030	16d	48
CMST12-42	10710.00 lbs	4x	9215.00 lbs	RR25713	ESR-2105	6911.25 lbs	0.030	16d	84
CMST14-33	7755.00 lbs	4x	6490.00 lbs	RR25713	ESR-2105	4867.50 lbs	0.030	16d	66
CS16-11	2080.00 lbs	4x	1705.00 lbs	RR25713	ESR-2105	1278.75 lbs	0.030	10d	22



MST STRAP CALCULATION				
STRAP	GAGE	NAIL CAPACITY	# NAILS	TOTAL CAP.
MST37	12	149.00	11.00	1639.00
MST48	12	149.00	17.00	2533.00
MST60	10	154.00	17.00	2618.00
MST72	10	154.00	24.00	3696.00

NAIL VALUES BASE ON 2012 NDS 11P - 16d w/ G=0.5

Ue

HOLD DOWN CAPACITY SCHEDULE:

HOLDOWN CAPACITIES BASED ON CODE TABLES AND RESEARCH REPORTS

SIMPSON HOLD DOWN CAPACITIES

SIMPSON HOLD DOWN	POST SIZE	ANCHOR DIAMETER	CATALOG CAPACITY lbs	da in	ICC REPORT	ICC CAPACITY lbs	ICC STRENGTH lbs	da in	LARR REPORT	75% CAP. lbs
HDU2-SDS2.5	4x4	0.625	3075.00	0.088	ICC-ESR 2330	3505.00	4907.00	0.088	25720	2628.75
HDU4-SDS2.5	4x4	0.625	4565.00	0.114	ICC-ESR 2330	4990.00	6986.00	0.114	25720	3742.50
HDU5-SDS2.5	4x4	0.625	5645.00	0.115	ICC-ESR 2330	5670.00	7938.00	0.115	25720	4252.50
HDQ8-SDS3	4x6	0.875	9230.00	0.095	ICC-ESR 2330	9230.00	12922.00	0.095	25720	6922.50
HDU11-SDS2.5	6x6	1.000	11175.00	0.137	ICC-ESR 2330	11175.00	15645.00	0.137	25270	8381.25
HHDQ14-SDS2.5	6x6	1.000	13710.00	0.107	ICC-ESR 2330	13710.00	19194.00	0.107	25270	10282.50
HD19	6x6	1.25	19070.00	0.137	ICC ES-0143	19070.00	26698.00	0.137	25828	19371.00
ZONE FOUR 48-9X	(2) 6x6	1.125	31174.00	0.032	ICC-ESR 5302	31174.00	43643.60	0.032	25334	31174.00

ANCHOR BOLTS IN 2,500 psi CONCRETE
TABLES FROM SIMPSON CATALOG C-2009

SIMPSON ANCHOR	ANCHOR BOLT	ANCHOR DIAMETER	CAPACITY lbs	de in	F in	ICC REPORT	ICC CAPACITY lbs	ICC STRENGTH lbs	LARR REPORT	LARR CAP lbs
SSTB16	5/8	0.625	4420.0	12.6	1.75	N.A.			25248	2695.00
SSTB20	5/8	0.625	4600.0	16.6	1.75	N.A.			25248	2987.00
SSTB24	5/8	0.625	5175.0	20.6	1.75	N.A.			25248	3360.00
SSTB28	7/8	0.875	10100.0	24.9	1.75	N.A.			25248	6558.00
	1	1.00	14120.0	10.0	15.00	N.A.			-	-
	1 1/8	1.13	45808.0	14.0	7.00	ICC-ESR 5302			25828	45808.00
	1 1/4	1.250	22580.0	14.0	21.00	N.A.			-	-

HOLD DOWN SUMMARY

SIMPSON HOLD DOWN	**POST** SIZE	**ANCHOR** LA CITY	**ANCHOR** ALTERNATE	**75% ASSEMBLY CAPACITY** lbs	**100% ASSEMBLY CAPACITY** lbs	**ds** in
HDU2-SDS2.5	4x4	SSTB16	5/8	2628.75	2695.00	0.1180
HDU4-SDS2.5	4x4	SSTB24	5/8	3360.00	3360.00	0.1540
HDU5-SDS2.5	4x4	SSTB24	5/8	3360.00	3360.00	0.1580
HDQ8-SDS3	4x6	SSTB28	7/8	6558.00	9230.00	0.1300
HDU11-SDS2.5	6x6	SEE ALT >	1	8381.25	11175.00	0.1820
HHDQ14-SDS2.5	6x8	SEE ALT >	1	10282.50	13710.00	0.1440
HD19	6x6	SEE ALT >	1 1/4	19371.00	25828.00	0.1855
ZONE FOUR 48-9X	(2) 6x6	SEE ALT >	1 1/8	31174.00	31174.00	0.032
				USE FOR UPLIFT DESIGN	USE FOR DEFLECTION DESIGN	

SEISMIC STRENGTH SHEAR FORCES TO SHEAR LINES AND ALLOWABLE STRESS SHEAR WALL FORCES:

ADJUST H/W < 3.5/1 USING 2w/h FOR CAPACITIES

LINE	V STRENGTH	A	B	C	D	E	F	G	H	LENGTH	v ASD = 0.7 V / L	TYPE	CAPACITY	H	W	H / W		2 W / H	ADJUSTED CAPACITY
1	6139.72 lb	20.00 ft +	14.00 ft +	ft +	ft +	ft +	ft +	ft +	ft =	34.00 ft	126.41 plf	SIP6	360.00	10.00	14.00	0.71			360.00
2	6139.72 lb	26.00 ft +	7.50 ft +	ft +	ft +	ft +	ft +	ft +	ft =	33.50 ft	128.29 plf	SIP6	360.00	10.00	7.50	1.33			360.00
3	2235.64 lb	3.50 ft +	4.50 ft +	ft +	ft +	ft +	ft +	ft +	ft =	8.00 ft	195.62 plf	SIP6	360.00	10.00	3.50	2.86	x	0.70	252.00
4	5340.70 lb	12.00 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	12.00 ft	311.54 plf	6	340.00 plf	10.00	12.00	0.83			340.00 plf
5	4684.75 lb	3.50 ft +	3.50 ft +	ft +	ft +	ft +	ft +	ft +	ft =	7.00 ft	468.48 plf	SIP2	920.00	10.00	3.50	2.86	x	0.70	644.00
6	6782.73 lb	6.50 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	6.50 ft	730.45 plf	SIP2	920.00	9.00	6.50	1.38			920.00
7	6782.73 lb	9.00 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	9.00 ft	527.55 plf	SIP3	720.00	9.00	9.00	1.00			720.00
8	5975.50 lb	12.00 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	12.00 ft	348.57 plf	4	510.00 plf	9.00	12.00	0.75			510.00 plf
9	5390.42 lb	3.50 ft +	3.50 ft +	ft +	ft +	ft +	ft +	ft +	ft =	7.00 ft	539.04 plf	SIP2	920.00	9.00	3.50	2.57	x	0.78	715.56
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-							

WIND SHEAR FORCES TO SHEAR LINES AND ALLOWABLE STRESS SHEAR WALL FORCES:

ALLOW FOR H/W < 3.5/1 PER CBC TA 2305.3.4

LINE	V ASD	A	B	C	D	E	F	G	H	LENGTH	v ASD = V / L	TYPE	CAPACITY	H	W	H / W < 3.5	ADJUSTED CAPACITY
1	1205.02 lb	20.00 ft +	14.00 ft +	ft +	ft +	ft +	ft +	ft +	ft =	34.00 ft	35.44 plf	SIP6	360.00	10.00	14.00	0.71	360.00 plf
2	1205.02 lb	26.00 ft +	7.50 ft +	ft +	ft +	ft +	ft +	ft +	ft =	33.50 ft	35.97 plf	SIP6	360.00	10.00	7.50	1.33	360.00 plf
3	1026.61 lb	3.50 ft +	4.50 ft +	ft +	ft +	ft +	ft +	ft +	ft =	8.00 ft	128.33 plf	SIP6	360.00	10.00	3.50	2.86	360.00 plf
4	2452.47 lb	12.00 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	12.00 ft	204.37 plf	6	349.00 plf	10.00	12.00	0.83	349.00 plf
5	2151.26 lb	3.50 ft +	3.50 ft +	ft +	ft +	ft +	ft +	ft +	ft =	7.00 ft	307.32 plf	SIP2	920.00	10.00	3.50	2.86	920.00 plf
6	3615.05 lb	6.50 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	6.50 ft	556.16 plf	SIP2	920.00	9.00	6.50	1.38	920.00 plf
7	3615.05 lb	9.00 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	9.00 ft	401.67 plf	SIP3	720.00	9.00	9.00	1.00	720.00 plf
8	5304.17 lb	12.00 ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	12.00 ft	442.01 plf	4	714.00 plf	9.00	12.00	0.75	714.00 plf
9	6395.74 lb	3.50 ft +	3.50 ft +	ft +	ft +	ft +	ft +	ft +	ft =	7.00 ft	913.68 plf	SIP2	920.00	9.00	3.50	2.57	920.00 plf
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					
	lb	ft +	ft +	ft +	ft +	ft +	ft +	ft +	ft =	ft	plf	-					

SEISMIC OVERTURNING DESIGN:

ASD LEVEL FORCES

USE EQUATION ASCE 7-11 12.4.2.3 #8

(0.6 - 0.14 Sds) D + 0.7 ρ Qe

Sds = 0.633

0.6 - 0.14 Sds = 0.511

USE THE LESSER OF ICC-ES AND LARR VALUES FOR HOLD DOWNS...CAPACITY IS BASED ON 0.75 x HOLD DOWN ALLOWABLE STRESS DESIGN CAPACITY

LINE #		LENGTH	HEIGHT	ADJUSTED ASD	OVERTURNING	0.7 ρ Qe	WEIGHT	ADD LOAD	ADD LOAD	(0.6-0.14Sds) D RST	OVERTURNING	+/- LEVER ARM	ADDED UPLIFT	WALL	NET UPLIFT	HOLDOWN	CAPACITY
#		ft	ft	plf	ft-lb	lb	plf	plf	lb	ft-lb	ft-lb	ft	lb	ABV.	lb	TYPE	lb
1	A	20.00	10.00	126.41	25281.2	1264.1	150.0	370.0		53187.0	-27905.8	-0.50		-	-1431.1	-	
1	B	14.00	10.00	126.41	17696.8	1264.1	150.0	100.0		12529.6	5167.2	-0.50		-	382.8	MST48	3153.8
2	A	26.00	10.00	128.29	33356.1	1282.9	150.0	100.0		43214.4	-9858.3	-0.50		-	-386.6	-	
2	B	7.50	10.00	128.29	9621.9	1282.9	150.0	100.0		3595.9	6026.1	-0.50		-	860.9	MST48	3153.8
3	A	3.50	10.00	195.62	6846.6	1956.2	150.0	50.0		626.5	6220.2	-0.50		-	2073.4	HDU2-SDS2.5	2628.8
3	B	4.50	10.00	195.62	8802.8	1956.2	150.0	50.0		1035.6	7767.2	-0.50		-	1941.8	HDU2-SDS2.5	2628.8
4	A	12.00	10.00	311.54	37384.9	3115.4	150.0	100.0		9205.4	28179.4	-0.50		-	2450.4	MST48	3153.8
5	A	3.50	10.00	468.48	16396.6	4684.8	150.0	100.0		783.1	15613.5	-0.50		-	5204.5	CMST12-45	6911.3
5	B	3.50	10.00	468.48	16396.6	4684.8	100.0	100.0		626.5	15770.2	-0.50		-	5256.7	CMST12-45	6911.3
6	A	6.50	9.00	604.04	35336.5	5436.4	100.0	100.0		2160.7	33175.7	-0.50	382.76	1B	5912.0	HDU11-SDS2.5	8381.3
7	A	9.00	9.00	399.25	32339.5	3593.3	100.0	100.0		4142.4	28197.1	-0.50	860.87	2B	4178.2	HDU11-SDS2.5	8381.3
8	A	12.00	9.00	37.03	3999.2	333.3	100.0	100.0		7364.3	-3365.1	-0.50	2450.39	4A	2157.8	HDU4-SDS2.5	3360.0
9	A	3.50	9.00	70.57	2222.8	635.1	100.0	100.0		626.5	1596.4	-0.50	5204.51	5A	5736.6	HHDQ14-SDS2.5	10282.5
9	B	3.50	9.00	70.57	2222.8	635.1	100.0	100.0		626.5	1596.4	-0.50	5256.72	5B	5788.8	HHDQ14-SDS2.5	10282.5
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RICHMOND HOFFMAYER INC.
STRUCTURAL ENGINEERING

DATE 7/7/2017
SHEET ______

WIND OVERTURNING DESIGN:

ASD LEVEL FORCES
USE EQUATION ASCE 7-11 2.4.1 #7
0.6D + W
USE THE LESSER OF ICC-ES AND LARR VALUES FOR HOLD DOWNS

LINE # #	LENGTH ft		HEIGHT ft	ASD FORCE plf	OVERTURNING ft-lb	W lb	WEIGHT plf	ADD LOAD plf	ADD LOAD lb	0.6 D RST ft-lb	OVERTURNING ft-lb	+/- LEVER ARM ft	ADDED UPLIFT lb	WALL ABV.	NET UPLIFT lb	HOLDOWN TYPE	CAPACITY lb
1	20.00	A	10.00	35.44	7088.33	354.42	150.00	370.0		62400.00	-55311.67	-0.50		-	-2836.50	-	
1	14.00	B	10.00	35.44	4961.83	354.42	150.00	100.0		14700.00	-9738.17	-0.50		-	-721.35	MST48	3153.75
2	26.00	A	10.00	35.97	9352.36	359.71	150.00	100.0		50700.00	-41347.64	-0.50		-	-1621.48	-	
2	7.50	B	10.00	35.97	2697.80	359.71	150.00	100.0		4218.75	-1520.95	-0.50		-	-217.28	MST48	3153.75
3	3.50	A	10.00	128.33	4491.44	1283.27	150.00	50.0		735.00	3756.44	-0.50		-	1252.15	HDU2-SDS2.5	2628.75
3	4.50	B	10.00	128.33	5774.71	1283.27	150.00	50.0		1215.00	4559.71	-0.50		-	1139.93	HDU2-SDS2.5	2628.75
4	12.00	A	10.00	204.37	24524.67	2043.72	150.00	100.0		10800.00	13724.67	-0.50		-	1193.45	MST48	3153.75
5	3.50	A	10.00	307.32	10756.28	3073.22	150.00	100.0		918.75	9837.53	-0.50		-	3279.18	CMST12-45	6911.25
5	3.50	B	10.00	307.32	10756.28	3073.22	150.00	100.0		918.75	9837.53	-0.50		-	3279.18	CMST12-45	6911.25
6	6.50	A	9.00	520.72	30462.10	4686.48	100.00	100.0		2535.00	27927.10	-0.50		1B	4654.52	HDU11-SDS2.5	8381.25
7	9.00	A	9.00	365.70	29621.82	3291.31	100.00	100.0		4860.00	24761.82	-0.50		2B	2913.15	HDU11-SDS2.5	8381.25
8	12.00	A	9.00	237.64	25665.36	2138.78	100.00	100.0		8640.00	17025.36	-0.50	1193.45	4A	2673.92	HDU4-SDS2.5	3360.00
9	3.50	A	9.00	606.35	19100.16	5457.19	100.00	100.0		735.00	18365.16	-0.50	3279.18	5A	9400.90	HHDQ14-SDS2.5	10282.50
9	3.50	B	9.00	606.35	19100.16	5457.19	100.00	100.0		735.00	18365.16	-0.50	3279.18	5B	9400.90	HHDQ14-SDS2.5	10282.50
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														-		-	
														-		-	

SHEAR WALL SEISMIC LOAD DEFLECTION CHECK - FLEXIBLE DIAPHRAGM ASSUMPTION:

STRENGTH LEVEL DESIGN

STRUCT I PLY - SPECIES GROUP 1, GRADE STESS LEVEL S-2 - E = 1,800,000 psi G = 90,000 psi & 19% MOISTURE CONTENT ASSUMED

I = 1.00
δ xe = (8vh^3 / Eab + vh/1000Ga + Σ da h/b)
δ x = Cd δxe / I
δ x' = (δx x 1.00)
Δa = 0.0250 hsx
ρ = 1.30 PER ASCE 7-10 12.3.4.1 ρ = 1.0 FOR DRIFT CALCULATIONS - ADJUST v ASD BY 1 / ρ

					STRENGTH			ASD		HOLDOWN		USE		VERTICAL ELONGATION									COMPARE	
LINE #	LENGTH ft	h ft	PLY TYPE	v ASD plf	1.4 v / ρ plf	HOLDOWN TYPE	A POST in^2	T HD / ρ lb	Ta HD lb	D/C RATIO	ds in	ds D/C in	Ga	ds SLIP in	ds CRUSH in	ds SHRINK in	8vh^3 /Eab in	vh/1000Ga in	Σ ds x h/b in	δ xe in	Cd	δ x' in	Δa in
1	20.00	10.00	SIP6	126.4	136.1	-		-1101					13.0	0.010	0.020	0.050		0.105	0.040	0.148	4.0	0.59	3.00
1	14.00	10.00	SIP6	126.4	136.1	MST48	12.25	294	3154	0.0934	0.030	0.003	13.0	0.010	0.020	0.050	0.004	0.105	0.059	0.171	4.0	0.68	3.00
2	26.00	10.00	SIP6	128.3	138.2	-		-297					13.0	0.010	0.020	0.050		0.106	0.031	0.141	4.0	0.56	3.00
2	7.50	10.00	SIP6	128.3	138.2	MST48	12.25	662	3154	0.2100	0.030	0.006	13.0	0.010	0.020	0.050	0.007	0.106	0.115	0.232	4.0	0.93	3.00
3	3.50	10.00	SIP6	195.6	210.7	HDU2-SDS2.5	12.25	1595	2629	0.6067	0.118	0.072	13.0	0.010	0.020	0.050	0.022	0.162	0.433	0.625	4.0	2.50	3.00
3	4.50	10.00	SIP6	195.6	210.7	HDU2-SDS2.5	12.25	1494	2629	0.5682	0.118	0.067	13.0	0.010	0.020	0.050	0.017	0.162	0.327	0.514	4.0	2.06	3.00
4	12.00	10.00	6	311.5	335.5	MST48	12.25	1885	3154	0.5977	0.030	0.018	19.2	0.010	0.020	0.050	0.010	0.175	0.082	0.533	4.0	2.13	3.00
5	3.50	10.00	SIP2	468.5	504.5	CMST12-45	12.25	4003	6911	0.5793	0.030	0.017	24.0	0.010	0.020	0.050	0.052	0.210	0.278	0.546	4.0	2.18	3.00
5	3.50	10.00	SIP2	468.5	504.5	CMST12-45	12.25	4003	6911	0.5793	0.030	0.017	24.0	0.010	0.020	0.050	0.052	0.210	0.278	0.546	4.0	2.18	3.00
6	6.50	9.00	SIP2	730.4	786.6	HDU11-SDS2.5	12.25	4548	8381	0.5426	0.182	0.099	24.0	0.010	0.020	0.050	0.032	0.295	0.248	0.585	4.0	2.34	2.70
7	9.00	9.00	SIP3	527.5	568.1	HDU11-SDS2.5	12.25	3214	8381	0.3835	0.182	0.070	19.0	0.010	0.020	0.050	0.017	0.269	0.150	0.448	4.0	1.79	2.70
8	12.00	9.00	4	348.6	375.4	HDU4-SDS2.5	12.25	1660	3360	0.4940	0.154	0.076	24.0	0.010	0.020	0.050	0.008	0.141	0.117	0.404	4.0	1.62	2.70
9	3.50	9.00	SIP2	539.0	580.5	HHDQ14-SDS2.5	39.88	4413	10283	0.4292	0.144	0.062	24.0	0.010	0.020	0.050	0.013	0.218	0.365	0.602	4.0	2.41	2.70
9	3.50	9.00	SIP2	539.0	580.5	HHDQ14-SDS2.5	39.88	4413	10283	0.4292	0.144	0.062	24.0	0.010	0.020	0.050	0.013	0.218	0.365	0.602	4.0	2.41	2.70
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SHEAR WALL SUMMARY:

COMPARE SEISMIC AND WIND REQUIREMENTS FOR FINAL RESULTS

LINE #	LENGTH ft	SEISMIC FORCE plf	WIND FORCE plf	GOVERNING FORCE plf	WALL TYPE PER SCHED.	HOLD DOWN SIMPSON	WALL CAPACITY plf	
1	20.0	126.4	35.4	126.4	SIP6	-	360.0	S
1	14.0	126.4	35.4	126.4	SIP6	MST48	360.0	S
2	26.0	128.3	36.0	128.3	SIP6	-	360.0	S
2	7.5	128.3	36.0	128.3	SIP6	MST48	360.0	S
3	3.5	195.6	128.3	195.6	SIP6	HDU2-SDS2.5	360.0	S
3	4.5	195.6	128.3	195.6	SIP6	HDU2-SDS2.5	360.0	S
4	12.0	311.5	204.4	311.5	6	MST48	340.0	S
5	3.5	468.5	307.3	468.5	SIP2	CMST12-45	920.0	S
5	3.5	468.5	307.3	468.5	SIP2	CMST12-45	920.0	S
6	6.5	730.4	556.2	730.4	SIP2	HDU11-SDS2.5	920.0	S
7	9.0	527.5	401.7	527.5	SIP3	HDU11-SDS2.5	720.0	S
8	12.0	348.6	442.0	442.0	4	HDU4-SDS2.5	714.0	W
9	3.5	539.0	913.7	913.7	SIP2	HHDQ14-SDS2.5	920.0	W
9	3.5	539.0	913.7	913.7	SIP2	HHDQ14-SDS2.5	920.0	W
						-		-
						-		-
						-		-
						-		-
						-		-
						-		-
						-		-
						-		-
						-		-
						-		-
						-		-
						-		-
						-		-
						-		-

OVERSTRENGTH LOAD SUMMARY

ASD LEVEL FORCES

USE EQUATION ASCE 7-11 12.4.3.2 #7

(0.6 - 0.14 Sds) D + Ωo Qe

Sds = 0.633

0.6 - 0.14 Sds = 0.511

Ωo = 3.00

LINE # #		LENGTH ft	HEIGHT ft	ADJUSTED V ASD plf	Ωo OVERTURNING ft-lb	Ωo Qe lb	WEIGHT plf	ADD LOAD plf	ADD LOAD lb	(0.6-0.14Sds) D RST ft-lb	NET O.T. ft-lb	+/- LEVER ARM ft	ADDED UPLIFT lb	WALL ABV.	OVERSTRENGTH (0.6 - 0.14 Sds) D + Ωo Qe lbs
1	A	20.00	10.00	126.41	58341.2	6731.7	150.0	370.0		53187.0	5154.3	-0.50		-	264.3
1	B	14.00	10.00	126.41	40838.9	6731.7	150.0	100.0		12529.6	28309.2	-0.50		-	2097.0
2	A	26.00	10.00	128.29	76975.6	6832.2	150.0	100.0		43214.4	33761.2	-0.50		-	1324.0
2	B	7.50	10.00	128.29	22204.5	6832.2	150.0	100.0		3595.9	18608.6	-0.50		-	2658.4
3	A	3.50	10.00	195.62	15800.0	10417.6	150.0	50.0		626.5	15173.5	-0.50		-	5057.8
3	B	4.50	10.00	195.62	20314.2	10417.6	150.0	50.0		1035.6	19278.6	-0.50		-	4819.7
4	A	12.00	10.00	311.54	86272.8	16590.9	150.0	100.0		9205.4	77067.4	-0.50		-	6701.5
5	A	3.50	10.00	468.48	37838.4	24948.4	150.0	100.0		783.1	37055.3	-0.50		-	12351.8
5	B	3.50	10.00	468.48	37838.4	24948.4	150.0	100.0		783.1	37055.3	-0.50		-	12351.8
6	A	6.50	9.00	604.04	81545.7	28951.1	100.0	100.0		2160.7	79384.9	-0.50	2096.98	1B	15327.8
7	A	9.00	9.00	399.25	74629.6	19135.8	100.0	100.0		4142.4	70487.2	-0.50	2658.37	2B	10951.0
8	A	12.00	9.00	37.03	9229.0	1774.8	100.0	100.0		7364.3	1864.7	-0.50	6701.51	4A	6863.7
9	A	3.50	9.00	70.57	5129.6	3382.2	100.0	100.0		626.5	4503.2	-0.50	12351.77	5A	13852.8
9	B	3.50	9.00	70.57	5129.6	3382.2	100.0	100.0		626.5	4503.2	-0.50	12351.77	5B	13852.8
														-	
														-	
														-	
														-	

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PROJECT VILLAGE NEST **JOB** 2017-0610
CLIENT -
ADDRESS EDEN, UT
DESIGN HUUM
ENGINEER J.H. **DATE** 6/14/2017
RICHMOND PE 44628 HOFFMAYER SE 3455 **SHEET** 227 **OF**

DIAPHRAGMS:

$F_{x\,ROOF} = \left.\begin{matrix}394\\191\end{matrix}\right\}$ 394 PLF ← SEISMIC STRENGTH

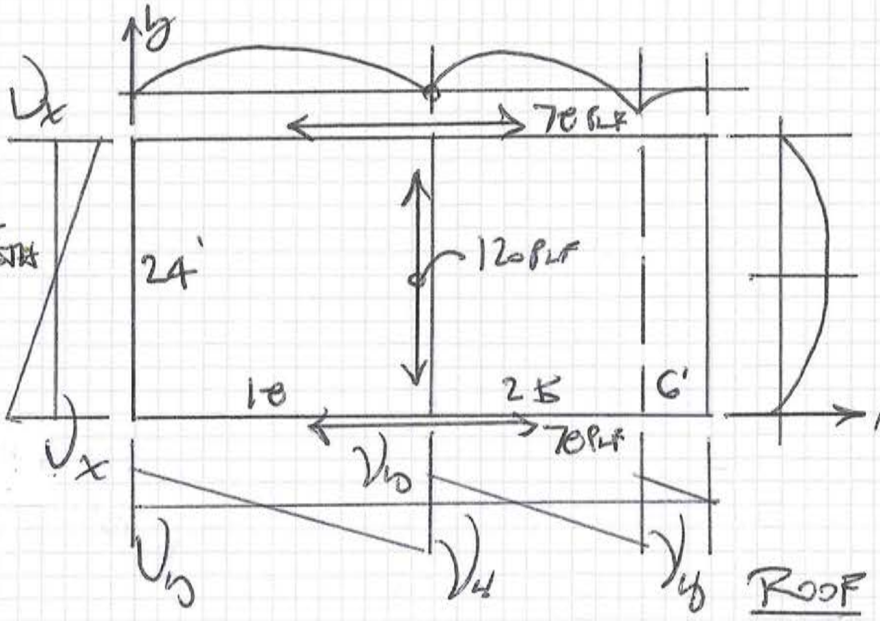
$F_{y\,ROOF} = \left.\begin{matrix}191\\191\end{matrix}\right\}$ 191 PLF

$V_{MAX_x} = 0.7(394)\frac{24}{2(43)}$

$= \underline{\underline{78\text{ PLF}}}$

$V_{MAX_y} = 0.7(191)\frac{43}{2(24)}$

$= \underline{\underline{120\text{ PLF}}}$ ROOF



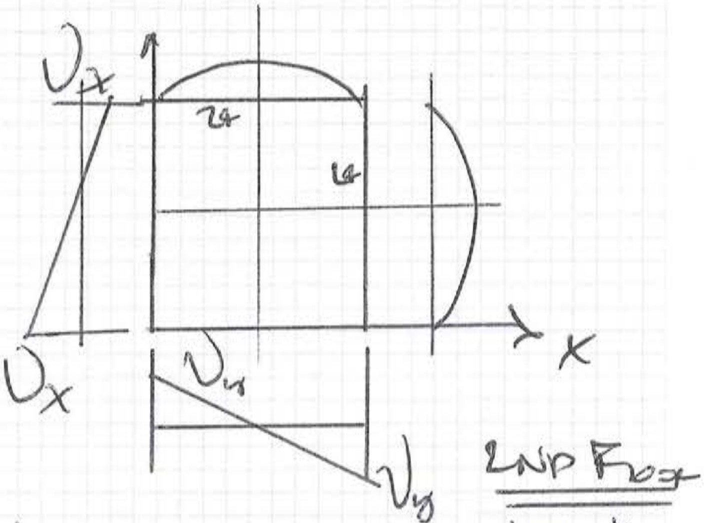
ROOF

WIND STRENGTH

$F_{x\,2ND} = \left.\begin{matrix}42\\332\end{matrix}\right\}$ 332 PLF

$F_{y\,2ND} = \left.\begin{matrix}39\\380\end{matrix}\right\}$ 380 PLF

$V_{MAX} = 0.6(380)\frac{24}{24\cdot 2} = \underline{\underline{114\text{ PLF}}}$
2ND FLOOR



2ND FLOOR

ROOF W/ SNOW REQUIRES PI 40/20 FOR 264 PSF & 16" SPANS

ROOF 5/8" CDX OR OSB PLY PI 40/20 T & G
W/ 10d @ 6", 12"

2ND FLOOR 5/8" STRUCT I OR THICKER T & G PI 48/24
W/ 10d @ 6, 12"

CHORDS & DRAGS:

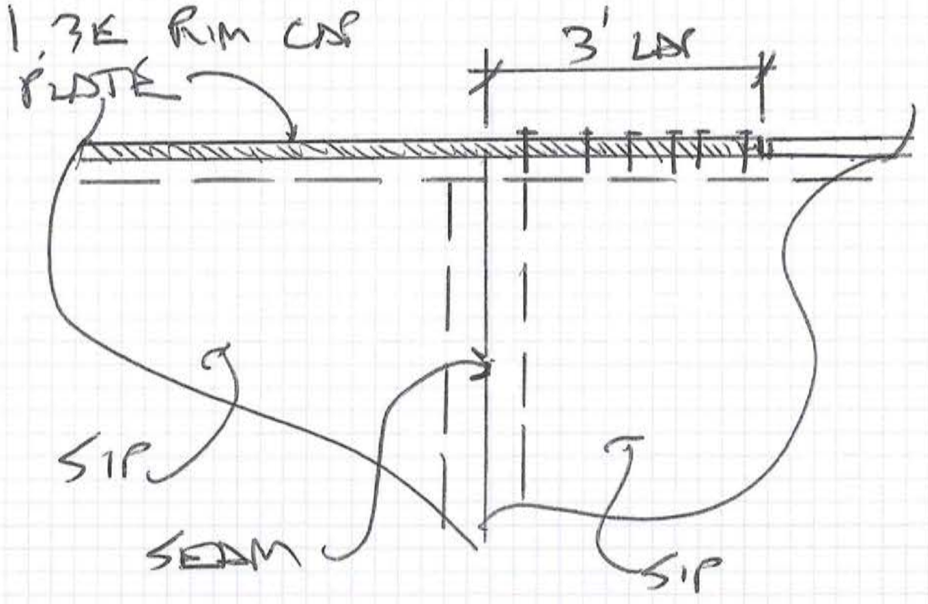
MAX CHORD ⇒ $\left(0.6(380)\overset{M_{MAX}}{24^2}/8\right)/24 = \underline{684\text{ lb}}$

USE [SINGLE 2x TOP PLATE
W/ 1.3E RIM BOARD CAP PLATE
SEE PG. #157]

LAP CAP PL W/ (2)-8d NAILS @ 6" O.C.

$Z_{\parallel\, NAIL} = 63\text{ lb}/\text{NAIL} \times 2 \times 12/6 = 252\text{ lb}/\text{ft LAP}$

LAP CAP PL $684/252 = 2.71 \rightarrow \underline{\underline{3'}}$



STAIR STRINGERS AND TREADS:

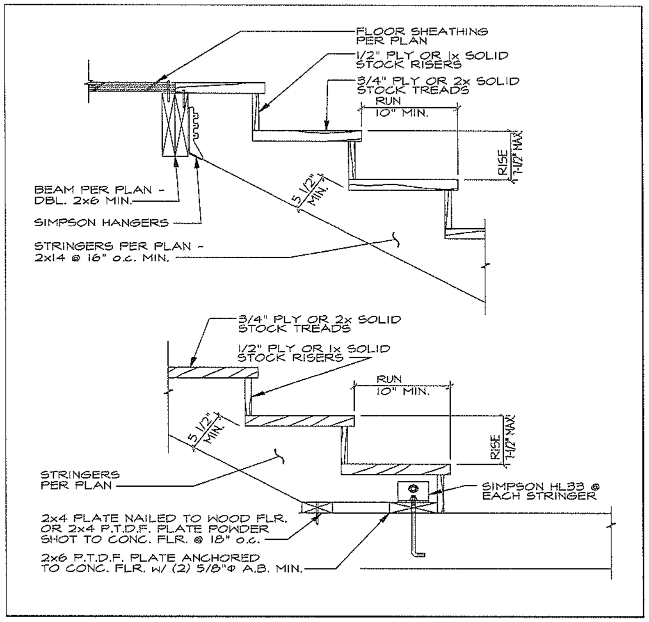
UNIFORM LOAD
WD = 10.00 psf DEAD LOAD
RISE = 7.50 in MAX. RISE
RUN 10.00 in MIN. RUN
WD' = 15.54 psf FACTORED DEAD LOAD FOR SLOPE
WL = 40.00 psf RESIDENTIAL LIVE LOAD
PL = 300.00 lbs PLACED ON TREADS FOR MAXIMUM STRESS

NOTCH = 6.00 in MAXIMUM NOTCH IN STRINGERS

DF#2 TIMBER PROPERTIES
Fv = 95.00 psi ALLOWABLE SHEAR STRESS
Fb = 870.00 psi ALLOWABLE BENDING STRESS
E = 1.60E+06 psi MODULUS OF ELASTICITY

LSL LUMBER PROPERTIES
Fv = 400.00 psi ALLOWABLE SHEAR STRESS
Fb = 1700.00 psi ALLOWABLE BENDING STRESS
E = 1.30E+06 psi MODULUS OF ELASTICITY

PSL LUMBER PROPERTIES
Fv = 290.00 psi ALLOWABLE SHEAR STRESS
Fb = 2900.00 psi ALLOWABLE BENDING STRESS
E = 2.00E+06 psi MODULUS OF ELASTICITY


FLOOR SHEATHING PER PLAN
1/2" PLY OR 1x SOLID STOCK RISERS
3/4" PLY OR 2x SOLID STOCK TREADS
RUN 10" MIN.
RISE 7-1/2" MAX
5 1/2" MIN.
BEAM PER PLAN - DBL. 2x6 MIN.
SIMPSON HANGERS
STRINGERS PER PLAN - 2x14 @ 16" o.c. MIN.
3/4" PLY OR 2x SOLID STOCK TREADS
1/2" PLY OR 1x SOLID STOCK RISERS
RUN 10" MIN.
RISE 7-1/2" MAX
5 1/2" MIN.
STRINGERS PER PLAN
SIMPSON HL33 @ EACH STRINGER
2x4 PLATE NAILED TO WOOD FLR. OR 2x4 P.T.D.F. PLATE POWDER SHOT TO CONC. FLR. @ 18" o.c.
2x6 P.T.D.F. PLATE ANCHORED TO CONC. FLR. W/ (2) 5/8"φ A.B. MIN.

STRINGER SIZE	Cf	F'b	WIDTH B	DEPTH D	D'	MOMENT CAPACITY	SHEAR CAPACITY
2x10 LSL	1.02	1736.93 psi	1.25	9.50	3.50	369.40 ft-lbs	1166.67 lbs
DBL 2x10 DF	1.10	957.00 psi	3.00	9.25	3.25	421.18 ft-lbs	2600.00 lbs
2x14 DF	0.90	783.00 psi	1.50	13.25	7.25	857.43 ft-lbs	2900.00 lbs
DBL 2x12 DF	1.00	870.00 psi	3.00	11.25	5.25	999.14 ft-lbs	4200.00 lbs
2x12 LSL	1.00	1701.64 psi	1.25	11.88	5.88	1019.67 ft-lbs	1958.33 lbs
4x12 DF	1.10	957.00 psi	3.50	11.25	5.25	1282.23 ft-lbs	4900.00 lbs
DBL 2x14 DF	0.90	783.00 psi	3.00	13.25	7.25	1714.85 ft-lbs	5800.00 lbs
2x14 LSL	0.99	1676.06 psi	1.25	14.00	8.00	1862.29 ft-lbs	2666.67 lbs
4x14 DF	1.00	870.00 psi	3.50	13.25	7.25	2222.96 ft-lbs	6766.67 lbs
4x12 PSL	1.00	2903.37 psi	3.50	11.88	5.88	4871.40 ft-lbs	5483.33 lbs
4x14 PSL	0.98	2850.80 psi	3.50	14.00	8.00	8869.16 ft-lbs	7466.67 lbs

STRINGER SPACING	TOTAL UNIF. LOAD	HORIZONTAL PROJECTED SPAN VS. STRIINGER MOMENT AND SHEAR	6.00 ft	7.00 ft	8.00 ft	9.00 ft	10.00 ft	11.00 ft	12.00 ft	13.00 ft	14.00 ft	15.00 ft	16.00 ft
12 in o.c.	55.5 plf	MOMENT	249.93	340.18	444.32	562.34	694.25	840.04	999.72	1173.28	1360.73	1562.06	1777.28
		SHEAR	166.62	194.39	222.16	249.93	277.70	305.47	333.24	361.01	388.78	416.55	444.32
		SIZE	2x10 LSL	2x10 LSL	2x14 DF	2x14 DF	2x14 DF	2x14 DF	2x12 LSL	4x12 DF	DBL 2x14 DF	DBL 2x14 DF	2x14 LSL
16 in o.c.	74.1 plf	MOMENT	333.24	453.58	592.43	749.79	925.67	1120.06	1332.96	1564.38	1814.31	2082.75	2369.71
		SHEAR	222.16	259.19	296.21	333.24	370.27	407.29	444.32	481.35	518.37	555.40	592.43
		SIZE	2x10 LSL	2x14 DF	2x14 DF	2x14 DF	DBL 2x12 DF	4x12 DF	DBL 2x14 DF	DBL 2x14 DF	2x14 LSL	4x14 DF	4x12 PSL
18 in o.c.	83.3 plf	MOMENT	374.89	510.27	666.48	843.51	1041.37	1260.06	1499.58	1759.92	2041.09	2343.09	2665.92
		SHEAR	249.93	291.58	333.24	374.89	416.55	458.20	499.86	541.51	583.17	624.82	666.48
		SIZE	DBL 2x10 DF	2x14 DF	2x14 DF	2x14 DF	4x12 DF	4x12 DF	DBL 2x14 DF	2x14 LSL	4x14 DF	4x12 PSL	4x12 PSL
24 in o.c.	111.1 plf	MOMENT	499.86	680.36	888.64	1124.68	1388.50	1680.08	1999.44	2346.56	2721.46	3124.12	3554.56
		SHEAR	333.24	388.78	444.32	499.86	555.40	610.94	666.48	722.02	777.56	833.10	888.64
		SIZE	2x14 DF	2x14 DF	DBL 2x12 DF	4x12 DF	DBL 2x14 DF	DBL 2x14 DF	4x14 DF	4x12 PSL	4x12 PSL	4x12 PSL	4x12 PSL
30 in o.c.	138.8 plf	MOMENT	624.82	850.46	1110.80	1405.86	1735.62	2100.11	2499.30	2933.21	3401.82	3905.16	4443.20
		SHEAR	416.55	485.97	555.40	624.82	694.25	763.67	833.10	902.52	971.95	1041.37	1110.80
		SIZE	2x14 DF	2x14 DF	4x12 DF	DBL 2x14 DF	2x14 LSL	4x14 DF	4x12 PSL	4x12 PSL	4x12 PSL	4x12 PSL	4x12 PSL
32 in o.c.	148.1 plf	MOMENT	666.48	907.15	1184.85	1499.58	1851.33	2240.11	2665.92	3128.75	3628.61	4165.50	4739.41
		SHEAR	444.32	518.37	592.43	666.48	740.53	814.59	888.64	962.69	1036.75	1110.80	1184.85
		SIZE	2x14 DF	DBL 2x12 DF	4x12 DF	DBL 2x14 DF	2x14 LSL	4x12 PSL	4x12 PSL	4x12 PSL	4x12 PSL	4x12 PSL	4x12 PSL
36 in o.c.	166.6 plf	MOMENT	749.79	1020.55	1332.96	1687.03	2082.75	2520.13	2999.16	3519.85	4082.19	4686.19	5331.84
		SHEAR	499.86	583.17	666.48	749.79	833.10	916.41	999.72	1083.03	1166.34	1249.65	1332.96
		SIZE	2x14 DF	4x12 DF	DBL 2x14 DF	DBL 2x14 DF	4x14 DF	4x12 PSL	4x12 PSL	4x12 PSL	4x12 PSL	4x12 PSL	4x14 PSL
48 in o.c.	222.2 plf	MOMENT	999.72	1360.73	1777.28	2249.37	2777.00	3360.17	3998.88	4693.13	5442.92	6248.25	7109.12
		SHEAR	666.48	777.56	888.64	999.72	1110.80	1221.88	1332.96	1444.04	1555.12	1666.20	1777.28
		SIZE	2x12 LSL	DBL 2x14 DF	2x14 LSL	4x12 PSL	4x12 PSL	4x12 PSL	4x12 PSL	4x12 PSL	4x14 PSL	4x14 PSL	4x14 PSL

USE 2x14 1.9E LVL @ 16" o.c. STRINGERS

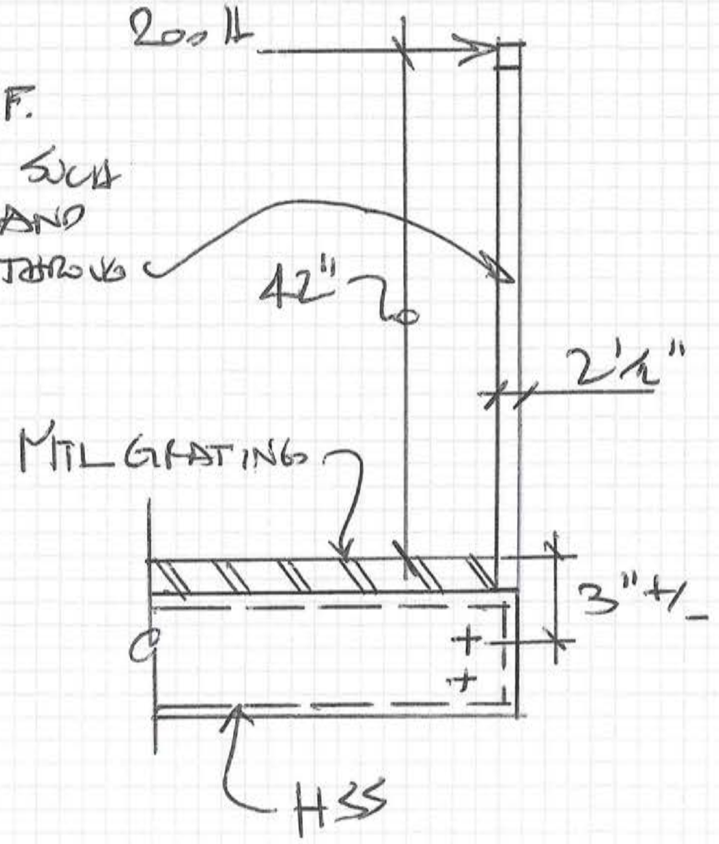
TREAD DESIGN
P = 300.00 lbs POINT LOAD - APPLY ANYWHERE ON TREAD
L = 48.00 in MAXIMUM TREAD SPAN
V MAX = 300.00 lbs MAXIMUM TREAD SHEAR - LOAD AT SIDE REACTION
M MAX = 3600.00 in-lbs MAXIMUM TREAD MOMENT - LOAD AT MID SPAN
A REQ'D = 4.74 in^2 DF #2
S REQ'D = 4.11 in^3 DF #2
TRY 2x8 MIN FLAT TREAD - TRUE SIZE = 2" x 8" MIN.
A = 18.13 in^2 O.K.
S = 4.83 in^3 O.K.

PROJECT VILLAGE NEST
JOB 2017-0610
CLIENT -
ADDRESS EDEN, UT
DESIGN HUUM
ENGINEER J.H.
RICHMOND PE 44628 HOFFMAYER SE 3455
DATE 6/14/2017
SHEET 23 OF

BALCONY GUARD:

RESIDENTIAL LOAD
200 lb AT 42" ABV. F.F.

CLOSED BALUSTRADE SUCH THAT A SPHERE 4"ϕ AND LARGER CAN NOT PASS THROUGH



$V = \underline{200\ lb}$

$M = 200 \times 45" = \underline{9000\ in\text{-}lb}$

TRY C1010 STEEL - $f_y = 16{,}800$ PSI

$S_{REQD} = 9000/16{,}800 = \underline{0.536\ in^3}$

USE TS $2\frac{1}{2} \times 1\frac{1}{2}$ C1010 MECH. WELDED TUBING - WELD OR BOLT TO HSS
$S = 0.593\ in^3$ AT 11 ga

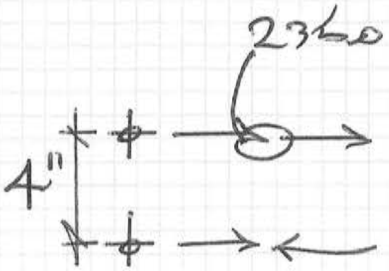
BOLT TO HSS 8 x 4 x 3/8 BEAMS

$F_B = 200/2 \pm 9000/4 =$

$= 100 \pm 2260$

$= 2360$ lb

& 2160 lb



DBL SHEAR 5/8"Ø M.B. A307

$r_n/\Omega_v = 7.36$ k/BOLT >> 2.36 k ✓

WELD:

$l = 4"$

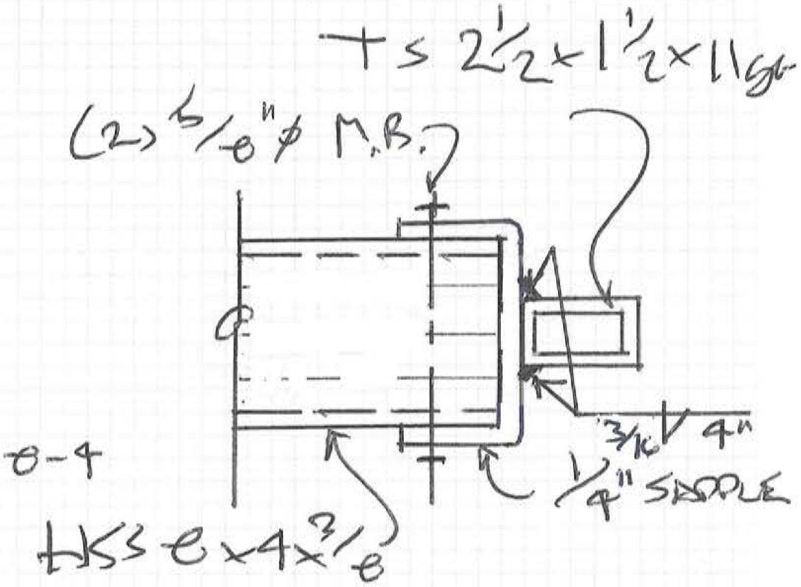
$e_x = al = 4.5"$

$a \geq 4.5/4 = 11.26$

$k = 0$

$C = 0.439$ → AISC TA. 8-4

$D_{MIN} = \frac{2(0.2k)}{0.439(1)4} = 0.23$



PLAN

NTS

USE [1/4" SADDLE
w/ (2) 5/8"Ø M.B. @ 4" o.c.
& 3/16" x 4" MIN FILLETS to TS

B3 & B4 TO B5 → FELDSPAR UVN

PER AISC TA10-9A

(2) 3/4" ϕ A325 BOLTS

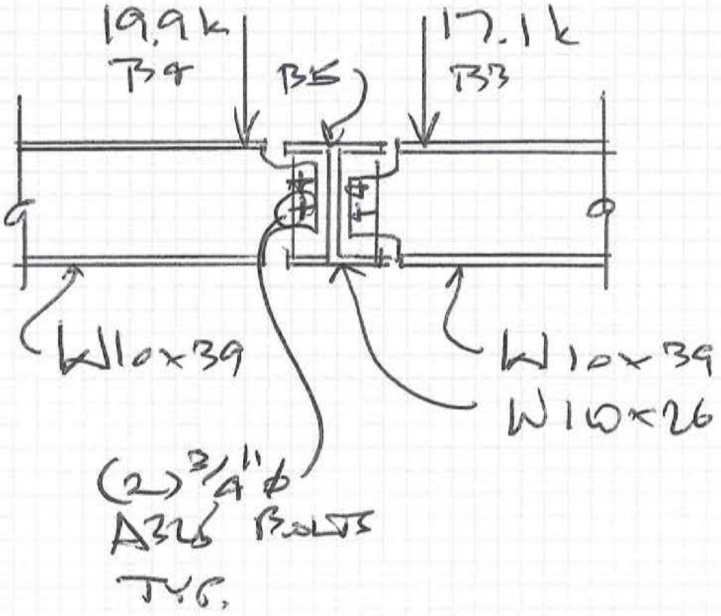
3/8" PL - N THREADS

CAP = 21.2 k ✓

L = 5" MIN OK

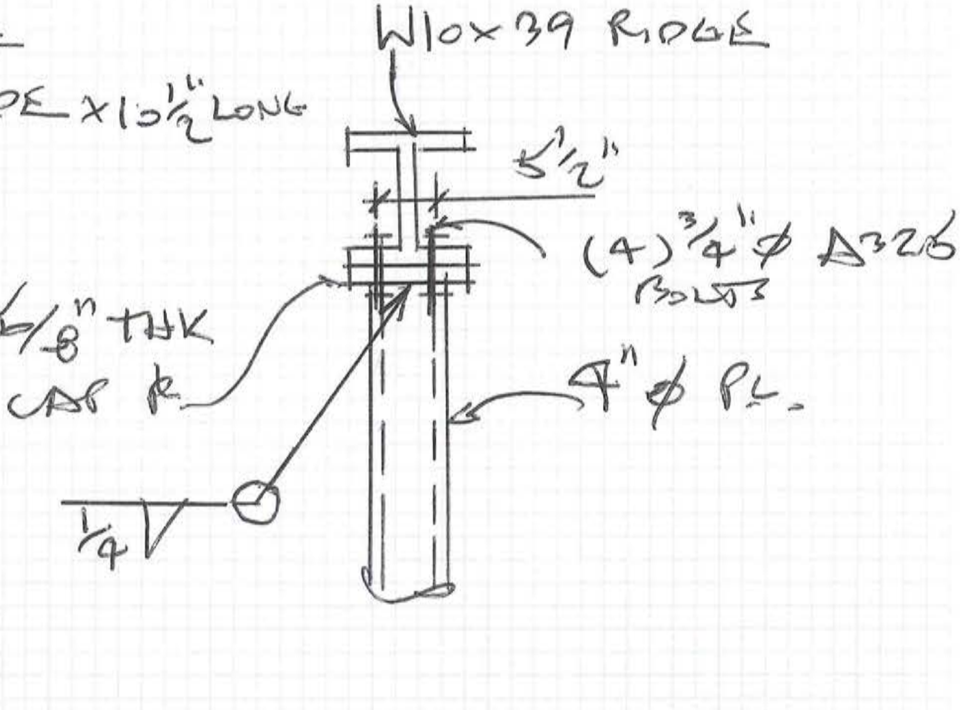
WELD = 5/8(3/8)

= 0.23 ≈ 1/4"


19.9k
B4
B5
17.1k
B3
W10x39
W10x39
W10x26
(2) 3/4"ϕ A325 BOLTS TYP.

RIDGE TO PL

CAP PL = 8" WIDE x 10 1/2" LONG x 5/8" THK


W10x39 RIDGE
5 1/2"
(4) 3/4"ϕ A325 BOLTS
5/8" THK CAP PL
4"ϕ PL.
1/4

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Steel Base Plate

Description : P.C. BASE PLATE RIDGE SUPPORT – MAX

Code References

Calculations per AISC Design Guide # 1, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

General Information

Material Properties

AISC Design Method	Load Resistance Factor Design	Φ c : LRFD Resistance Factor	0.60
Steel Plate Fy	= 36.0 ksi		
Concrete Support f'c	= 2.50 ksi		
Assumed Bearing Area :	Full Bearing	Allowable Bearing Fp per J8	4.250 ksi

Column & Plate

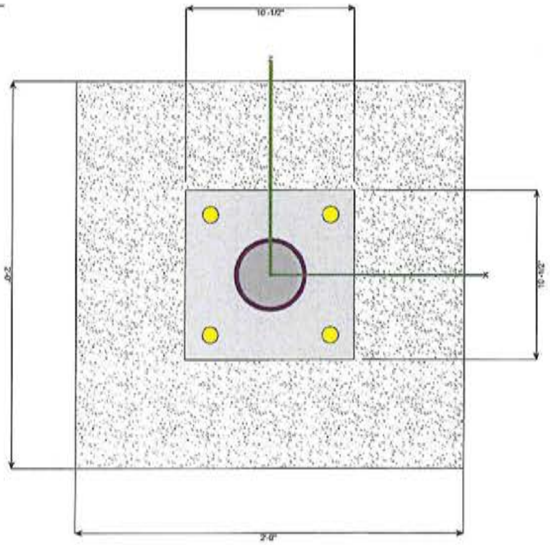
Column Properties

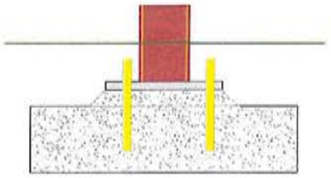
Steel Section : Pipe4STD

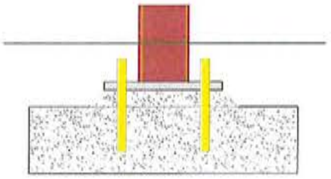
Depth	4.5 in	Area	2.96 in^2
Width	4.5 in	Ixx	6.82 in^4
Flange Thickness	0.221 in	Iyy	6.82 in^4
Web Thickness	0 in		

Plate Dimensions		Support Dimensions	
N : Length	10.50 in	Width along "X"	24.0 in
B : Width	10.50 in	Length along "Z"	24.0 in
Thickness	0.750 in		

Column assumed welded to base plate.



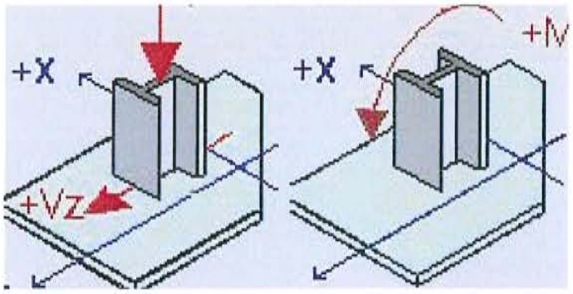




Applied Loads

	P-Y	V-Z	M-X
D : Dead Load	3.807 k	k	k-ft
L : Live	k	k	k-ft
Lr : Roof Live	4.015 k	k	k-ft
S : Snow	49.381 k	k	k-ft
W : Wind	k	k	k-ft
E : Earthquake	k	k	k-ft
H : Lateral Earth	k	k	k-ft

" P " = Gravity load, "+" sign is downward.
"+" Moments create higher soil pressure at +Z edge.
"+" Shears push plate towards +Z edge.



GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method	Load Resistance Factor Design
Governing Load Combination	+1.20D+1.60S
Governing Load Case Type	Axial Load Only
Design Plate Size	**10 -1/2" x 10 -1/2" x 0 -3/4"**
Pu : Axial	83.578 k
Mu : Moment	0.000 k-ft

Mu : Max. Moment	4.512 k-in
fb : Max. Bending Stress	32.082 ksi
Fb : Allowable : Fy * Phi	32.400 ksi
Bending Stress Ratio	0.990
	Bending Stress OK
fu : Max. Plate Bearing Stress	0.758 ksi
Fp : Allowable : min(0.85*f'c*sqrt(A2/A1), 1.7* f'c)*Phi	2.550 ksi
Bearing Stress Ratio	0.297
	Bearing Stress OK

Steel Base Plate

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Lic. # : KW-06002886

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Description : HSS BASE PLATE

Code References

Calculations per AISC Design Guide # 1, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

General Information

Material Properties

AISC Design Method: Load Resistance Factor Design
Steel Plate Fy = 36.0 ksi
Concrete Support f'c = 2.50 ksi
Assumed Bearing Area : **Full Bearing**

Φ_c : LRFD Resistance Factor 0.60
Allowable Bearing Fp per J8 4.250 ksi

Column & Plate

Column Properties

Steel Section : HSS6x6x3/8

Depth	6	in	Area	7.58	in^2
Width	6	in	Ixx	39.5	in^4
Flange Thickness	0.349	in	Iyy	39.5	in^4
Web Thickness		in			

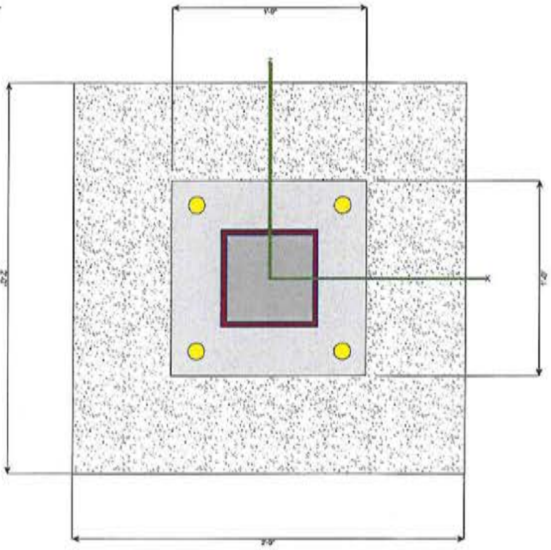
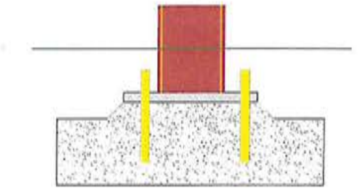


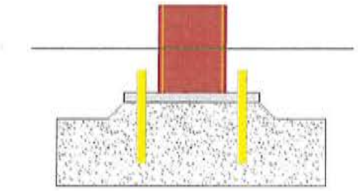
Plate Dimensions

N : Length 12.0 in
B : Width 12.0 in
Thickness 0.8750 in
Column assumed welded to base plate.

Support Dimensions

Width along "X" 24.0 in
Length along "Z" 24.0 in

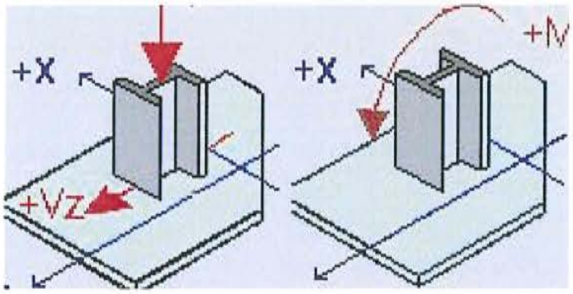




Applied Loads

	P-Y		V-Z		M-X	
D : Dead Load	9.050	k		k		k-ft
L : Live	7.440	k		k		k-ft
Lr : Roof Live	4.028	k		k		k-ft
S : Snow	90.670	k		k		k-ft
W : Wind		k		k		k-ft
E : Earthquake		k		k		k-ft
H : Lateral Earth		k		k		k-ft

" P " = Gravity load, "+" sign is downward.
"+" Moments create higher soil pressure at +Z edge.
"+" Shears push plate towards +Z edge.



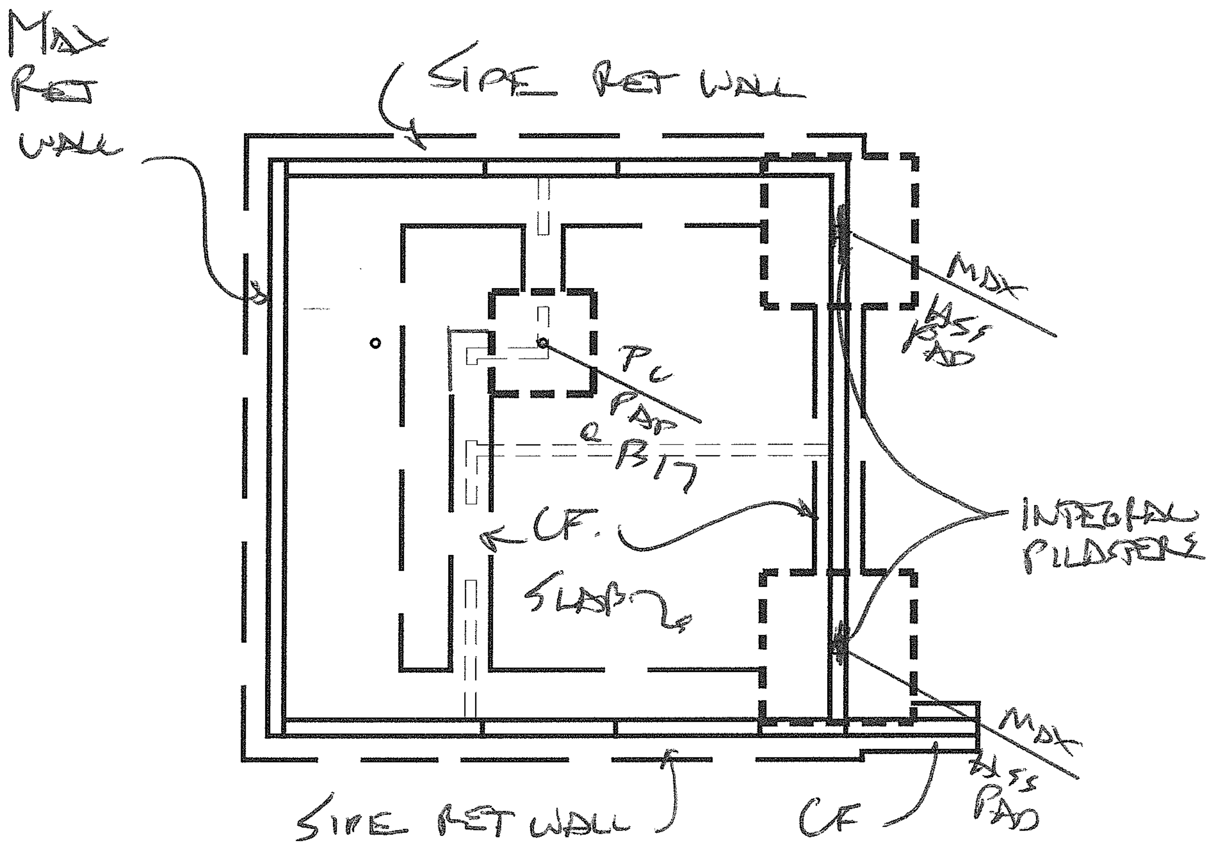
GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method	Load Resistance Factor Design
Governing Load Combination	+1.20D+0.50L+1.60S
Governing Load Case Type	Axial Load Only
Design Plate Size	**1'-0" x 1'-0" x 0 -7/8"**
Pu : Axial	159.652 k
Mu : Moment	0.000 k-ft

Mu : Max. Moment	5.501 k-in
fb : Max. Bending Stress	28.737 ksi
Fb : Allowable : Fy * Phi	32.400 ksi
Bending Stress Ratio	0.887
	Bending Stress OK
fu : Max. Plate Bearing Stress	1.109 ksi
Fp : Allowable : min(0.85*f'c*sqrt(A2/A1), 1.7* f'c)*Phi	2.550 ksi
Bearing Stress Ratio	0.435
	Bearing Stress OK

PROJECT FELDSPAR LOWER JOB 2017-0610
CLIENT VILLAGE NEST
ADDRESS EDEN, UTAH
DESIGN HUUM
ENGINEER J.H. DATE 6/16/2017
RICHMOND PE 49628 HOFFMAYER SE 3935 SHEET 23b OF

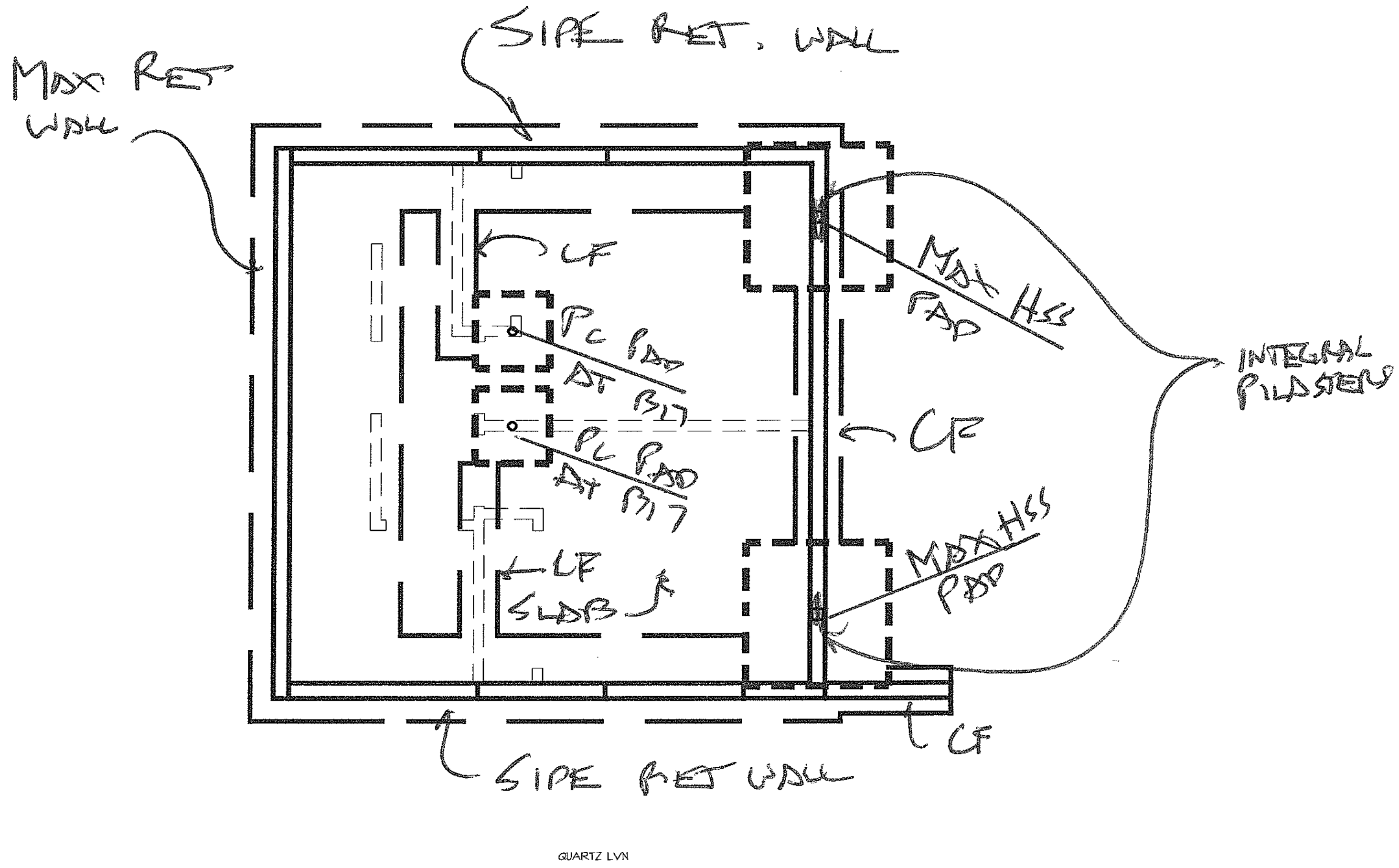

MAX
RET
WALL
SIDE RET WALL
MAX
BASS
PAD
PL
PAD
@ B17
CF.
INTEGRAL
PILASTERS
SLAB 4"
MAX
BASS
PAD
SIDE RET WALL
CF

FELDSPAR LVN

FOUNDATION PLAN

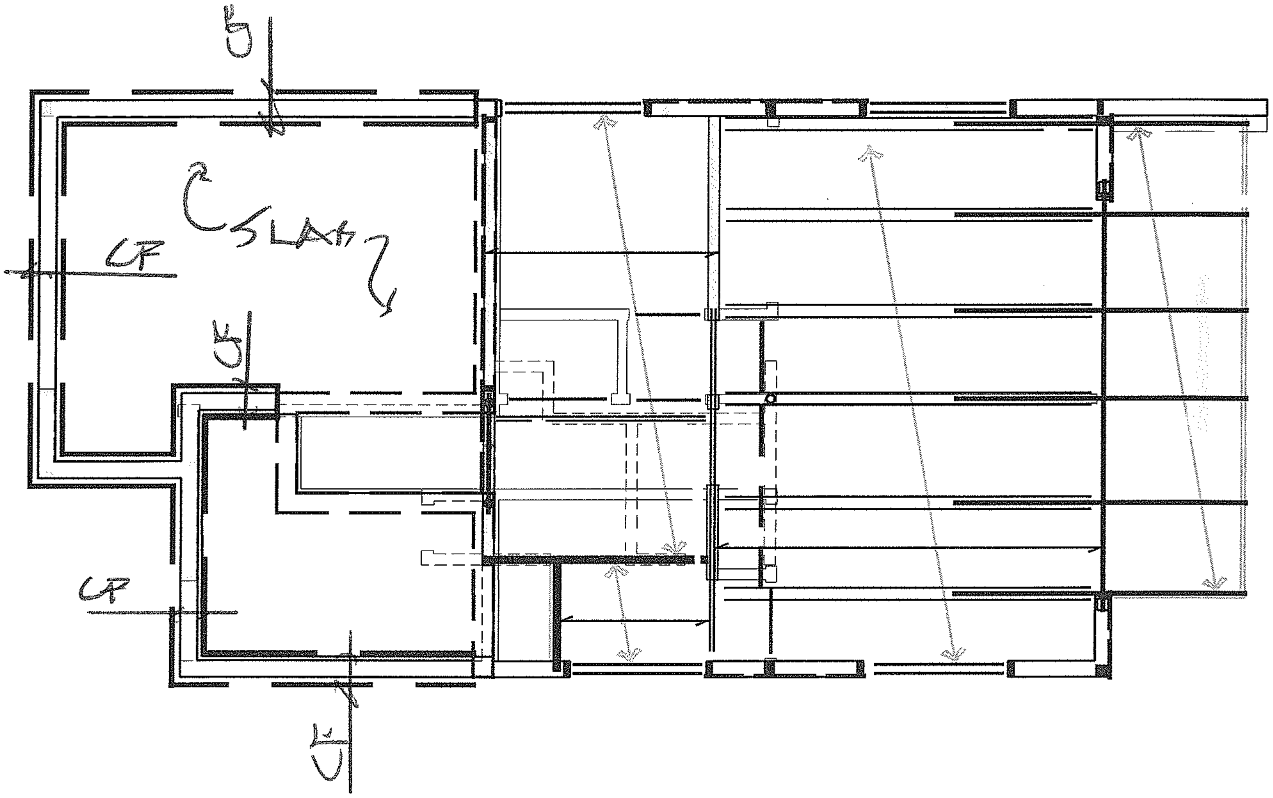
FELDSPAR LVN
N.T.S.

PROJECT FELDSPAR LOWER JOB 2017-0610
CLIENT VILLAGE NEST
ADDRESS EDEN, UTAH
DESIGN HUUM
ENGINEER J.H. DATE 6/16/2017
RICHMOND PE 49628 HOFFMAYER SE 3935 SHEET 236 OF


SIDE RET. WALL
MAX RET WALL
CF
PC PAD AT B17
PC PAD AT B17
MAX HSS PAD
INTEGRAL PILASTERS
CF
CF
SLAB
MAX HSS PAD
CF
SIDE RET WALL
QUARTZ LVN

FOUNDATION PLAN
QUARTZ LVN
N.T.S

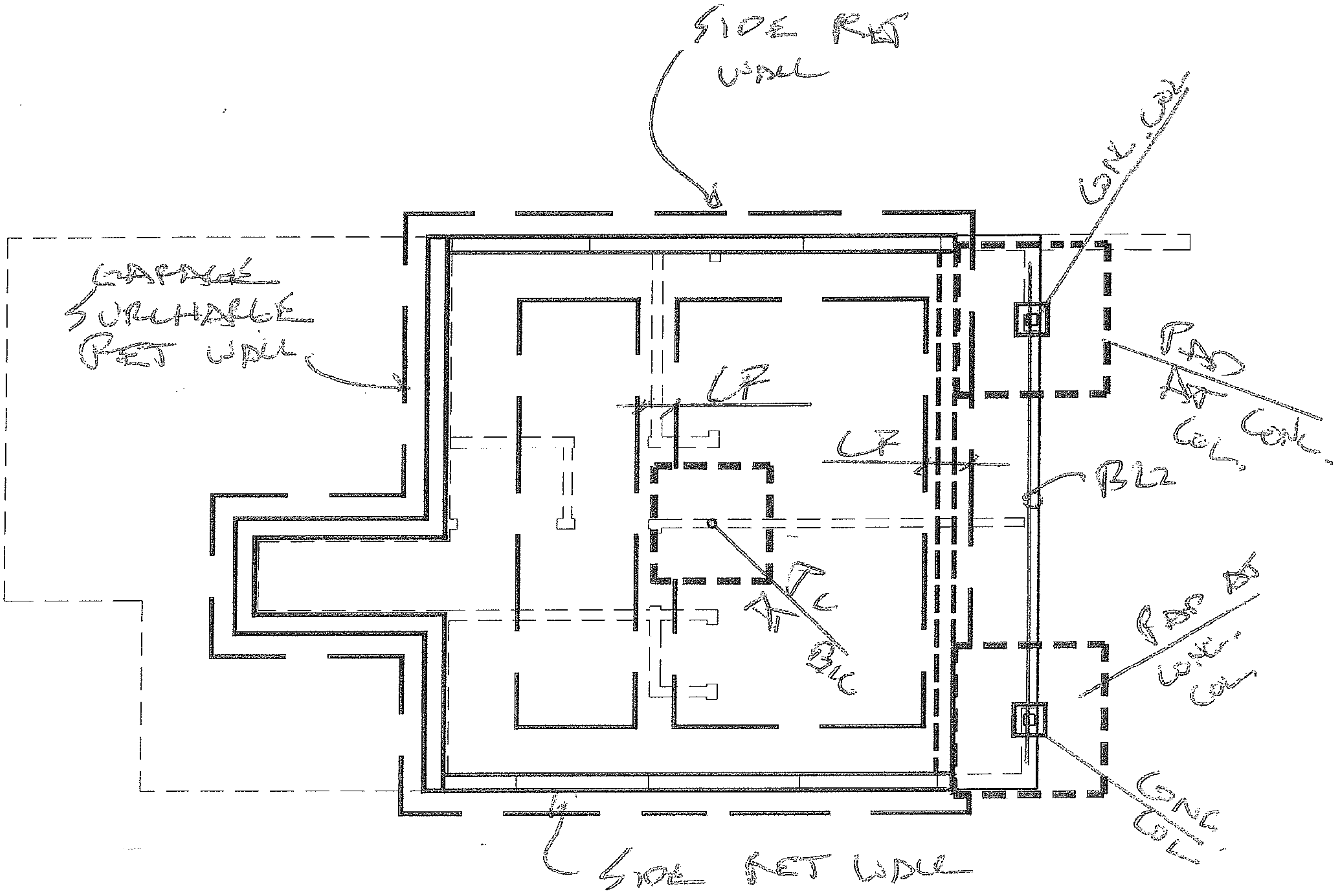
PROJECT FELDSPAR LOWER
JOB 2017-0610
CLIENT VILLAGE NEST
ADDRESS EDEN, UTAH
DESIGN HUUM
ENGINEER J.H.
RICHMOND PE 49628 HOFFMAYER SE 3935
DATE 6/16/2017
SHEET 237 OF



FELDSPAR LVN

FOUNDATION PLAN - MAIN

FELDSPAR LVN
N.T.S.


SIDE RET WALL
CONC. COL.
GARAGE SURCHARGE RET WALL
PAD AT CONC. COL.
LF
LF
B22
FTG AT B/C
PAD AT CONC. COL.
CONC. COL.
SIDE RET WALL

FELDSPAR UVN

LOWER FOUNDATION PLAN

FELDSPAR UVN
NTS

SOILS REPORT:

IGES 01628-003 NOV. 9, 2012
01628-015 DEC. 1, 2016

OVER EXCAVATE 24" BELOW FOOTING BOTTOM

FOOTINGS ON 24" STRUCTURAL FILL

Q_{ALLOW} = 2600 PSF BEARING CAPACITY

RECOMMENDED 42" BELOW LOWEST ADJACENT GRADE FOR FROST

MIN FOOTING = 20"

MAX FOOTING = 5' CONTINUOUS & 7' SQUARE

μ = 0.45 FRICTION

EFP = 40 PCF LEVEL & 64 PCF 2:1

PASSIVE = 360 PCF

PASSIVE REDUCED BY ½ WHEN USED WITH FRICTION

MIN. 5" SLAB W/ #4 @ 16" OC EACH WAY OVER 10 MIL VAPOR BARRIER ON GRAVEL ON APPROVED GRADE.

RICHMOND HOFFMAYER INC.
STRUCTURAL ENGINEERING

PROJECT VILLAGE NEST JOB 2017-0610
CLIENT -
ADDRESS EDEN, UT
DESIGN HUUM
ENGINEER J.H. DATE 6/14/2017
RICHMOND PE 44628 HOFFMAYER SE 3435 SHEET 240 OF

CONTINUOUS FOOTINGS:

$$\text{MAX LOAD} = (15.6 + 20 + 264)\,{}^{15'}\!/_{2} \quad \text{ROOF}$$
$$+\ 15.2 \times 20' \quad \text{WALLS}$$
$$+\ (14.3 + 40)\,1.33 \quad \text{2ND FLOOR}$$
$$W = \underline{\underline{2625\ \text{PLF}}}$$

20" WIDE × 42" BELOW GRADE

$$W_{CAP} = 2500\ \text{PSF} \times {}^{20}\!/_{12} = \underline{\underline{4167\ \text{PLF}}} \quad \checkmark$$

USE 20" WIDE × 42" BELOW GRADE
CONTINUOUS FOOTINGS
W/ #4 EACH WAY @ 24" o.c. – TYP.

Cantilevered Retaining Wall

File = d:\ENERCALC Projects\2017-0610.ec6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : MAX RETAINING WALL - FELDSPAR & QUARTZ LVN

Calculations per ACI 318-14, ACI 530-11, IBC 2015, CBC 2016, ASCE 7-10

Criteria

Retained Height	=	9.00 ft
Wall height above soil	=	1.00 ft
Slope Behind Wall	=	2.00 : 1
Height of Soil over Toe	=	30.00 in
Water height over heel	=	0.0 ft

Vertical component of active
Lateral soil pressure options:
USED for Soil Pressure.
USED for Sliding Resistance.
USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	2,500.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	64.0 psf/ft
Toe Active Pressure	=	40.0 psf/ft
Passive Pressure	=	360.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Friction Coeff btwn Ftg & Soil	=	0.450
Soil height to ignore for passive pressure	=	0.00 in

Surcharge Loads

Surcharge Over Heel = 0.0 psf
NOT Used To Resist Sliding & Overturning
Surcharge Over Toe = 0.0 psf
NOT Used for Sliding & Overturning

Lateral Load Applied to Stem

Lateral Load	=	0.0 plf
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Wind on Exposed Stem	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	600.0 lbs
Axial Live Load	=	2,000.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Wall Stability Ratios

Overturning	=	2.73 OK
Sliding	=	1.49 Ratio < 1.5!
Total Bearing Load	=	8,913 lbs
...resultant ecc.	=	3.18 in
Soil Pressure @ Toe	=	1,036 psf OK
Soil Pressure @ Heel	=	1,706 psf OK
Allowable	=	2,500 psf

Soil Pressure Less Than Allowable

ACI Factored @ Toe	=	1,089 psf
ACI Factored @ Heel	=	1,792 psf
Footing Shear @ Toe	=	27.2 psi OK
Footing Shear @ Heel	=	21.8 psi OK
Allowable	=	75.0 psi

Sliding Calcs (Vertical Component Used)

Lateral Sliding Force	=	3,311.1 lbs
less 50 % Passive Force	= -	1,822.5 lbs
less 100% Friction Force	= -	3,110.0 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	33.0 lbs NG

Load Factors

Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		Top Stem	2nd
		Stem OK	Stem OK
Design Height Above Ftg	ft =	4.00	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Thickness	in =	8.00	8.00
Rebar Size	=	# 5	# 5
Rebar Spacing	in =	12.00	6.00
Rebar Placed at	=	Edge	Edge
Design Data			
fb/FB + fa/Fa	=	0.263	0.806
Total Force @ Section	lbs =	1,280.0	3,947.2
Moment....Actual	ft-l =	2,133.3	12,274.9
Moment.....Allowable	ft-l =	8,121.3	15,222.0
Shear.....Actual	psi =	17.2	53.2
Shear.....Allowable	psi =	75.0	75.0
Wall Weight	psf =	100.0	100.0
Rebar Depth 'd'	in =	6.19	6.19
Lap splice if above	in =	23.40	23.40
Lap splice if below	in =	23.40	4.94
Hook embed into footing	in =	23.40	4.94
Concrete Data			
f'c	psi =	2,500.0	2,500.0
Fy	psi =	60,000.0	60,000.0

Project Title: VILLAGE NEST
Engineer: JJH
Project Descr: MULTI UNIT PROJECT

Project ID: 2017-0610
242

Printed: 11 JUL 2017, 3:33PM

Cantilevered Retaining Wall

File = d:\ENERCALC Projects\2017-0610.ec6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : MAX RETAINING WALL - FELDSPAR & QUARTZ LVN

Footing Dimensions & Strengths

Toe Width = 4.75 ft
Heel Width = 1.75
Total Footing Width = 6.50
Footing Thickness = 12.00 in
Key Width = 12.00 in
Key Depth = 12.00 in
Key Distance from Toe = 5.00 ft
f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	1,089	1,792 psf
Mu' : Upward	=	14,214	1,184 ft-lb
Mu' : Downward	=	5,753	3,912 ft-lb
Mu: Design	=	8,461	2,727 ft-lb
Actual 1-Way Shear	=	27.24	21.81 psi
Allow 1-Way Shear	=	75.00	75.00 psi
Toe Reinforcing	=	# 5 @ 8.00 in	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: #4@ 8.00 in, #5@ 12.25 in, #6@ 17.50 in, #7@ 23.75 in, #8@ 31.25 in, #9@ 39
Heel: #4@ 11.75 in, #5@ 18.25 in, #6@ 25.75 in, #7@ 35.25 in, #8@ 46.25 in, #9@ 4
Key: #4@ 12.50 in, #5@ 19.25 in, #6@ 27.25 in, #7@ 37.25 in,

Summary of Overturning & Resisting Forces & Moments

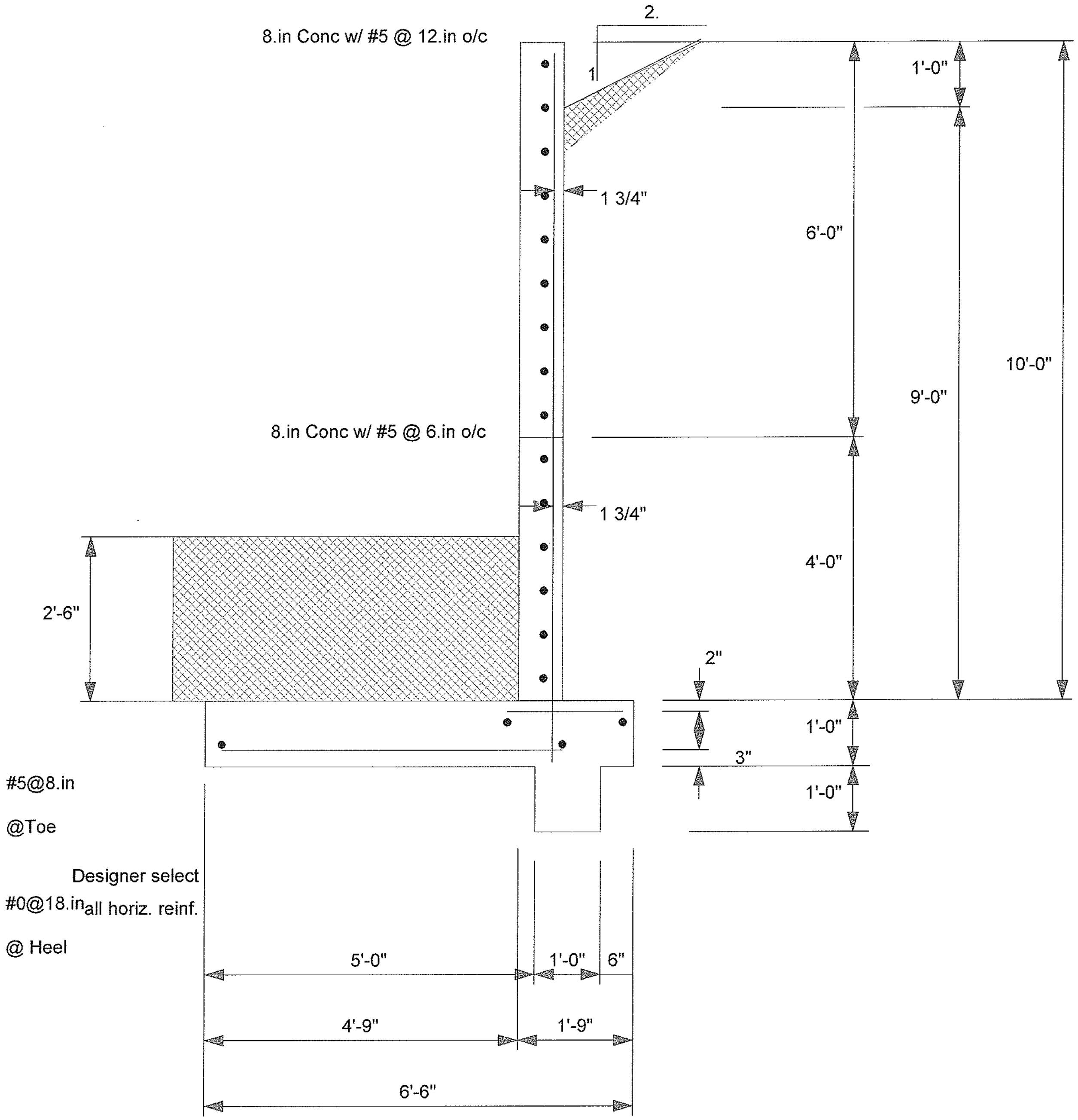
		.....OVERTURNING.....		
Item		Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	3,556.1	3.51	12,495.6
Surcharge over Heel	=			
Toe Active Pressure	=	-245.0	1.17	-285.8
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Total	=	3,311.1	**O.T.M.** =	12,209.8

Resisting/Overturning Ratio = **2.73**
Vertical Loads used for Soil Pressure = 8,913.4 lbs

Vertical component of active pressure used for soil pressure

		.....RESISTING.....		
		Force lbs	Distance ft	Moment ft-lb
Soil Over Heel	=	1,072.5	5.96	6,390.3
Sloped Soil Over Heel	=	32.3	6.14	198.1
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=	600.0	5.08	3,050.0
* Axial Live Load on Stem	=	2,000.0	5.08	10,166.7
Soil Over Toe	=	1,306.3	2.38	3,102.3
Surcharge Over Toe	=			
Stem Weight(s)	=	1,000.0	5.08	5,083.3
Earth @ Stem Transitions	=			
Footing Weight	=	975.0	3.25	3,168.8
Key Weight	=	150.0	5.50	825.0
Vert. Component	=	1,777.4	6.50	11,553.2
Total =		6,913.4 lbs	**R.M. =**	33,371.1

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.


2.
1
8.in Conc w/ #5 @ 12.in o/c
1'-0"
1 3/4"
6'-0"
10'-0"
9'-0"
8.in Conc w/ #5 @ 6.in o/c
1 3/4"
4'-0"
2'-6"
2"
1'-0"
3"
1'-0"
#5@8.in
@Toe
Designer select
#0@18.in all horiz. reinf.
@ Heel
5'-0"
1'-0"
6"
4'-9"
1'-9"
6'-6"

Cantilevered Retaining Wall

File = d:\ENERCALC Projects\2017-0610.ec6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29
Lic. # : KW-06002886
Licensee : RICHMOND HOFFMAYER

Description : MAX RETAINING WALL - FELDSPAR UVN

Calculations per ACI 318-14, ACI 530-11, IBC 2015, CBC 2016, ASCE 7-10

Criteria

Retained Height = 9.00 ft
Wall height above soil = 0.00 ft
Slope Behind Wall = 0.00 : 1
Height of Soil over Toe = 30.00 in
Water height over heel = 0.0 ft
Vertical component of active
Lateral soil pressure options:
USED for Soil Pressure.
USED for Sliding Resistance.
USED for Overturning Resistance.

Soil Data

Allow Soil Bearing = 2,500.0 psf
Equivalent Fluid Pressure Method
Heel Active Pressure = 40.0 psf/ft
Toe Active Pressure = 40.0 psf/ft
Passive Pressure = 360.0 psf/ft
Soil Density, Heel = 110.00 pcf
Soil Density, Toe = 110.00 pcf
Friction Coeff btwn Ftg & Soil = 0.450
Soil height to ignore for passive pressure = 0.00 in

Surcharge Loads

Surcharge Over Heel = 100.0 psf
Used To Resist Sliding & Overturning
Surcharge Over Toe = 0.0 psf
NOT Used for Sliding & Overturning

Lateral Load Applied to Stem

Lateral Load = 0.0 plf
...Height to Top = 0.00 ft
...Height to Bottom = 0.00 ft

Wind on Exposed Stem = 0.0 psf

Adjacent Footing Load

Adjacent Footing Load = 0.0 lbs
Footing Width = 0.00 ft
Eccentricity = 0.00 in
Wall to Ftg CL Dist = 0.00 ft
Footing Type Line Load
Base Above/Below Soil at Back of Wall = 0.0 ft
Poisson's Ratio = 0.300

Axial Load Applied to Stem

Axial Dead Load = 200.0 lbs
Axial Live Load = 200.0 lbs
Axial Load Eccentricity = 0.0 in

Design Summary

Wall Stability Ratios
Overturning = 1.59 OK
Sliding = 1.73 OK

Total Bearing Load = 4,275 lbs
...resultant ecc. = 14.35 in

Soil Pressure @ Toe = 2,185 psf OK
Soil Pressure @ Heel = 0 psf OK
Allowable = 2,500 psf
Soil Pressure Less Than Allowable
ACI Factored @ Toe = 2,662 psf
ACI Factored @ Heel = 0 psf
Footing Shear @ Toe = 29.3 psi OK
Footing Shear @ Heel = 14.5 psi OK
Allowable = 75.0 psi

Sliding Calcs (Vertical Component Used)
Lateral Sliding Force = 2,118.6 lbs
less 50 % Passive Force = - 1,822.5 lbs
less 100% Friction Force = - 1,830.6 lbs
Added Force Req'd = 0.0 lbs OK
....for 1.5 : 1 Stability = 0.0 lbs OK

Load Factors

Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		Top Stem	2nd
		Stem OK	Stem OK
Design Height Above Ftg	ft =	4.00	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Thickness	in =	8.00	8.00
Rebar Size	=	# 5	# 5
Rebar Spacing	in =	12.00	6.00
Rebar Placed at	=	Edge	Edge
Design Data			
fb/FB + fa/Fa	=	0.254	0.655
Total Force @ Section	lbs =	1,090.9	2,915.6
Moment....Actual	ft-l =	2,060.6	9,965.7
Moment.....Allowable	ft-l =	8,121.3	15,222.0
Shear.....Actual	psi =	14.7	39.3
Shear.....Allowable	psi =	75.0	75.0
Wall Weight	psf =	100.0	100.0
Rebar Depth 'd'	in =	6.19	6.19
Lap splice if above	in =	23.40	23.40
Lap splice if below	in =	23.40	3.93
Hook embed into footing	in =	23.40	3.93
Concrete Data			
f'c	psi =	2,500.0	2,500.0
Fy	psi =	60,000.0	60,000.0

Cantilevered Retaining Wall

Description : MAX RETAINING WALL - FELDSPAR UVN

Footing Dimensions & Strengths

Toe Width = 3.25 ft
Heel Width = 1.75
Total Footing Width = 5.00
Footing Thickness = 12.00 in
Key Width = 12.00 in
Key Depth = 12.00 in
Key Distance from Toe = 3.00 ft
f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	2,662	0 psf
Mu' : Upward	=	10,168	0 ft-lb
Mu' : Downward	=	2,693	897 ft-lb
Mu: Design	=	7,475	897 ft-lb
Actual 1-Way Shear	=	29.35	14.52 psi
Allow 1-Way Shear	=	75.00	75.00 psi
Toe Reinforcing	=	# 5 @ 8.00 in	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: #4@ 9.00 in, #5@ 14.00 in, #6@ 19.75 in, #7@ 27.00 in, #8@ 35.50 in, #9@ 45
Heel: Not req'd, Mu < S * Fr
Key: #4@ 12.50 in, #5@ 19.25 in, #6@ 27.25 in, #7@ 37.25 in,

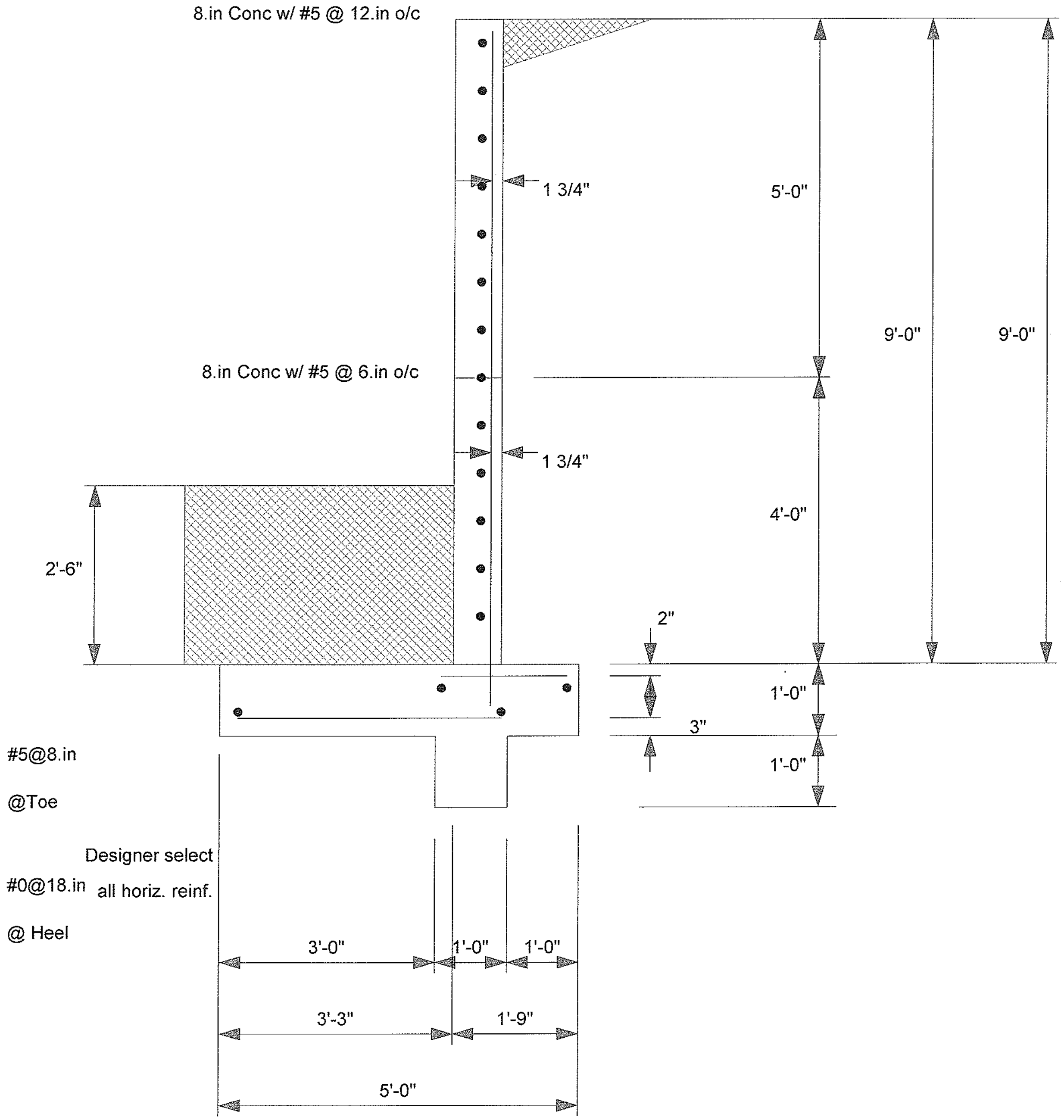
Summary of Overturning & Resisting Forces & Moments

		.....OVERTURNING.....		
Item		Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	2,000.0	3.33	6,666.7
Surcharge over Heel	=	363.6	5.00	1,818.2
Toe Active Pressure	=	-245.0	1.17	-285.8
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Total	=	2,118.6	**O.T.M.** =	8,199.0

Resisting/Overturning Ratio = **1.59**
Vertical Loads used for Soil Pressure = 4,274.6 lbs

		.....RESISTING.....		
		Force lbs	Distance ft	Moment ft-lb
Soil Over Heel	=	1,072.5	4.46	4,781.6
Sloped Soil Over Heel	=			
Surcharge Over Heel	=	108.3	4.46	483.0
Adjacent Footing Load	=			
Axial Dead Load on Stem	=	200.0	3.58	716.7
* Axial Live Load on Stem	=	200.0	3.58	716.7
Soil Over Toe	=	893.8	1.63	1,452.3
Surcharge Over Toe	=			
Stem Weight(s)	=	900.0	3.58	3,225.0
Earth @ Stem Transitions	=			
Footing Weight	=	750.0	2.50	1,875.0
Key Weight	=	150.0	3.50	525.0
Vert. Component	=		5.00	
Total =		4,074.6 lbs	**R.M.** =	13,058.6

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.


8.in Conc w/ #5 @ 12.in o/c
1 3/4"
5'-0"
9'-0"
9'-0"
8.in Conc w/ #5 @ 6.in o/c
1 3/4"
4'-0"
2'-6"
2"
1'-0"
3"
1'-0"
#5@8.in
@Toe
Designer select
all horiz. reinf.
#0@18.in
@ Heel
3'-0"
1'-0"
1'-0"
3'-3"
1'-9"
5'-0"
]

RICHMOND
HOFFMAYER INC.
STRUCTURAL ENGINEERING

Project Title: VILLAGE NEST
Engineer: JJH
Project Descr: MULTI UNIT PROJECT

Project ID: 2017-0610

247

Printed: 11 JUL 2017, 3:31PM

Cantilevered Retaining Wall

File = d:\ENERCALC Projects\2017-0610.ec6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : SIDE RETAINING WALLS - FELDSPAR & QUARTZ LVN & UVN

Calculations per ACI 318-14, ACI 530-11, IBC 2015, CBC 2016, ASCE 7-10

Criteria

Retained Height	=	7.50 ft
Wall height above soil	=	1.00 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	30.00 in
Water height over heel	=	0.0 ft

Vertical component of active
Lateral soil pressure options:
USED for Soil Pressure.
USED for Sliding Resistance.
USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	2,500.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	40.0 psf/ft
Toe Active Pressure	=	40.0 psf/ft
Passive Pressure	=	360.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Friction Coeff btwn Ftg & Soil	=	0.450
Soil height to ignore for passive pressure	=	0.00 in

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
NOT Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
NOT Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	600.0 lbs
Axial Live Load	=	2,000.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 plf
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Wind on Exposed Stem	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Design Summary

Wall Stability Ratios

Overturning	=	2.06	OK
Sliding	=	2.21	OK
Total Bearing Load	=	5,456 lbs	
...resultant ecc.	=	3.34 in	
Soil Pressure @ Toe	=	2,102 psf	OK
Soil Pressure @ Heel	=	808 psf	OK
Allowable	=	2,500 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	2,831 psf	
ACI Factored @ Heel	=	1,088 psf	
Footing Shear @ Toe	=	24.2 psi	OK
Footing Shear @ Heel	=	1.6 psi	OK
Allowable	=	75.0 psi	

Sliding Calcs (Vertical Component Used)

Lateral Sliding Force	=	1,200.0 lbs	
less 50 % Passive Force	= -	1,102.5 lbs	
less 100% Friction Force	= -	1,556.0 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	0.0 lbs	OK

Load Factors

Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		Top Stem	2nd
		Stem OK	Stem OK
Design Height Above Ftg	ft =	4.00	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Thickness	in =	8.00	8.00
Rebar Size	=	# 5	# 5
Rebar Spacing	in =	12.00	12.00
Rebar Placed at	=	Edge	Edge
Design Data			
fb/FB + fa/Fa	=	0.056	0.534
Total Force @ Section	lbs =	392.0	1,600.0
Moment....Actual	ft-l =	457.3	4,333.3
Moment.....Allowable	ft-l =	8,121.3	8,121.3
Shear.....Actual	psi =	5.3	21.5
Shear.....Allowable	psi =	75.0	75.0
Wall Weight	psf =	100.0	100.0
Rebar Depth 'd'	in =	6.19	6.19
Lap splice if above	in =	23.40	23.40
Lap splice if below	in =	23.40	3.60
Hook embed into footing	in =	23.40	3.60
Concrete Data			
f'c	psi =	2,500.0	2,500.0
Fy	psi =	60,000.0	60,000.0

24e

Cantilevered Retaining Wall

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ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : SIDE RETAINING WALLS - FELDSPAR & QUARTZ LVN & UVN

Footing Dimensions & Strengths

Toe Width = 2.00 ft
Heel Width = 1.75
Total Footing Width = 3.75
Footing Thickness = 12.00 in
Key Width = 12.00 in
Key Depth = 0.00 in
Key Distance from Toe = 1.75 ft
f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	2,831	1,088 psf
Mu' : Upward	=	5,042	737 ft-lb
Mu' : Downward	=	1,020	687 ft-lb
Mu: Design	=	4,022	50 ft-lb
Actual 1-Way Shear	=	24.19	1.61 psi
Allow 1-Way Shear	=	75.00	75.00 psi
Toe Reinforcing	=	# 5 @ 8.00 in	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: #4@ 13.25 in, #5@ 20.50 in, #6@ 29.00 in, #7@ 39.25 in, #8@ 48.25 in, #9@ 4
Heel: Not req'd, Mu < S * Fr
Key: Not req'd, Mu < S * Fr

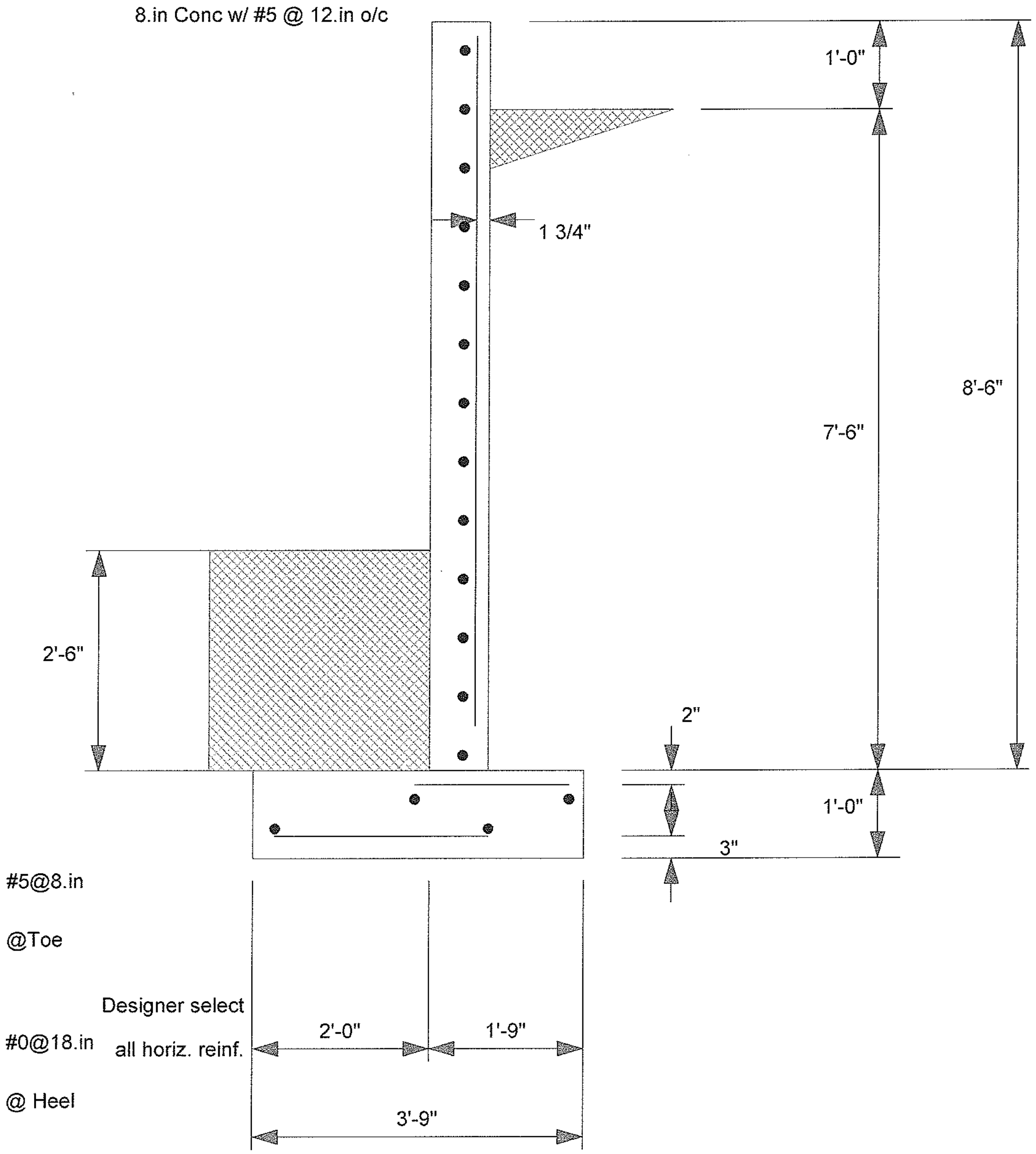
Summary of Overturning & Resisting Forces & Moments

		.....OVERTURNING.....		
Item		Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	1,445.0	2.83	4,094.2
Surcharge over Heel	=			
Toe Active Pressure	=	-245.0	1.17	-285.8
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Total	=	1,200.0	O.T.M. =	3,808.3

Resisting/Overturning Ratio = **2.06**
Vertical Loads used for Soil Pressure = 5,456.3 lbs

		.....RESISTING.....		
		Force lbs	Distance ft	Moment ft-lb
Soil Over Heel	=	893.8	3.21	2,867.4
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=	600.0	2.33	1,400.0
* Axial Live Load on Stem	=	2,000.0	2.33	4,666.7
Soil Over Toe	=	550.0	1.00	550.0
Surcharge Over Toe	=			
Stem Weight(s)	=	850.0	2.33	1,983.3
Earth @ Stem Transitions	=			
Footing Weight	=	562.5	1.88	1,054.7
Key Weight	=		2.25	
Vert. Component	=		3.75	
Total =		3,456.3 lbs	R.M. =	7,855.5

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.


8.in Conc w/ #5 @ 12.in o/c
1'-0"
1 3/4"
8'-6"
7'-6"
2'-6"
2"
1'-0"
3"
#5@8.in
@Toe
Designer select
all horiz. reinf.
#0@18.in
@ Heel
2'-0"
1'-9"
3'-9"

250

Cantilevered Retaining Wall

Description : BOTTOM STEM WALLS - FELDSPAR & QUARTZ LVN & UVN

Calculations per ACI 318-14, ACI 530-11, IBC 2015, CBC 2016, ASCE 7-10

Criteria

Retained Height	=	4.00 ft
Wall height above soil	=	2.00 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	30.00 in
Water height over heel	=	0.0 ft

Vertical component of active
Lateral soil pressure options:
USED for Soil Pressure.
USED for Sliding Resistance.
USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	2,500.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	40.0 psf/ft
Toe Active Pressure	=	40.0 psf/ft
Passive Pressure	=	360.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Friction Coeff btwn Ftg & Soil	=	0.450
Soil height to ignore for passive pressure	=	0.00 in

Surcharge Loads

Surcharge Over Heel	=	40.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
NOT Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	0.0 plf
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Wind on Exposed Stem	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	600.0 lbs
Axial Live Load	=	1,000.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Wall Stability Ratios

Overturning	=	2.88	OK
Sliding	=	6.11	OK
Total Bearing Load	=	3,003 lbs	
...resultant ecc.	=	2.52 in	
Soil Pressure @ Toe	=	2,449 psf	OK
Soil Pressure @ Heel	=	554 psf	OK
Allowable	=	2,500 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	3,265 psf	
ACI Factored @ Heel	=	738 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	2.2 psi	OK
Allowable	=	75.0 psi	

Sliding Calcs (Vertical Component Used)

Lateral Sliding Force	=	327.7 lbs	
less 50 % Passive Force	= -	1,102.5 lbs	
less 100% Friction Force	= -	900.0 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	0.0 lbs	OK

Load Factors

Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		Top Stem Stem OK
Design Height Above Ftg	ft =	0.00
Wall Material Above "Ht"	=	Concrete
Thickness	in =	8.00
Rebar Size	=	# 5
Rebar Spacing	in =	12.00
Rebar Placed at	=	Edge
Design Data		
fb/FB + fa/Fa	=	0.086
Total Force @ Section	lbs =	405.1
Moment....Actual	ft-l =	702.2
Moment.....Allowable	ft-l =	8,121.3
Shear.....Actual	psi =	5.5
Shear.....Allowable	psi =	75.0
Wall Weight	psf =	100.0
Rebar Depth 'd'	in =	6.19
Lap splice if above	in =	23.40
Lap splice if below	in =	6.00
Hook embed into footing	in =	6.00
Concrete Data		
f'c	psi =	2,500.0
Fy	psi =	

Cantilevered Retaining Wall

File = d:\ENERCALC Projects\2017-0610.ec6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06002886 **Licensee : RICHMOND HOFFMAYER**

Description : BOTTOM STEM WALLS - FELDSPAR & QUARTZ LVN & UVN

Footing Dimensions & Strengths

Toe Width = 0.67 ft
Heel Width = 1.33
Total Footing Width = 2.00
Footing Thickness = 12.00 in

Key Width = 12.00 in
Key Depth = 0.00 in
Key Distance from Toe = 1.75 ft

f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	3,265	738 psf
Mu' : Upward	=	669	224 ft-lb
Mu' : Downward	=	114	170 ft-lb
Mu: Design	=	555	54 ft-lb
Actual 1-Way Shear	=	0.00	2.24 psi
Allow 1-Way Shear	=	75.00	75.00 psi
Toe Reinforcing	=	# 5 @ 8.00 in	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: Not req'd, Mu < S * Fr

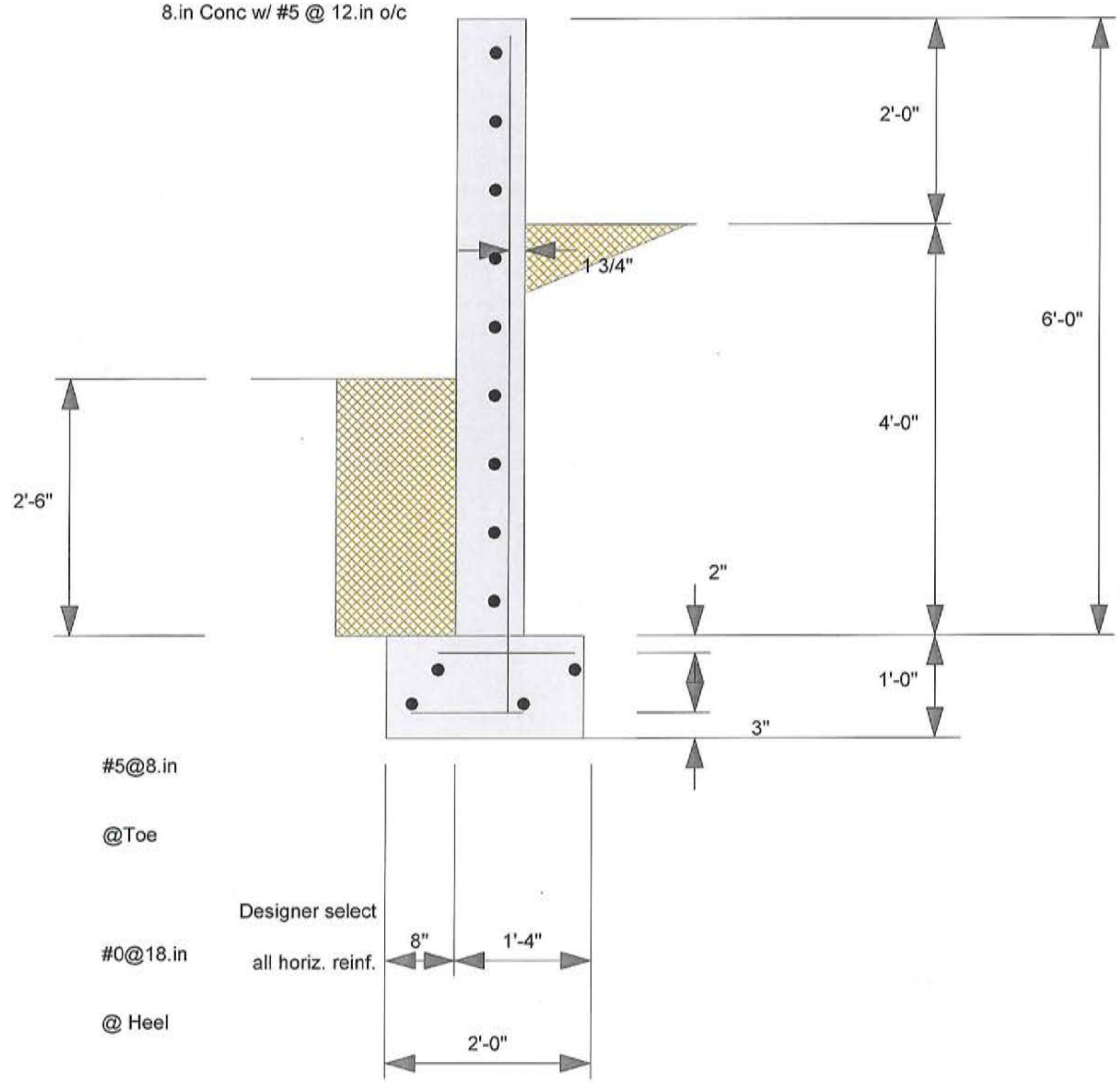
Summary of Overturning & Resisting Forces & Moments

Item		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	500.0	1.67	833.3
Surcharge over Heel	=	72.7	2.50	181.8
Toe Active Pressure	=	-245.0	1.17	-285.8
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Total	=	327.7	**O.T.M.** =	729.3

Resisting/Overturning Ratio = **2.88**
Vertical Loads used for Soil Pressure = 3,002.7 lbs

		.....RESISTING..... Force lbs	Distance ft	Moment ft-lb
Soil Over Heel	=	291.9	1.67	486.9
Sloped Soil Over Heel	=			
Surcharge Over Heel	=	26.5	1.67	44.3
Adjacent Footing Load	=			
Axial Dead Load on Stem	=	600.0	1.00	602.0
* Axial Live Load on Stem	=	1,000.0	1.00	1,003.3
Soil Over Toe	=	184.3	0.34	61.7
Surcharge Over Toe	=			
Stem Weight(s)	=	600.0	1.00	602.0
Earth @ Stem Transitions	=			
Footing Weight	=	300.0	1.00	300.0
Key Weight	=		2.25	
Vert. Component	=		2.00	
Total =		2,002.7 lbs	**R.M. =**	2,096.9

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.


8.in Conc w/ #5 @ 12.in o/c
2'-0"
1 3/4"
6'-0"
4'-0"
2'-6"
2"
1'-0"
3"
#5@8.in
@Toe
Designer select
all horiz. reinf.
#0@18.in
@ Heel
8"
1'-4"
2'-0"

253

General Footing

Description : MAX INTERIOR PAD FOOTING: 4" P.C.

Code References

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

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 = 0.850

Analysis Settings
Min Steel % Bending Reinf. =
Min Allow % Temp Reinf. = 0.00180
Min. Overturning Safety Factor = 1.50 : 1
Min. Sliding Safety Factor = 1.50 : 1
Add Ftg Wt for Soil Pressure : No
Use ftg wt for stability, moments & shears : No
Add Pedestal Wt for Soil Pressure : No
Use Pedestal wt for stability, mom & shear : No

Soil Design Values
Allowable Soil Bearing = 2.50 ksf
Increase Bearing By Footing Weight = No
Soil Passive Resistance (for Sliding) = 180.0 pcf
Soil/Concrete Friction Coeff. = 0.450

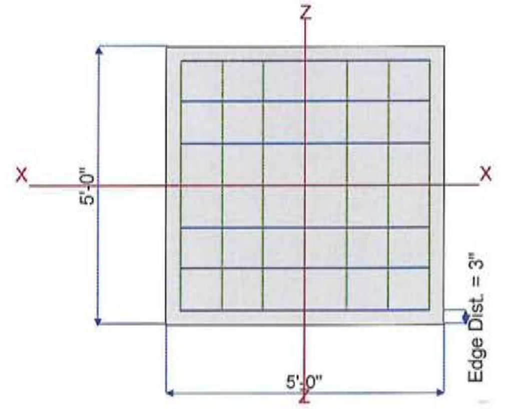
Increases based on footing Depth
Footing base depth below soil surface = 2.0 ft
Allow press. increase per foot of depth = ksf
when footing base is below = ft

Increases based on footing plan dimension
Allowable pressure increase per foot of depth = ksf
when max. length or width is greater than = ft

Dimensions

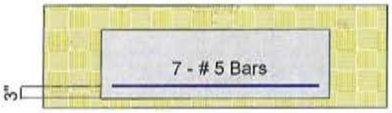
Width parallel to X-X Axis = 5.0 ft
Length parallel to Z-Z Axis = 5.0 ft
Footing Thickness = 18.0 in

Pedestal dimensions...
px : parallel to X-X Axis = in
pz : parallel to Z-Z Axis = in
Height = in
Rebar Centerline to Edge of Concrete...
at Bottom of footing = 3.0 in

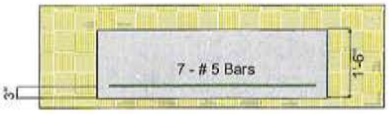


Reinforcing

Bars parallel to X-X Axis
Number of Bars = 7.0
Reinforcing Bar Size = # 5
Bars parallel to Z-Z Axis
Number of Bars = 7.0
Reinforcing Bar Size = # 5



X-X Section Looking to +Z



Z-Z Section Looking to +X

Bandwidth Distribution Check (ACI 15.4.4.2)
Direction Requiring Closer Separation n/a
Bars required within zone n/a
Bars required on each side of zone n/a

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	3.807	4.015		49.381				k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

254

General Footing

Description : MAX INTERIOR PAD FOOTING: 4" P.C.

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8732	Soil Bearing	2.183 ksf	2.50 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
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	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.3692	Z Flexure (+X)	10.447 k-ft/ft	28.298 k-ft/ft	+1.20D+1.60S
PASS	0.3692	Z Flexure (-X)	10.447 k-ft/ft	28.298 k-ft/ft	+1.20D+1.60S
PASS	0.3692	X Flexure (+Z)	10.447 k-ft/ft	28.298 k-ft/ft	+1.20D+1.60S
PASS	0.3692	X Flexure (-Z)	10.447 k-ft/ft	28.298 k-ft/ft	+1.20D+1.60S
PASS	0.2731	1-way Shear (+X)	23.216 psi	85.0 psi	+1.20D+1.60S
PASS	0.2731	1-way Shear (-X)	23.216 psi	85.0 psi	+1.20D+1.60S
PASS	0.2731	1-way Shear (+Z)	23.216 psi	85.0 psi	+1.20D+1.60S
PASS	0.2731	1-way Shear (-Z)	23.216 psi	85.0 psi	+1.20D+1.60S
PASS	0.5074	2-way Punching	86.261 psi	170.0 psi	+1.20D+1.60S

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	2.50	n/a	0.0	0.2073	0.2073	n/a	n/a	0.083
X-X, +D+Lr	2.50	n/a	0.0	0.3679	0.3679	n/a	n/a	0.147
X-X, +D+S	2.50	n/a	0.0	2.183	2.183	n/a	n/a	0.873
X-X, +D+0.750Lr	2.50	n/a	0.0	0.3277	0.3277	n/a	n/a	0.131
X-X, +D+0.750S	2.50	n/a	0.0	1.689	1.689	n/a	n/a	0.676
X-X, +0.60D	2.50	n/a	0.0	0.1244	0.1244	n/a	n/a	0.050
Z-Z, D Only	2.50	0.0	n/a	n/a	n/a	0.2073	0.2073	0.083
Z-Z, +D+Lr	2.50	0.0	n/a	n/a	n/a	0.3679	0.3679	0.147
Z-Z, +D+S	2.50	0.0	n/a	n/a	n/a	2.183	2.183	0.873
Z-Z, +D+0.750Lr	2.50	0.0	n/a	n/a	n/a	0.3277	0.3277	0.131
Z-Z, +D+0.750S	2.50	0.0	n/a	n/a	n/a	1.689	1.689	0.676
Z-Z, +0.60D	2.50	0.0	n/a	n/a	n/a	0.1244	0.1244	0.050

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	0.6662	+Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.40D	0.6662	-Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+0.50Lr	0.8220	+Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+0.50Lr	0.8220	-Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+0.50S	3.657	+Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+0.50S	3.657	-Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+1.60Lr	1.374	+Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+1.60Lr	1.374	-Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+1.60S	10.447	+Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+1.60S	10.447	-Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+0.20S	1.806	+Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+0.20S	1.806	-Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +0.90D	0.4283	+Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +0.90D	0.4283	-Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.40D	0.6662	-X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.40D	0.6662	+X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+0.50Lr	0.8220	-X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+0.50Lr	0.8220	+X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+0.50S	3.657	-X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+0.50S	3.657	+X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+1.60Lr	1.374	-X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+1.60Lr	1.374	+X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+1.60S	10.447	-X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+1.60S	10.447	+X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK

General Footing

Description : MAX INTERIOR PAD FOOTING: 4" P.C.

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
Z-Z, +1.20D+0.20S	1.806	-X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+0.20S	1.806	+X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +0.90D	0.4283	-X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +0.90D	0.4283	+X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	1.48 psi	1.48 psi	1.48 psi	1.48 psi	1.48 psi	85.00 psi	0.02	0.00
+1.20D+0.50Lr	1.83 psi	1.83 psi	1.83 psi	1.83 psi	1.83 psi	85.00 psi	0.02	0.00
+1.20D+0.50S	8.13 psi	8.13 psi	8.13 psi	8.13 psi	8.13 psi	85.00 psi	0.10	0.00
+1.20D+1.60Lr	3.05 psi	3.05 psi	3.05 psi	3.05 psi	3.05 psi	85.00 psi	0.04	0.00
+1.20D+1.60S	23.22 psi	23.22 psi	23.22 psi	23.22 psi	23.22 psi	85.00 psi	0.27	0.00
+1.20D+0.20S	4.01 psi	4.01 psi	4.01 psi	4.01 psi	4.01 psi	85.00 psi	0.05	0.00
+0.90D	0.95 psi	0.95 psi	0.95 psi	0.95 psi	0.95 psi	85.00 psi	0.01	0.00

All units k

Two-Way "Punching" Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	5.50 psi	170.00 psi	0.03236	OK
+1.20D+0.50Lr	6.79 psi	170.00 psi	0.03992	OK
+1.20D+0.50S	30.20 psi	170.00 psi	0.1776	OK
+1.20D+1.60Lr	11.35 psi	170.00 psi	0.06674	OK
+1.20D+1.60S	86.26 psi	170.00 psi	0.5074	OK
+1.20D+0.20S	14.91 psi	170.00 psi	0.0877	OK
+0.90D	3.54 psi	170.00 psi	0.0208	OK

256

General Footing

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Description : INTERIOR PAD FOOTING: FELDSPAR LVN 4" P.C. UNDER B17

Code References

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

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 = 0.850

Analysis Settings
Min Steel % Bending Reinf. =
Min Allow % Temp Reinf. = 0.00180
Min. Overturning Safety Factor = 1.50 : 1
Min. Sliding Safety Factor = 1.50 : 1
Add Ftg Wt for Soil Pressure : No
Use ftg wt for stability, moments & shears : No
Add Pedestal Wt for Soil Pressure : No
Use Pedestal wt for stability, mom & shear : No

Soil Design Values
Allowable Soil Bearing = 2.50 ksf
Increase Bearing By Footing Weight = No
Soil Passive Resistance (for Sliding) = 180.0 pcf
Soil/Concrete Friction Coeff. = 0.450

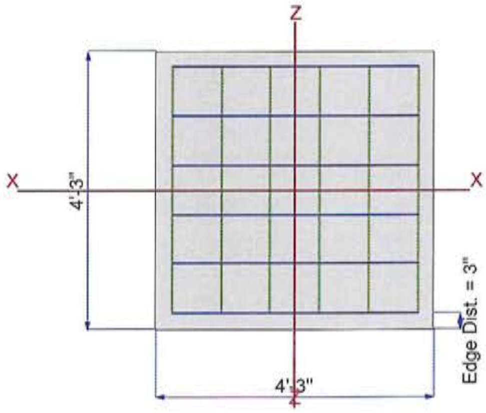
Increases based on footing Depth
Footing base depth below soil surface = 2.0 ft
Allow press. increase per foot of depth = ksf
when footing base is below = ft

Increases based on footing plan dimension
Allowable pressure increase per foot of depth = ksf
when max. length or width is greater than = ft

Dimensions

Width parallel to X-X Axis = 4.250 ft
Length parallel to Z-Z Axis = 4.250 ft
Footing Thickness = 18.0 in

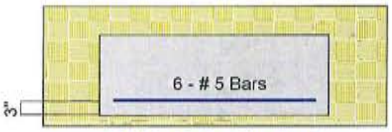
Pedestal dimensions...
px : parallel to X-X Axis = in
pz : parallel to Z-Z Axis = in
Height = in
Rebar Centerline to Edge of Concrete...
at Bottom of footing = 3.0 in



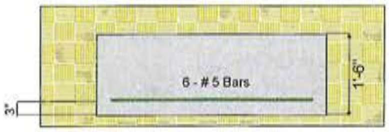
Reinforcing

Bars parallel to X-X Axis =
Number of Bars = 6.0
Reinforcing Bar Size = # 5
Bars parallel to Z-Z Axis
Number of Bars = 6.0
Reinforcing Bar Size = # 5

Bandwidth Distribution Check (ACI 15.4.4.2)
Direction Requiring Closer Separation n/a
Bars required within zone n/a
Bars required on each side of zone n/a



X-X Section Looking to +Z



Z-Z Section Looking to +X

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	2.843	2.906	0.280	36.394				k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing

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Description : INTERIOR PAD FOOTING: FELDSPAR LVN 4" P.C. UNDER B17

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8908	Soil Bearing	2.227 ksf	2.50 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
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	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.2707	Z Flexure (+X)	7.723 k-ft/ft	28.527 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.2707	Z Flexure (-X)	7.723 k-ft/ft	28.527 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.2707	X Flexure (+Z)	7.723 k-ft/ft	28.527 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.2707	X Flexure (-Z)	7.723 k-ft/ft	28.527 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.190	1-way Shear (+X)	16.152 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.190	1-way Shear (-X)	16.152 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.190	1-way Shear (+Z)	16.152 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.190	1-way Shear (-Z)	16.152 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.3675	2-way Punching	62.468 psi	170.0 psi	+1.20D+0.50L+1.60S

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	2.50	n/a	0.0	0.2124	0.2124	n/a	n/a	0.085
X-X, +D+L	2.50	n/a	0.0	0.2279	0.2279	n/a	n/a	0.091
X-X, +D+Lr	2.50	n/a	0.0	0.3733	0.3733	n/a	n/a	0.149
X-X, +D+S	2.50	n/a	0.0	2.227	2.227	n/a	n/a	0.891
X-X, +D+0.750Lr+0.750L	2.50	n/a	0.0	0.3447	0.3447	n/a	n/a	0.138
X-X, +D+0.750L+0.750S	2.50	n/a	0.0	1.735	1.735	n/a	n/a	0.694
X-X, +0.60D	2.50	n/a	0.0	0.1274	0.1274	n/a	n/a	0.051
Z-Z, D Only	2.50	0.0	n/a	n/a	n/a	0.2124	0.2124	0.085
Z-Z, +D+L	2.50	0.0	n/a	n/a	n/a	0.2279	0.2279	0.091
Z-Z, +D+Lr	2.50	0.0	n/a	n/a	n/a	0.3733	0.3733	0.149
Z-Z, +D+S	2.50	0.0	n/a	n/a	n/a	2.227	2.227	0.891
Z-Z, +D+0.750Lr+0.750L	2.50	0.0	n/a	n/a	n/a	0.3447	0.3447	0.138
Z-Z, +D+0.750L+0.750S	2.50	0.0	n/a	n/a	n/a	1.735	1.735	0.694
Z-Z, +0.60D	2.50	0.0	n/a	n/a	n/a	0.1274	0.1274	0.051

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	0.4975	+Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.40D	0.4975	-Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+0.50Lr+1.60L	0.6641	+Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+0.50Lr+1.60L	0.6641	-Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+1.60L+0.50S	2.757	+Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+1.60L+0.50S	2.757	-Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+1.60Lr+0.50L	1.025	+Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+1.60Lr+0.50L	1.025	-Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+1.60Lr	1.008	+Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+1.60Lr	1.008	-Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+0.50L+1.60S	7.723	+Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+0.50L+1.60S	7.723	-Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+1.60S	7.705	+Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+1.60S	7.705	-Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+0.50Lr+0.50L	0.6256	+Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+0.50Lr+0.50L	0.6256	-Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+0.50L+0.50S	2.719	+Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+0.50L+0.50S	2.719	-Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+0.50L+0.20S	1.354	+Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +1.20D+0.50L+0.20S	1.354	-Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +0.90D	0.3198	+Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
X-X, +0.90D	0.3198	-Z	Bottom	0.3888	Min Temp %	0.4376	28.527	OK

General Footing

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Description : INTERIOR PAD FOOTING: FELDSPAR LVN 4" P.C. UNDER B17

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
Z-Z, +1.40D	0.4975	-X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.40D	0.4975	+X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+0.50Lr+1.60L	0.6641	-X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+0.50Lr+1.60L	0.6641	+X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+1.60L+0.50S	2.757	-X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+1.60L+0.50S	2.757	+X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+1.60Lr+0.50L	1.025	-X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+1.60Lr+0.50L	1.025	+X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+1.60Lr	1.008	-X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+1.60Lr	1.008	+X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+0.50L+1.60S	7.723	-X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+0.50L+1.60S	7.723	+X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+1.60S	7.705	-X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+1.60S	7.705	+X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+0.50Lr+0.50L	0.6256	-X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+0.50Lr+0.50L	0.6256	+X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+0.50L+0.50S	2.719	-X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+0.50L+0.50S	2.719	+X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+0.50L+0.20S	1.354	-X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +1.20D+0.50L+0.20S	1.354	+X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +0.90D	0.3198	-X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK
Z-Z, +0.90D	0.3198	+X	Bottom	0.3888	Min Temp %	0.4376	28.527	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	1.04 psi	1.04 psi	1.04 psi	1.04 psi	1.04 psi	85.00 psi	0.01	0.00
+1.20D+0.50Lr+1.60L	1.39 psi	1.39 psi	1.39 psi	1.39 psi	1.39 psi	85.00 psi	0.02	0.00
+1.20D+1.60L+0.50S	5.77 psi	5.77 psi	5.77 psi	5.77 psi	5.77 psi	85.00 psi	0.07	0.00
+1.20D+1.60Lr+0.50L	2.14 psi	2.14 psi	2.14 psi	2.14 psi	2.14 psi	85.00 psi	0.03	0.00
+1.20D+1.60Lr	2.11 psi	2.11 psi	2.11 psi	2.11 psi	2.11 psi	85.00 psi	0.02	0.00
+1.20D+0.50L+1.60S	16.15 psi	16.15 psi	16.15 psi	16.15 psi	16.15 psi	85.00 psi	0.19	0.00
+1.20D+1.60S	16.12 psi	16.12 psi	16.12 psi	16.12 psi	16.12 psi	85.00 psi	0.19	0.00
+1.20D+0.50Lr+0.50L	1.31 psi	1.31 psi	1.31 psi	1.31 psi	1.31 psi	85.00 psi	0.02	0.00
+1.20D+0.50L+0.50S	5.69 psi	5.69 psi	5.69 psi	5.69 psi	5.69 psi	85.00 psi	0.07	0.00
+1.20D+0.50L+0.20S	2.83 psi	2.83 psi	2.83 psi	2.83 psi	2.83 psi	85.00 psi	0.03	0.00
+0.90D	0.67 psi	0.67 psi	0.67 psi	0.67 psi	0.67 psi	85.00 psi	0.01	0.00

All units k

Two-Way "Punching" Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	4.02 psi	170.00 psi	0.02367	OK
+1.20D+0.50Lr+1.60L	5.37 psi	170.00 psi	0.0316	OK
+1.20D+1.60L+0.50S	22.30 psi	170.00 psi	0.1312	OK
+1.20D+1.60Lr+0.50L	8.29 psi	170.00 psi	0.04878	OK
+1.20D+1.60Lr	8.15 psi	170.00 psi	0.04795	OK
+1.20D+0.50L+1.60S	62.47 psi	170.00 psi	0.3675	OK
+1.20D+1.60S	62.33 psi	170.00 psi	0.3666	OK
+1.20D+0.50Lr+0.50L	5.06 psi	170.00 psi	0.02977	OK
+1.20D+0.50L+0.50S	21.99 psi	170.00 psi	0.1294	OK
+1.20D+0.50L+0.20S	10.95 psi	170.00 psi	0.06442	OK
+0.90D	2.59 psi	170.00 psi	0.01522	OK

General Footing

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Lic. # : KW-06002886

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Description : INTERIOR PAD FOOTING: QUARTZ LVN 4" P.C. UNDER B16 - NOT AT RIDGE

Code References

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

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 = 0.850

Analysis Settings

Min Steel % Bending Reinf. =
Min Allow % Temp Reinf. = 0.00180
Min. Overturning Safety Factor = 1.50 : 1
Min. Sliding Safety Factor = 1.50 : 1
Add Ftg Wt for Soil Pressure : No
Use ftg wt for stability, moments & shears : No
Add Pedestal Wt for Soil Pressure : No
Use Pedestal wt for stability, mom & shear : No

Soil Design Values

Allowable Soil Bearing = 2.50 ksf
Increase Bearing By Footing Weight = No
Soil Passive Resistance (for Sliding) = 180.0 pcf
Soil/Concrete Friction Coeff. = 0.450

Increases based on footing Depth

Footing base depth below soil surface = 2.0 ft
Allow press. increase per foot of depth = ksf
when footing base is below = ft

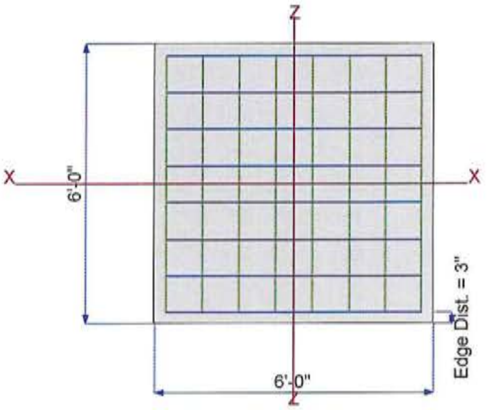
Increases based on footing plan dimension

Allowable pressure increase per foot of depth = ksf
when max. length or width is greater than = ft

Dimensions

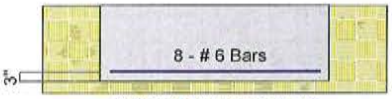
Width parallel to X-X Axis = 6.0 ft
Length parallel to Z-Z Axis = 6.0 ft
Footing Thickness = 24.0 in

Pedestal dimensions...
px : parallel to X-X Axis = in
pz : parallel to Z-Z Axis = in
Height = in
Rebar Centerline to Edge of Concrete...
at Bottom of footing = 3.0 in

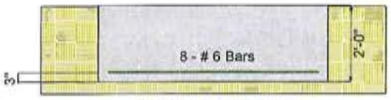


Reinforcing

Bars parallel to X-X Axis =
Number of Bars 8.0
Reinforcing Bar Size = # 6
Bars parallel to Z-Z Axis =
Number of Bars 8.0
Reinforcing Bar Size = # 6



X-X Section Looking to +Z



Z-Z Section Looking to +X

Bandwidth Distribution Check (ACI 15.4.4.2)
Direction Requiring Closer Separation n/a
Bars required within zone n/a
Bars required on each side of zone n/a

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	6.880	1.120	7.625	60.0				k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing

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Description : INTERIOR PAD FOOTING: QUARTZ LVN 4" P.C. UNDER B16 - NOT AT RIDGE

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.7432	Soil Bearing	1.858 ksf	2.50 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
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	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.2519	Z Flexure (+X)	13.509 k-ft/ft	53.618 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.2519	Z Flexure (-X)	13.509 k-ft/ft	53.618 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.2519	X Flexure (+Z)	13.509 k-ft/ft	53.618 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.2519	X Flexure (-Z)	13.509 k-ft/ft	53.618 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.1822	1-way Shear (+X)	15.486 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.1822	1-way Shear (-X)	15.486 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.1822	1-way Shear (+Z)	15.486 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.1822	1-way Shear (-Z)	15.486 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.3279	2-way Punching	55.750 psi	170.0 psi	+1.20D+0.50L+1.60S

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	2.50	n/a	0.0	0.1911	0.1911	n/a	n/a	0.076
X-X, +D+L	2.50	n/a	0.0	0.4029	0.4029	n/a	n/a	0.161
X-X, +D+Lr	2.50	n/a	0.0	0.2222	0.2222	n/a	n/a	0.089
X-X, +D+S	2.50	n/a	0.0	1.858	1.858	n/a	n/a	0.743
X-X, +D+0.750Lr+0.750L	2.50	n/a	0.0	0.3733	0.3733	n/a	n/a	0.149
X-X, +D+0.750L+0.750S	2.50	n/a	0.0	1.60	1.60	n/a	n/a	0.640
X-X, +0.60D	2.50	n/a	0.0	0.1147	0.1147	n/a	n/a	0.046
Z-Z, D Only	2.50	0.0	n/a	n/a	n/a	0.1911	0.1911	0.076
Z-Z, +D+L	2.50	0.0	n/a	n/a	n/a	0.4029	0.4029	0.161
Z-Z, +D+Lr	2.50	0.0	n/a	n/a	n/a	0.2222	0.2222	0.089
Z-Z, +D+S	2.50	0.0	n/a	n/a	n/a	1.858	1.858	0.743
Z-Z, +D+0.750Lr+0.750L	2.50	0.0	n/a	n/a	n/a	0.3733	0.3733	0.149
Z-Z, +D+0.750L+0.750S	2.50	0.0	n/a	n/a	n/a	1.60	1.60	0.640
Z-Z, +0.60D	2.50	0.0	n/a	n/a	n/a	0.1147	0.1147	0.046

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	1.204	+Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.40D	1.204	-Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+0.50Lr+1.60L	2.627	+Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+0.50Lr+1.60L	2.627	-Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+1.60L+0.50S	6.307	+Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+1.60L+0.50S	6.307	-Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+1.60Lr+0.50L	1.733	+Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+1.60Lr+0.50L	1.733	-Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+1.60Lr	1.256	+Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+1.60Lr	1.256	-Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+0.50L+1.60S	13.509	+Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+0.50L+1.60S	13.509	-Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+1.60S	13.032	+Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+1.60S	13.032	-Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+0.50Lr+0.50L	1.579	+Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+0.50Lr+0.50L	1.579	-Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+0.50L+0.50S	5.259	+Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+0.50L+0.50S	5.259	-Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+0.50L+0.20S	3.009	+Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +1.20D+0.50L+0.20S	3.009	-Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +0.90D	0.7740	+Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X, +0.90D	0.7740	-Z	Bottom	0.5184	Min Temp %	0.5867	53.618	OK

General Footing

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Description : INTERIOR PAD FOOTING: QUARTZ LVN 4" P.C. UNDER B16 - NOT AT RIDGE

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
Z-Z, +1.40D	1.204	-X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.40D	1.204	+X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+0.50Lr+1.60L	2.627	-X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+0.50Lr+1.60L	2.627	+X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+1.60L+0.50S	6.307	-X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+1.60L+0.50S	6.307	+X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+1.60Lr+0.50L	1.733	-X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+1.60Lr+0.50L	1.733	+X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+1.60Lr	1.256	-X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+1.60Lr	1.256	+X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+0.50L+1.60S	13.509	-X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+0.50L+1.60S	13.509	+X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+1.60S	13.032	-X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+1.60S	13.032	+X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+0.50Lr+0.50L	1.579	-X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+0.50Lr+0.50L	1.579	+X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+0.50L+0.50S	5.259	-X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+0.50L+0.50S	5.259	+X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+0.50L+0.20S	3.009	-X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +1.20D+0.50L+0.20S	3.009	+X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +0.90D	0.7740	-X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
Z-Z, +0.90D	0.7740	+X	Bottom	0.5184	Min Temp %	0.5867	53.618	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	1.38 psi	1.38 psi	1.38 psi	1.38 psi	1.38 psi	85.00 psi	0.02	0.00
+1.20D+0.50Lr+1.60L	3.01 psi	3.01 psi	3.01 psi	3.01 psi	3.01 psi	85.00 psi	0.04	0.00
+1.20D+1.60L+0.50S	7.23 psi	7.23 psi	7.23 psi	7.23 psi	7.23 psi	85.00 psi	0.09	0.00
+1.20D+1.60Lr+0.50L	1.99 psi	1.99 psi	1.99 psi	1.99 psi	1.99 psi	85.00 psi	0.02	0.00
+1.20D+1.60Lr	1.44 psi	1.44 psi	1.44 psi	1.44 psi	1.44 psi	85.00 psi	0.02	0.00
+1.20D+0.50L+1.60S	15.49 psi	15.49 psi	15.49 psi	15.49 psi	15.49 psi	85.00 psi	0.18	0.00
+1.20D+1.60S	14.94 psi	14.94 psi	14.94 psi	14.94 psi	14.94 psi	85.00 psi	0.18	0.00
+1.20D+0.50Lr+0.50L	1.81 psi	1.81 psi	1.81 psi	1.81 psi	1.81 psi	85.00 psi	0.02	0.00
+1.20D+0.50L+0.50S	6.03 psi	6.03 psi	6.03 psi	6.03 psi	6.03 psi	85.00 psi	0.07	0.00
+1.20D+0.50L+0.20S	3.45 psi	3.45 psi	3.45 psi	3.45 psi	3.45 psi	85.00 psi	0.04	0.00
+0.90D	0.89 psi	0.89 psi	0.89 psi	0.89 psi	0.89 psi	85.00 psi	0.01	0.00

All units k

Two-Way "Punching" Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	4.97 psi	170.00psi	0.02923	OK
+1.20D+0.50Lr+1.60L	10.84 psi	170.00psi	0.06377	OK
+1.20D+1.60L+0.50S	26.03 psi	170.00psi	0.1531	OK
+1.20D+1.60Lr+0.50L	7.15 psi	170.00psi	0.04206	OK
+1.20D+1.60Lr	5.18 psi	170.00psi	0.03049	OK
+1.20D+0.50L+1.60S	55.75 psi	170.00psi	0.3279	OK
+1.20D+1.60S	53.78 psi	170.00psi	0.3164	OK
+1.20D+0.50Lr+0.50L	6.52 psi	170.00psi	0.03832	OK
+1.20D+0.50L+0.50S	21.70 psi	170.00psi	0.1277	OK
+1.20D+0.50L+0.20S	12.42 psi	170.00psi	0.07304	OK
+0.90D	3.19 psi	170.00psi	0.01879	OK

General Footing

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Licensee : RICHMOND HOFFMAYER

Description : INTERIOR PAD FOOTING: QUARTZ LVN 4" P.C. UNDER B16 - UNDER RIDGE

Code References

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

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	=	0.850

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.50 : 1
Min. Sliding Safety Factor	=	1.50 : 1
Add Ftg Wt for Soil Pressure	:	No
Use ftg wt for stability, moments & shears	:	No
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	2.50 ksf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	180.0 pcf
Soil/Concrete Friction Coeff.	=	0.450

Increases based on footing Depth

Footing base depth below soil surface	=	2.0 ft
Allow press. increase per foot of depth	=	ksf
when footing base is below	=	ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=	ksf
when max. length or width is greater than	=	ft

Dimensions

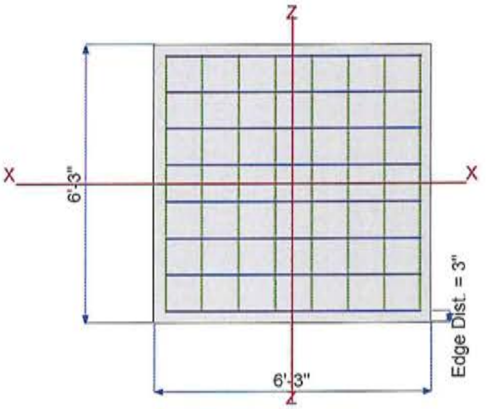
Width parallel to X-X Axis	=	6.250 ft
Length parallel to Z-Z Axis	=	6.250 ft
Footing Thickness	=	24.0 in

Pedestal dimensions...

px : parallel to X-X Axis	=	in
pz : parallel to Z-Z Axis	=	in
Height	=	in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0 in



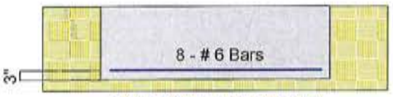
Reinforcing

Bars parallel to X-X Axis

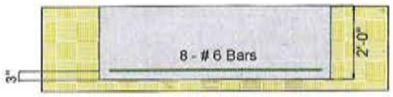
Number of Bars	=	8.0
Reinforcing Bar Size	=	# 6

Bars parallel to Z-Z Axis

Number of Bars	=	8.0
Reinforcing Bar Size	=	# 6



X-X Section Looking to +Z



Z-Z Section Looking to +X

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	9.0	1.0	9.0	80.0				k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

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General Footing

Description : INTERIOR PAD FOOTING: QUARTZ LVN 4" P.C. UNDER B16 - UNDER RIDGE

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.9112	Soil Bearing	2.278 ksf	2.50 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
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	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.3475	Z Flexure (+X)	17.913 k-ft/ft	51.543 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.3475	Z Flexure (-X)	17.913 k-ft/ft	51.543 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.3475	X Flexure (+Z)	17.913 k-ft/ft	51.543 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.3475	X Flexure (-Z)	17.913 k-ft/ft	51.543 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.2319	1-way Shear (+X)	19.713 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.2319	1-way Shear (-X)	19.713 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.2319	1-way Shear (+Z)	19.713 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.2319	1-way Shear (-Z)	19.713 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.4439	2-way Punching	75.459 psi	170.0 psi	+1.20D+0.50L+1.60S

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	2.50	n/a	0.0	0.2304	0.2304	n/a	n/a	0.092
X-X, +D+L	2.50	n/a	0.0	0.4608	0.4608	n/a	n/a	0.184
X-X, +D+Lr	2.50	n/a	0.0	0.2560	0.2560	n/a	n/a	0.102
X-X, +D+S	2.50	n/a	0.0	2.278	2.278	n/a	n/a	0.911
X-X, +D+0.750Lr+0.750L	2.50	n/a	0.0	0.4224	0.4224	n/a	n/a	0.169
X-X, +D+0.750L+0.750S	2.50	n/a	0.0	1.939	1.939	n/a	n/a	0.776
X-X, +0.60D	2.50	n/a	0.0	0.1382	0.1382	n/a	n/a	0.055
Z-Z, D Only	2.50	0.0	n/a	n/a	n/a	0.2304	0.2304	0.092
Z-Z, +D+L	2.50	0.0	n/a	n/a	n/a	0.4608	0.4608	0.184
Z-Z, +D+Lr	2.50	0.0	n/a	n/a	n/a	0.2560	0.2560	0.102
Z-Z, +D+S	2.50	0.0	n/a	n/a	n/a	2.278	2.278	0.911
Z-Z, +D+0.750Lr+0.750L	2.50	0.0	n/a	n/a	n/a	0.4224	0.4224	0.169
Z-Z, +D+0.750L+0.750S	2.50	0.0	n/a	n/a	n/a	1.939	1.939	0.776
Z-Z, +0.60D	2.50	0.0	n/a	n/a	n/a	0.1382	0.1382	0.055

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	1.575	+Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.40D	1.575	-Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+0.50Lr+1.60L	3.213	+Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+0.50Lr+1.60L	3.213	-Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+1.60L+0.50S	8.150	+Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+1.60L+0.50S	8.150	-Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+1.60Lr+0.50L	2.113	+Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+1.60Lr+0.50L	2.113	-Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+1.60Lr	1.550	+Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+1.60Lr	1.550	-Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+0.50L+1.60S	17.913	+Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+0.50L+1.60S	17.913	-Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+1.60S	17.350	+Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+1.60S	17.350	-Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+0.50Lr+0.50L	1.975	+Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+0.50Lr+0.50L	1.975	-Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+0.50L+0.50S	6.913	+Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+0.50L+0.50S	6.913	-Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+0.50L+0.20S	3.913	+Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +1.20D+0.50L+0.20S	3.913	-Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +0.90D	1.013	+Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
X-X, +0.90D	1.013	-Z	Bottom	0.5184	Min Temp %	0.5632	51.543	OK

General Footing

File = d:\ENERCALC Projects\2017-0610.ec6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Lic. # : KW-06002886

Licensee : RICHMOND HOFFMAYER

Description : INTERIOR PAD FOOTING: QUARTZ LVN 4" P.C. UNDER B16 - UNDER RIDGE

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
Z-Z, +1.40D	1.575	-X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.40D	1.575	+X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+0.50Lr+1.60L	3.213	-X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+0.50Lr+1.60L	3.213	+X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+1.60L+0.50S	8.150	-X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+1.60L+0.50S	8.150	+X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+1.60Lr+0.50L	2.113	-X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+1.60Lr+0.50L	2.113	+X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+1.60Lr	1.550	-X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+1.60Lr	1.550	+X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+0.50L+1.60S	17.913	-X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+0.50L+1.60S	17.913	+X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+1.60S	17.350	-X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+1.60S	17.350	+X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+0.50Lr+0.50L	1.975	-X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+0.50Lr+0.50L	1.975	+X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+0.50L+0.50S	6.913	-X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+0.50L+0.50S	6.913	+X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+0.50L+0.20S	3.913	-X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +1.20D+0.50L+0.20S	3.913	+X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +0.90D	1.013	-X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK
Z-Z, +0.90D	1.013	+X	Bottom	0.5184	Min Temp %	0.5632	51.543	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	1.73 psi	1.73 psi	1.73 psi	1.73 psi	1.73 psi	85.00 psi	0.02	0.00
+1.20D+0.50Lr+1.60L	3.54 psi	3.54 psi	3.54 psi	3.54 psi	3.54 psi	85.00 psi	0.04	0.00
+1.20D+1.60L+0.50S	8.97 psi	8.97 psi	8.97 psi	8.97 psi	8.97 psi	85.00 psi	0.11	0.00
+1.20D+1.60Lr+0.50L	2.33 psi	2.33 psi	2.33 psi	2.33 psi	2.33 psi	85.00 psi	0.03	0.00
+1.20D+1.60Lr	1.71 psi	1.71 psi	1.71 psi	1.71 psi	1.71 psi	85.00 psi	0.02	0.00
+1.20D+0.50L+1.60S	19.71 psi	19.71 psi	19.71 psi	19.71 psi	19.71 psi	85.00 psi	0.23	0.00
+1.20D+1.60S	19.09 psi	19.09 psi	19.09 psi	19.09 psi	19.09 psi	85.00 psi	0.22	0.00
+1.20D+0.50Lr+0.50L	2.17 psi	2.17 psi	2.17 psi	2.17 psi	2.17 psi	85.00 psi	0.03	0.00
+1.20D+0.50L+0.50S	7.61 psi	7.61 psi	7.61 psi	7.61 psi	7.61 psi	85.00 psi	0.09	0.00
+1.20D+0.50L+0.20S	4.31 psi	4.31 psi	4.31 psi	4.31 psi	4.31 psi	85.00 psi	0.05	0.00
+0.90D	1.11 psi	1.11 psi	1.11 psi	1.11 psi	1.11 psi	85.00 psi	0.01	0.00

All units k

Two-Way "Punching" Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	6.64 psi	170.00 psi	0.03903	OK
+1.20D+0.50Lr+1.60L	13.53 psi	170.00 psi	0.07961	OK
+1.20D+1.60L+0.50S	34.33 psi	170.00 psi	0.202	OK
+1.20D+1.60Lr+0.50L	8.90 psi	170.00 psi	0.05235	OK
+1.20D+1.60Lr	6.53 psi	170.00 psi	0.03841	OK
+1.20D+0.50L+1.60S	75.46 psi	170.00 psi	0.4439	OK
+1.20D+1.60S	73.09 psi	170.00 psi	0.4299	OK
+1.20D+0.50Lr+0.50L	8.32 psi	170.00 psi	0.04894	OK
+1.20D+0.50L+0.50S	29.12 psi	170.00 psi	0.1713	OK
+1.20D+0.50L+0.20S	16.48 psi	170.00 psi	0.09695	OK
+0.90D	4.27 psi	170.00 psi	0.02509	OK

General Footing

Description : MAX EXTERIOR PAD FOOTING: 6" HSS

Code References

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

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 = 0.850

Analysis Settings

Min Steel % Bending Reinf. =
Min Allow % Temp Reinf. = 0.00180
Min. Overturning Safety Factor = 1.50 : 1
Min. Sliding Safety Factor = 1.50 : 1
Add Ftg Wt for Soil Pressure : No
Use ftg wt for stability, moments & shears : No
Add Pedestal Wt for Soil Pressure : No
Use Pedestal wt for stability, mom & shear : No

Soil Design Values

Allowable Soil Bearing = 2.50 ksf
Increase Bearing By Footing Weight = No
Soil Passive Resistance (for Sliding) = 180.0 pcf
Soil/Concrete Friction Coeff. = 0.450

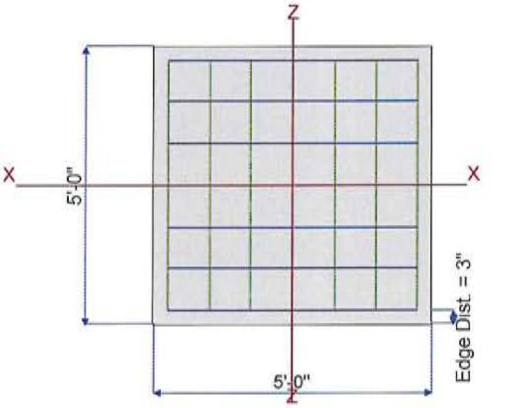
Increases based on footing Depth
Footing base depth below soil surface = 2.0 ft
Allow press. increase per foot of depth = ksf
when footing base is below = ft

Increases based on footing plan dimension
Allowable pressure increase per foot of depth = ksf
when max. length or width is greater than = ft

Dimensions

Width parallel to X-X Axis = 5.0 ft
Length parallel to Z-Z Axis = 5.0 ft
Footing Thickness = 18.0 in

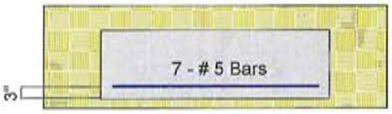
Pedestal dimensions...
px : parallel to X-X Axis = in
pz : parallel to Z-Z Axis = in
Height = in
Rebar Centerline to Edge of Concrete...
at Bottom of footing = 3.0 in



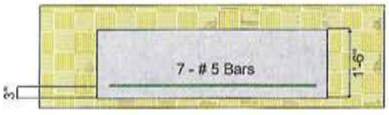
Reinforcing

Bars parallel to X-X Axis
Number of Bars = 7.0
Reinforcing Bar Size = # 5
Bars parallel to Z-Z Axis
Number of Bars = 7.0
Reinforcing Bar Size = # 5

Bandwidth Distribution Check (ACI 15.4.4.2)
Direction Requiring Closer Separation n/a
Bars required within zone n/a
Bars required on each side of zone n/a



X-X Section Looking to +Z



Z-Z Section Looking to +X

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	3.807	4.015		49.381				k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing

Lic. # : KW-06002886 **Licensee : RICHMOND HOFFMAYER**

Description : MAX EXTERIOR PAD FOOTING: 6" HSS

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8732	Soil Bearing	2.183 ksf	2.50 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
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	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.3692	Z Flexure (+X)	10.447 k-ft/ft	28.298 k-ft/ft	+1.20D+1.60S
PASS	0.3692	Z Flexure (-X)	10.447 k-ft/ft	28.298 k-ft/ft	+1.20D+1.60S
PASS	0.3692	X Flexure (+Z)	10.447 k-ft/ft	28.298 k-ft/ft	+1.20D+1.60S
PASS	0.3692	X Flexure (-Z)	10.447 k-ft/ft	28.298 k-ft/ft	+1.20D+1.60S
PASS	0.2731	1-way Shear (+X)	23.216 psi	85.0 psi	+1.20D+1.60S
PASS	0.2731	1-way Shear (-X)	23.216 psi	85.0 psi	+1.20D+1.60S
PASS	0.2731	1-way Shear (+Z)	23.216 psi	85.0 psi	+1.20D+1.60S
PASS	0.2731	1-way Shear (-Z)	23.216 psi	85.0 psi	+1.20D+1.60S
PASS	0.5074	2-way Punching	86.261 psi	170.0 psi	+1.20D+1.60S

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	2.50	n/a	0.0	0.2073	0.2073	n/a	n/a	0.083
X-X, +D+Lr	2.50	n/a	0.0	0.3679	0.3679	n/a	n/a	0.147
X-X, +D+S	2.50	n/a	0.0	2.183	2.183	n/a	n/a	0.873
X-X, +D+0.750Lr	2.50	n/a	0.0	0.3277	0.3277	n/a	n/a	0.131
X-X, +D+0.750S	2.50	n/a	0.0	1.689	1.689	n/a	n/a	0.676
X-X, +0.60D	2.50	n/a	0.0	0.1244	0.1244	n/a	n/a	0.050
Z-Z, D Only	2.50	0.0	n/a	n/a	n/a	0.2073	0.2073	0.083
Z-Z, +D+Lr	2.50	0.0	n/a	n/a	n/a	0.3679	0.3679	0.147
Z-Z, +D+S	2.50	0.0	n/a	n/a	n/a	2.183	2.183	0.873
Z-Z, +D+0.750Lr	2.50	0.0	n/a	n/a	n/a	0.3277	0.3277	0.131
Z-Z, +D+0.750S	2.50	0.0	n/a	n/a	n/a	1.689	1.689	0.676
Z-Z, +0.60D	2.50	0.0	n/a	n/a	n/a	0.1244	0.1244	0.050

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	0.6662	+Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.40D	0.6662	-Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+0.50Lr	0.8220	+Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+0.50Lr	0.8220	-Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+0.50S	3.657	+Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+0.50S	3.657	-Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+1.60Lr	1.374	+Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+1.60Lr	1.374	-Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+1.60S	10.447	+Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+1.60S	10.447	-Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+0.20S	1.806	+Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +1.20D+0.20S	1.806	-Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +0.90D	0.4283	+Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
X-X, +0.90D	0.4283	-Z	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.40D	0.6662	-X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.40D	0.6662	+X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+0.50Lr	0.8220	-X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+0.50Lr	0.8220	+X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+0.50S	3.657	-X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+0.50S	3.657	+X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+1.60Lr	1.374	-X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+1.60Lr	1.374	+X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+1.60S	10.447	-X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+1.60S	10.447	+X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK

General Footing

Description : MAX EXTERIOR PAD FOOTING: 6" HSS

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
Z-Z, +1.20D+0.20S	1.806	-X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +1.20D+0.20S	1.806	+X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +0.90D	0.4283	-X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK
Z-Z, +0.90D	0.4283	+X	Bottom	0.3888	Min Temp %	0.4340	28.298	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	1.48 psi	1.48 psi	1.48 psi	1.48 psi	1.48 psi	85.00 psi	0.02	0.00
+1.20D+0.50Lr	1.83 psi	1.83 psi	1.83 psi	1.83 psi	1.83 psi	85.00 psi	0.02	0.00
+1.20D+0.50S	8.13 psi	8.13 psi	8.13 psi	8.13 psi	8.13 psi	85.00 psi	0.10	0.00
+1.20D+1.60Lr	3.05 psi	3.05 psi	3.05 psi	3.05 psi	3.05 psi	85.00 psi	0.04	0.00
+1.20D+1.60S	23.22 psi	23.22 psi	23.22 psi	23.22 psi	23.22 psi	85.00 psi	0.27	0.00
+1.20D+0.20S	4.01 psi	4.01 psi	4.01 psi	4.01 psi	4.01 psi	85.00 psi	0.05	0.00
+0.90D	0.95 psi	0.95 psi	0.95 psi	0.95 psi	0.95 psi	85.00 psi	0.01	0.00

All units k

Two-Way "Punching" Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	5.50 psi	170.00psi	0.03236	OK
+1.20D+0.50Lr	6.79 psi	170.00psi	0.03992	OK
+1.20D+0.50S	30.20 psi	170.00psi	0.1776	OK
+1.20D+1.60Lr	11.35 psi	170.00psi	0.06674	OK
+1.20D+1.60S	86.26 psi	170.00psi	0.5074	OK
+1.20D+0.20S	14.91 psi	170.00psi	0.0877	OK
+0.90D	3.54 psi	170.00psi	0.0208	OK

26-8

Steel Beam

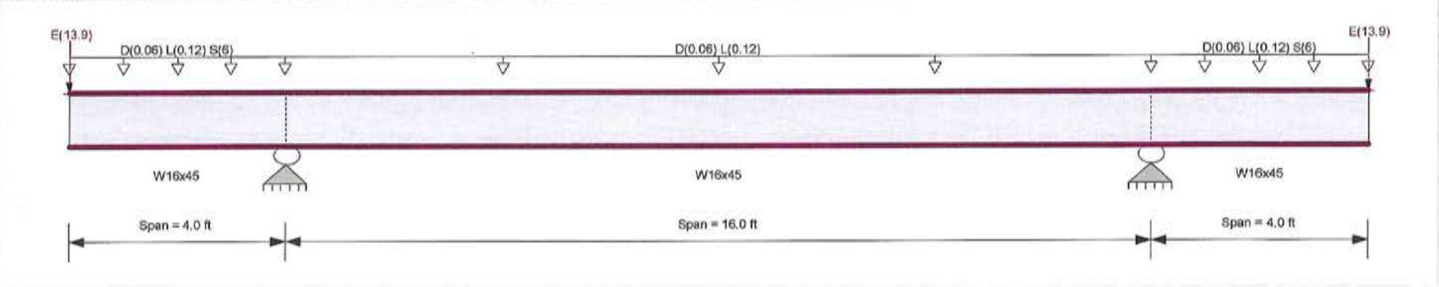
Description : FELDSPAR UVN LOWER FLOOR - B22

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.020, L = 0.040, S = 2.0 ksf, Tributary Width = 3.0 ft, (FLOOR)
Point Load : E = 13.90 k @ 0.0 ft, (HOLD DOWN OVERSTRENGTH)
Load for Span Number 2
Uniform Load : D = 0.020, L = 0.040 ksf, Tributary Width = 3.0 ft, (FLOOR)
Load for Span Number 3
Uniform Load : D = 0.020, L = 0.040, S = 2.0 ksf, Tributary Width = 3.0 ft, (FLOOR)
Point Load : E = 13.90 k @ 4.0 ft, (HOLD DOWN OVERSTRENGTH)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.325 : 1**	**Maximum Shear Stress Ratio =**	**0.235 : 1**
Section used for this span	**W16x45**	Section used for this span	**W16x45**
Ma : Applied	66.750 k-ft	Va : Applied	26.078 k
Mn / Omega : Allowable	205.339 k-ft	Vn/Omega : Allowable	111.090 k
Load Combination	+D+0.750L+0.750S+0.5250E	Load Combination	+D+0.750L+0.750S+0.5250E
Location of maximum on span	16.000ft	Location of maximum on span	16.000 ft
Span # where maximum occurs	Span # 2	Span # where maximum occurs	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.211 in	Ratio =	454 >=360
Max Upward Transient Deflection	-0.183 in	Ratio =	1,047 >=360
Max Downward Total Deflection	0.235 in	Ratio =	409 >=240.
Max Upward Total Deflection	-0.203 in	Ratio =	946 >=240.

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 4.00 ft	1	0.004	0.008		-0.84	0.84	342.92	205.34	1.00	1.00	0.84	166.64	111.09
Dsgn. L = 16.00 ft	2	0.012	0.008	2.52	-0.84	2.52	342.92	205.34	1.00	1.00	0.84	166.64	111.09
Dsgn. L = 4.00 ft	3	0.004	0.004		-0.84	0.84	342.92	205.34	1.00	1.00	0.42	166.64	111.09
+D+L													
Dsgn. L = 4.00 ft	1	0.009	0.016		-1.80	1.80	342.92	205.34	1.00	1.00	1.80	166.64	111.09
Dsgn. L = 16.00 ft	2	0.026	0.016	5.40	-1.80	5.40	342.92	205.34	1.00	1.00	1.80	166.64	111.09
Dsgn. L = 4.00 ft	3	0.009	0.008		-1.80	1.80	342.92	205.34	1.00	1.00	0.90	166.64	111.09
+D+S													
Dsgn. L = 4.00 ft	1	0.238	0.220		-48.84	48.84	342.92	205.34	1.00	1.00	24.42	166.64	111.09
Dsgn. L = 16.00 ft	2	0.238	0.220	-0.00	-48.84	48.84	342.92	205.34	1.00	1.00	24.42	166.64	111.09
Dsgn. L = 4.00 ft	3	0.238	0.220		-48.84	48.84	342.92	205.34	1.00	1.00	24.42	166.64	111.09
+D+0.750L													
Dsgn. L = 4.00 ft	1	0.008	0.014		-1.56	1.56	342.92	205.34	1.00	1.00	1.56	166.64	111.09
Dsgn. L = 16.00 ft	2	0.023	0.014	4.68	-1.56	4.68	342.92	205.34	1.00	1.00	1.56	166.64	111.09
Dsgn. L = 4.00 ft	3	0.008	0.007		-1.56	1.56	342.92	205.34	1.00	1.00	0.78	166.64	111.09
+D+0.750L+0.750S													
Dsgn. L = 4.00 ft	1	0.183	0.169		-37.56	37.56	342.92	205.34	1.00	1.00	18.78	166.64	111.09

Steel Beam

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Description : FELDSPAR UVN LOWER FLOOR - B22

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 16.00 ft	2	0.183	0.169	-0.00	-37.56	37.56	342.92	205.34	1.00	1.00	18.78	166.64	111.09
Dsgn. L = 4.00 ft	3	0.183	0.169		-37.56	37.56	342.92	205.34	1.00	1.00	18.78	166.64	111.09
+D+0.70E													
Dsgn. L = 4.00 ft	1	0.194	0.091		-39.76	39.76	342.92	205.34	1.00	1.00	10.15	166.64	111.09
Dsgn. L = 16.00 ft	2	0.194	0.091	-0.00	-39.76	39.76	342.92	205.34	1.00	1.00	10.15	166.64	111.09
Dsgn. L = 4.00 ft	3	0.194	0.091		-39.76	39.76	342.92	205.34	1.00	1.00	10.15	166.64	111.09
+D-0.70E													
Dsgn. L = 4.00 ft	1	0.185	0.088	38.08		38.08	342.92	205.34	1.00	1.00	9.73	166.64	111.09
Dsgn. L = 16.00 ft	2	0.202	0.084	41.44	38.08	41.44	342.92	205.34	1.00	1.00	9.31	166.64	111.09
Dsgn. L = 4.00 ft	3	0.185	0.088	38.08		38.08	342.92	205.34	1.00	1.00	9.73	166.64	111.09
+D+0.750L+0.750S+0.5250E													
Dsgn. L = 4.00 ft	1	0.325	0.235		-66.75	66.75	342.92	205.34	1.00	1.00	26.08	166.64	111.09
Dsgn. L = 16.00 ft	2	0.325	0.235	-0.00	-66.75	66.75	342.92	205.34	1.00	1.00	26.08	166.64	111.09
Dsgn. L = 4.00 ft	3	0.325	0.235		-66.75	66.75	342.92	205.34	1.00	1.00	26.08	166.64	111.09
+D+0.750L+0.750S-0.5250E													
Dsgn. L = 4.00 ft	1	0.041	0.103	5.67	-8.37	8.37	342.92	205.34	1.00	1.00	11.48	166.64	111.09
Dsgn. L = 16.00 ft	2	0.041	0.103	-0.00	-8.37	8.37	342.92	205.34	1.00	1.00	11.48	166.64	111.09
Dsgn. L = 4.00 ft	3	0.041	0.103	5.67	-8.37	8.37	342.92	205.34	1.00	1.00	11.48	166.64	111.09
+0.60D													
Dsgn. L = 4.00 ft	1	0.002	0.005		-0.50	0.50	342.92	205.34	1.00	1.00	0.50	166.64	111.09
Dsgn. L = 16.00 ft	2	0.007	0.005	1.51	-0.50	1.51	342.92	205.34	1.00	1.00	0.50	166.64	111.09
Dsgn. L = 4.00 ft	3	0.002	0.002		-0.50	0.50	342.92	205.34	1.00	1.00	0.25	166.64	111.09
+0.60D+0.70E													
Dsgn. L = 4.00 ft	1	0.192	0.090		-39.42	39.42	342.92	205.34	1.00	1.00	9.98	166.64	111.09
Dsgn. L = 16.00 ft	2	0.192	0.090	-0.00	-39.42	39.42	342.92	205.34	1.00	1.00	9.98	166.64	111.09
Dsgn. L = 4.00 ft	3	0.192	0.090		-39.42	39.42	342.92	205.34	1.00	1.00	9.98	166.64	111.09
+0.60D-0.70E													
Dsgn. L = 4.00 ft	1	0.187	0.088	38.42		38.42	342.92	205.34	1.00	1.00	9.73	166.64	111.09
Dsgn. L = 16.00 ft	2	0.197	0.085	40.43	38.42	40.43	342.92	205.34	1.00	1.00	9.48	166.64	111.09
Dsgn. L = 4.00 ft	3	0.187	0.088	38.42		38.42	342.92	205.34	1.00	1.00	9.73	166.64	111.09

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S+0.5250E	1	0.2349	0.000		0.0000	0.000
	2	0.0000	0.000	+D+0.750L+0.750S+0.5250E	-0.2030	8.000
+D+0.750L+0.750S+0.5250E	3	0.2346	4.000		0.0000	8.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum		27.638	27.638	
Overall MINimum		0.756	0.756	
D Only		1.260	1.260	
+D+L		2.700	2.700	
+D+S		25.260	25.260	
+D+0.750L		2.340	2.340	
+D+0.750L+0.750S		20.340	20.340	
+D+0.70E		10.990	10.990	
+D+0.750L+0.750S+0.5250E		27.638	27.638	
+0.60D		0.756	0.756	
+0.60D+0.70E		10.486	10.486	
L Only		1.440	1.440	
S Only		24.000	24.000	
E Only		13.900	13.900	

Concrete Column

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Description : B22 SUPPORT

Code References

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

General Information

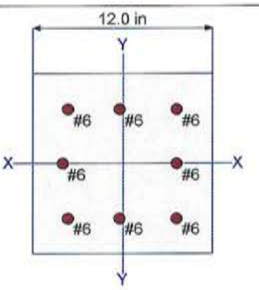
f'c : Concrete 28 day strength = 2.50 ksi
E = = 3,122.0 ksi
Density = 150.0 pcf
β = 0.850
fy - Main Rebar = 60.0 ksi
E - Main Rebar = 29,000.0 ksi
Allow. Reinforcing Limits *ASTM A615 Bars Used*
Min. Reinf. = 1.0 %
Max. Reinf. = 8.0 %

Overall Column Height = 6.0 ft
End Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 6.0 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for X-X Axis buckling = 6.0 ft, K = 1.0

Column Cross Section

Column Dimensions : 12.0in Square Column, Column Edge to Rebar Edge Cover = 2.0in

Column Reinforcing : 4 - #6 bars @ corners,, 1 - #6 bars top & bottom between corner bars, 1 - #6 bars left & right between corner bars



Applied Loads

Entered loads are factored per load combinations specified by user.

Column self weight included : 900.0 lbs * Dead Load Factor
AXIAL LOADS . . .
MAX: Axial Load at 6.0 ft above base, D = 10.0, LR = 10.0, L = 5.0, S = 90.0 k

DESIGN SUMMARY

Load Combination +1.20D+0.50L+1.60S
Location of max.above base 5.960 ft

Maximum Stress Ratio **0.604 : 1**
Ratio = (Pu^2+Mu^2)^.5 / (PhiPn^2+PhiMn^2)^.5
Pu = 159.580 k φ * Pn = 264.051 k
Mu-x = 0.0 k-ft φ * Mn-x = 0.0 k-ft
Mu-y = 12.766 k-ft φ * Mn-y = 20.863 k-ft
Mu Angle = 90.0 deg
Mu at Angle = 12.766 k-ft φMn at Angle = 21.116 k-ft
Pn & Mn values located at Pu-Mu vector intersection with capacity curve

Column Capacities . . .
Pnmax : Nominal Max. Compressive Axial Capacity 509.72 k
Pnmin : Nominal Min. Tension Axial Capacity -211.20 k
φ Pn, max : Usable Compressive Axial Capacity 265.054 k
φ Pn, min : Usable Tension Axial Capacity -137.280 k

Maximum SERVICE Load Reactions . .
Top along Y-Y 0.0 k Bottom along Y-Y 0.0 k
Top along X-X 0.0 k Bottom along X-X 0.0 k

Maximum SERVICE Load Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination :
Along X-X 0.0 in at 0.0 ft above base
for load combination :

General Section Information . φ = 0.650 β = 0.850 θ = 0.80
ρ : % Reinforcing 2.444 % Rebar % Ok
Reinforcing Area 3.520 in^2
Concrete Area 144.0 in^2

Governing Load Combination Results

Governing Factored Load Combination	Moment		Dist. from	Axial Load k		Bending Analysis k-ft							Utilization
	X-X	Y-Y	base ft	Pu	φ * Pn	δx	δx * Mux	δy	δy * Muy	Alpha (deg)	δ Mu	φ Mn	Ratio
+1.40D		M2,min	5.96	15.26	264.05			1.000	1.22	90.000	1.22	21.12	0.058
+1.20D+0.50Lr+1.60L		M2,min	5.96	26.08	264.05			1.000	2.09	90.000	2.09	21.12	0.099
+1.20D+1.60L+0.50S		M2,min	5.96	66.08	264.05			1.000	5.29	90.000	5.29	21.12	0.250

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Concrete Column

Description : B22 SUPPORT

Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	φ * Pn	δ x	δ x * Mux	δ y	δ y * Muy	Bending Analysis k-ft Alpha (deg)	δ Mu	φ Mn	Utilization Ratio
+1.20D+1.60Lr+0.50L		M2,min	5.96	31.58	264.05			1.000	2.53	90.000	2.53	21.12	0.120
+1.20D+1.60Lr		M2,min	5.96	29.08	264.05			1.000	2.33	90.000	2.33	21.12	0.110
+1.20D+0.50L+1.60S		M2,min	5.96	159.58	264.05			1.000	12.77	90.000	12.77	21.12	0.604
+1.20D+1.60S		M2,min	5.96	157.08	264.05			1.000	12.57	90.000	12.57	21.12	0.595
+1.20D+0.50Lr+0.50L		M2,min	5.96	20.58	264.05			1.000	1.65	90.000	1.65	21.12	0.078
+1.20D+0.50L+0.50S		M2,min	5.96	60.58	264.05			1.000	4.85	90.000	4.85	21.12	0.229
+1.20D+0.50L+0.20S		M2,min	5.96	33.58	264.05			1.000	2.69	90.000	2.69	21.12	0.127
+0.90D		M2,min	5.96	9.81	264.05			1.000	0.78	90.000	0.78	21.12	0.037

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	My - End Moments @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
D Only					10.900				
+D+L					15.900				
+D+Lr					20.900				
+D+S					100.900				
+D+0.750Lr+0.750L					22.150				
+D+0.750L+0.750S					82.150				
+0.60D					6.540				
Lr Only					10.000				
L Only					5.000				
S Only					90.000				

Maximum Moment Reactions

Note: Only non-zero reactions are listed.

Load Combination	Moment About X-X Axis @ Base	Moment About X-X Axis @ Top		Moment About Y-Y Axis @ Base	Moment About Y-Y Axis @ Top	
D Only			k-ft			k-ft
+D+L			k-ft			k-ft
+D+Lr			k-ft			k-ft
+D+S			k-ft			k-ft
+D+0.750Lr+0.750L			k-ft			k-ft
+D+0.750L+0.750S			k-ft			k-ft
+0.60D			k-ft			k-ft
Lr Only			k-ft			k-ft
L Only			k-ft			k-ft
S Only			k-ft			k-ft

Maximum Deflections for Load Combinations

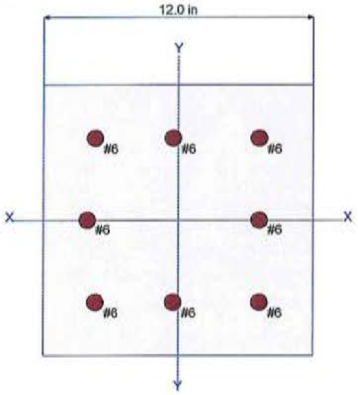
Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+L	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr+0.750L	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750L+0.750S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
Lr Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
L Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
S Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

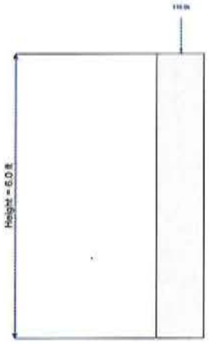
2512

Concrete Column

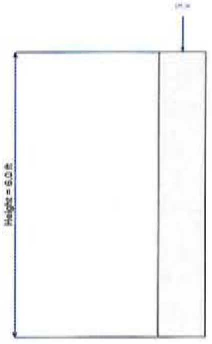
Description : B22 SUPPORT

Sketches



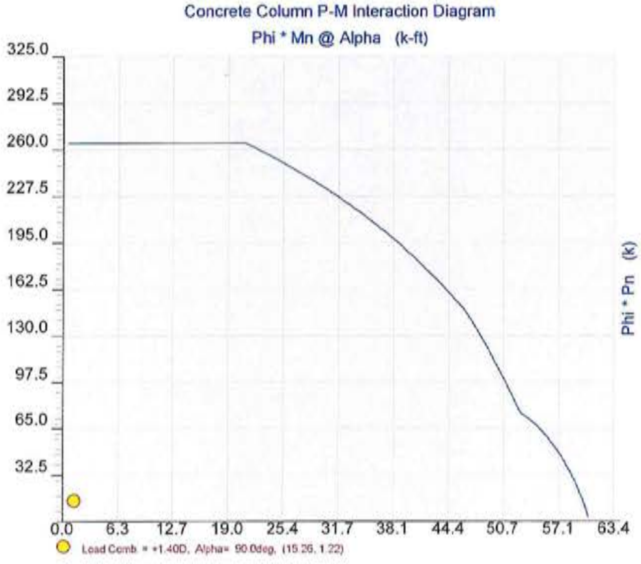


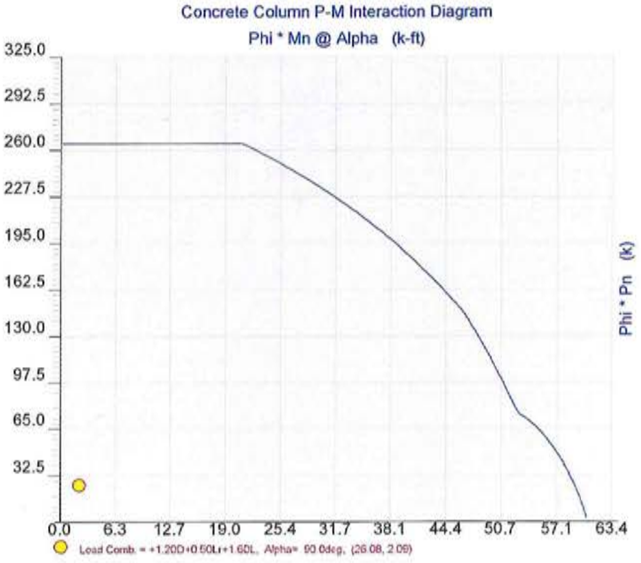
Looking along X-X Axis



Looking along Y-Y Axis

Interaction Diagrams





Concrete Column

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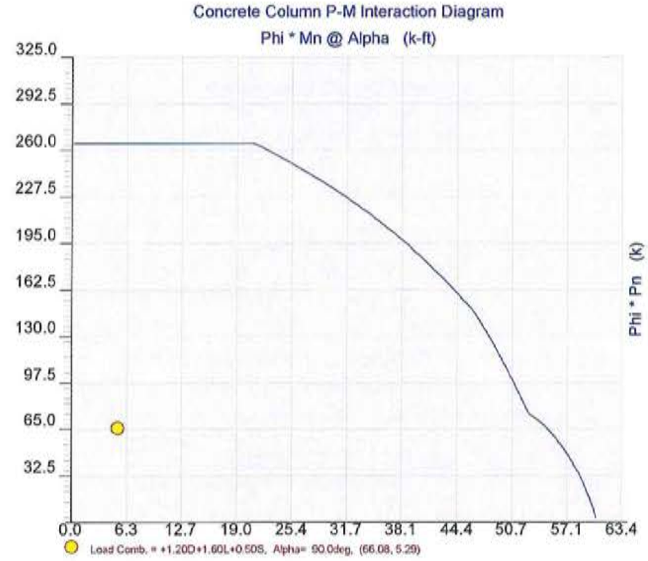
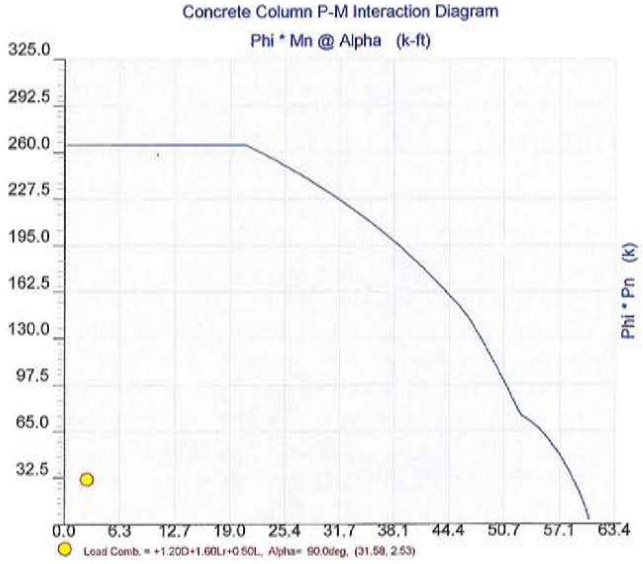
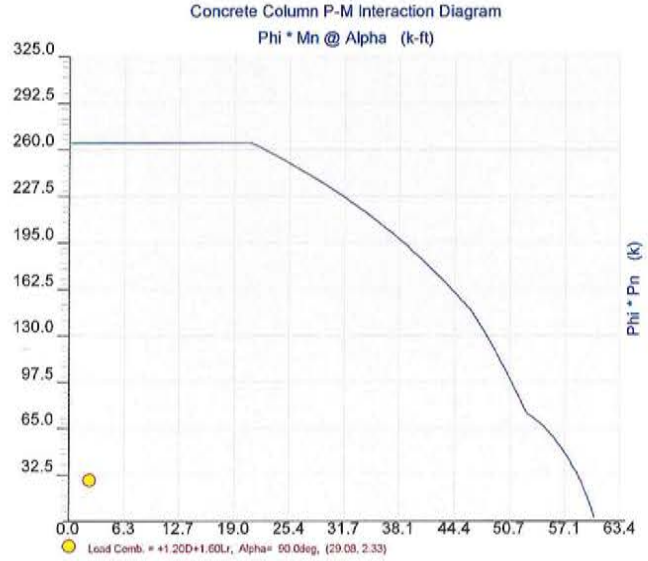
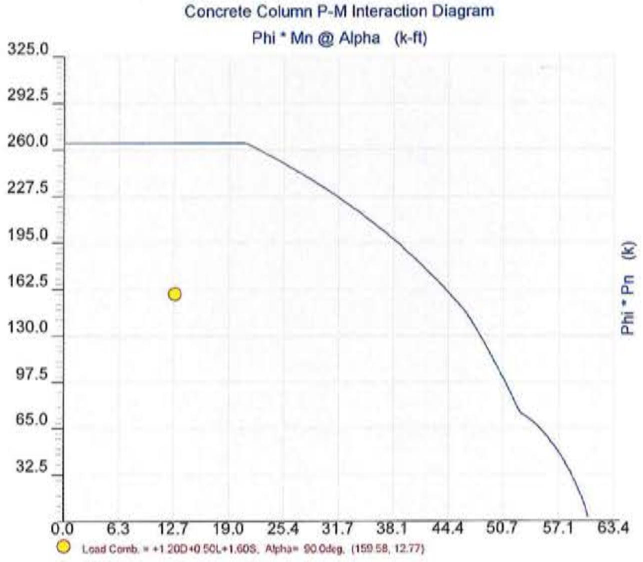
Description : B22 SUPPORT



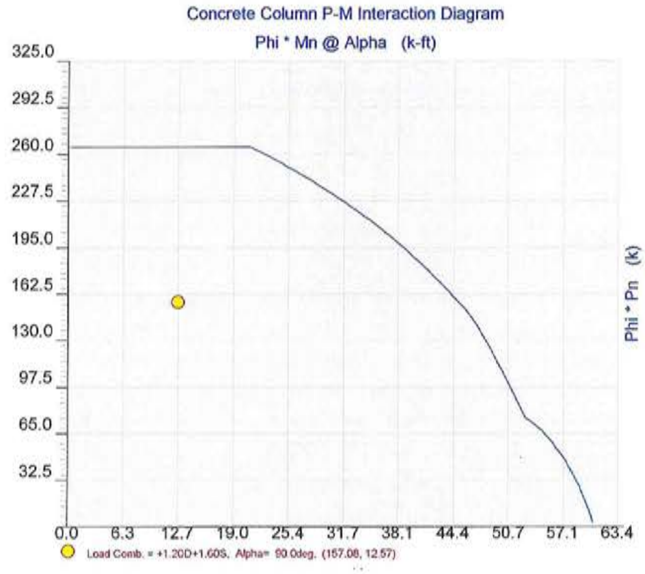


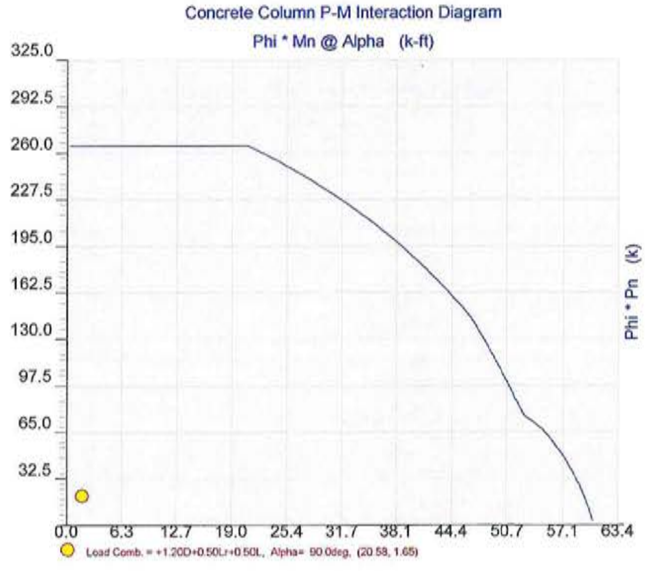


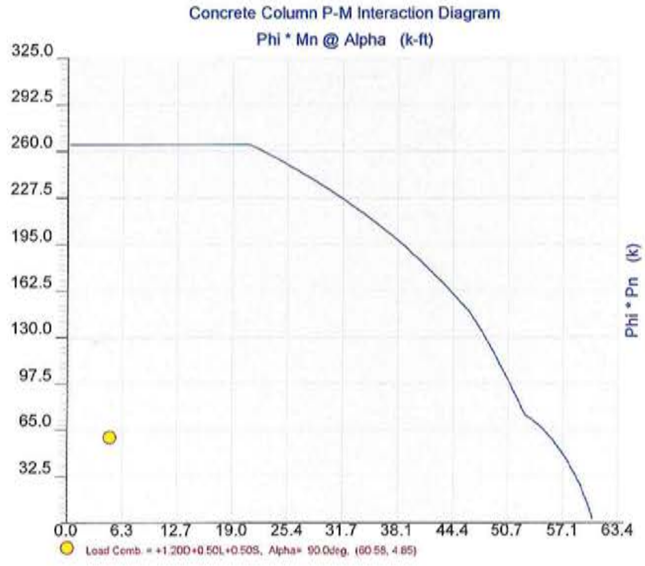


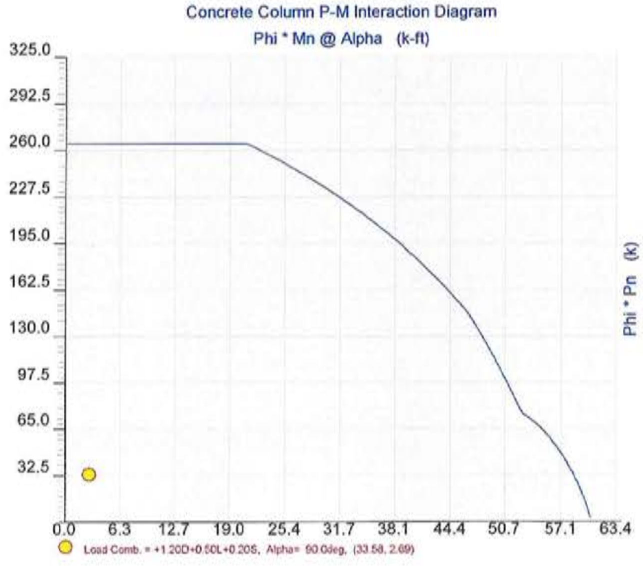
Concrete Column

Description : B22 SUPPORT









Concrete Column

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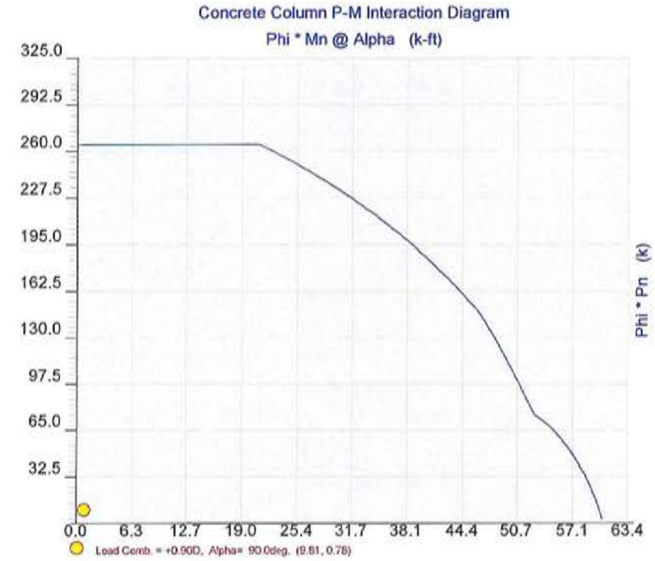
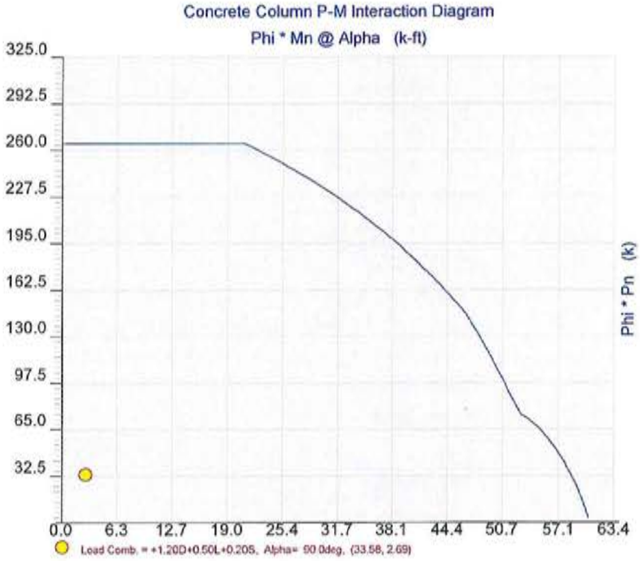
Description : B22 SUPPORT





General Footing

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Licensee : RICHMOND HOFFMAYER

Description : MAX PAD: FELDSPAR UVN

Code References

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

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 = 0.850

Analysis Settings
Min Steel % Bending Reinf. =
Min Allow % Temp Reinf. = 0.00180
Min. Overturning Safety Factor = 1.50 : 1
Min. Sliding Safety Factor = 1.50 : 1
Add Ftg Wt for Soil Pressure : No
Use ftg wt for stability, moments & shears : No
Add Pedestal Wt for Soil Pressure : No
Use Pedestal wt for stability, mom & shear : No

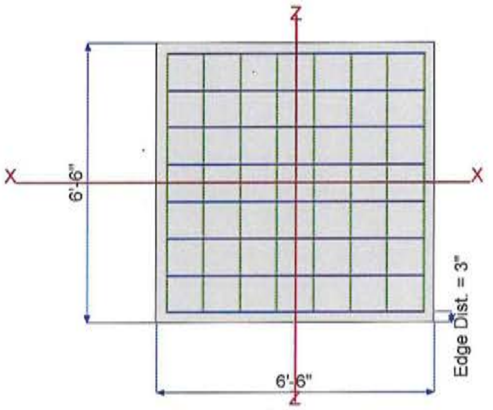
Soil Design Values
Allowable Soil Bearing = 2.50 ksf
Increase Bearing By Footing Weight = No
Soil Passive Resistance (for Sliding) = 180.0 pcf
Soil/Concrete Friction Coeff. = 0.450

Increases based on footing Depth
Footing base depth below soil surface = 2.0 ft
Allow press. increase per foot of depth = 0.0 ksf
when footing base is below = 0.0 ft

Increases based on footing plan dimension
Allowable pressure increase per foot of depth = 0.0 ksf
when max. length or width is greater than = 0.0 ft

Dimensions

Width parallel to X-X Axis = 6.50 ft
Length parallel to Z-Z Axis = 6.50 ft
Footing Thickness = 24.0 in

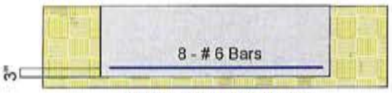


Pedestal dimensions...
px : parallel to X-X Axis = 0.0 in
pz : parallel to Z-Z Axis = 0.0 in
Height = 0.0 in
Rebar Centerline to Edge of Concrete...
at Bottom of footing = 3.0 in

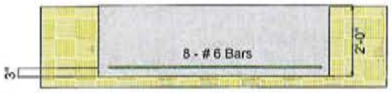
Reinforcing

Bars parallel to X-X Axis
Number of Bars = 8
Reinforcing Bar Size = # 6
Bars parallel to Z-Z Axis
Number of Bars = 8
Reinforcing Bar Size = # 6

Bandwidth Distribution Check (ACI 15.4.4.2)
Direction Requiring Closer Separation n/a
Bars required within zone n/a
Bars required on each side of zone n/a



X-X Section Looking to +Z



Z-Z Section Looking to +X

Applied Loads

		D	Lr	L	S	W	E	H
P : Column Load	=	10.0	10.0	5.0	90.0	0.0	0.0	0.0 k
OB : Overburden	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0 ksf
M-xx	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0 k-ft
M-zz	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0 k-ft
V-x	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0 k
V-z	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0 k

General Footing

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Description : MAX PAD: FELDSPAR UVN

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.9688	Soil Bearing	2.422 ksf	2.50 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
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	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.3993	Z Flexure (+X)	19.813 k-ft/ft	49.623 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.3993	Z Flexure (-X)	19.813 k-ft/ft	49.623 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.3993	X Flexure (+Z)	19.813 k-ft/ft	49.623 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.3993	X Flexure (-Z)	19.813 k-ft/ft	49.623 k-ft/ft	+1.20D+0.50L+1.60S
PASS	0.2656	1-way Shear (+X)	22.578 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.2656	1-way Shear (-X)	22.578 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.2656	1-way Shear (+Z)	22.578 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.2656	1-way Shear (-Z)	22.578 psi	85.0 psi	+1.20D+0.50L+1.60S
PASS	0.4910	2-way Punching	83.463 psi	170.0 psi	+1.20D+0.50L+1.60S

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	2.50	n/a	0.0	0.2917	0.2917	n/a	n/a	0.117
X-X, +D+L	2.50	n/a	0.0	0.410	0.410	n/a	n/a	0.164
X-X, +D+Lr	2.50	n/a	0.0	0.5284	0.5284	n/a	n/a	0.211
X-X, +D+S	2.50	n/a	0.0	2.422	2.422	n/a	n/a	0.969
X-X, +D+0.750Lr+0.750L	2.50	n/a	0.0	0.5580	0.5580	n/a	n/a	0.223
X-X, +D+0.750L+0.750S	2.50	n/a	0.0	1.978	1.978	n/a	n/a	0.791
X-X, +0.60D	2.50	n/a	0.0	0.1750	0.1750	n/a	n/a	0.070
Z-Z, D Only	2.50	0.0	n/a	n/a	n/a	0.2917	0.2917	0.117
Z-Z, +D+L	2.50	0.0	n/a	n/a	n/a	0.410	0.410	0.164
Z-Z, +D+Lr	2.50	0.0	n/a	n/a	n/a	0.5284	0.5284	0.211
Z-Z, +D+S	2.50	0.0	n/a	n/a	n/a	2.422	2.422	0.969
Z-Z, +D+0.750Lr+0.750L	2.50	0.0	n/a	n/a	n/a	0.5580	0.5580	0.223
Z-Z, +D+0.750L+0.750S	2.50	0.0	n/a	n/a	n/a	1.978	1.978	0.791
Z-Z, +0.60D	2.50	0.0	n/a	n/a	n/a	0.1750	0.1750	0.070

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	1.750	+Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.40D	1.750	-Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+0.50Lr+1.60L	3.125	+Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+0.50Lr+1.60L	3.125	-Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+1.60L+0.50S	8.125	+Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+1.60L+0.50S	8.125	-Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+1.60Lr+0.50L	3.813	+Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+1.60Lr+0.50L	3.813	-Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+1.60Lr	3.50	+Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+1.60Lr	3.50	-Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+0.50L+1.60S	19.813	+Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+0.50L+1.60S	19.813	-Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+1.60S	19.50	+Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+1.60S	19.50	-Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+0.50Lr+0.50L	2.438	+Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+0.50Lr+0.50L	2.438	-Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+0.50L+0.50S	7.438	+Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+0.50L+0.50S	7.438	-Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+0.50L+0.20S	4.063	+Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +1.20D+0.50L+0.20S	4.063	-Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +0.90D	1.125	+Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
X-X, +0.90D	1.125	-Z	Bottom	0.5184	Min Temp %	0.5415	49.623	OK

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General Footing

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Description : MAX PAD: FELDSPAR UVN

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
Z-Z, +1.40D	1.750	-X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.40D	1.750	+X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+0.50Lr+1.60L	3.125	-X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+0.50Lr+1.60L	3.125	+X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+1.60L+0.50S	8.125	-X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+1.60L+0.50S	8.125	+X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+1.60Lr+0.50L	3.813	-X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+1.60Lr+0.50L	3.813	+X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+1.60Lr	3.50	-X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+1.60Lr	3.50	+X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+0.50L+1.60S	19.813	-X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+0.50L+1.60S	19.813	+X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+1.60S	19.50	-X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+1.60S	19.50	+X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+0.50Lr+0.50L	2.438	-X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+0.50Lr+0.50L	2.438	+X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+0.50L+0.50S	7.438	-X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+0.50L+0.50S	7.438	+X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+0.50L+0.20S	4.063	-X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +1.20D+0.50L+0.20S	4.063	+X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +0.90D	1.125	-X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK
Z-Z, +0.90D	1.125	+X	Bottom	0.5184	Min Temp %	0.5415	49.623	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	1.99 psi	1.99 psi	1.99 psi	1.99 psi	1.99 psi	85.00 psi	0.02	0.00
+1.20D+0.50Lr+1.60L	3.56 psi	3.56 psi	3.56 psi	3.56 psi	3.56 psi	85.00 psi	0.04	0.00
+1.20D+1.60L+0.50S	9.26 psi	9.26 psi	9.26 psi	9.26 psi	9.26 psi	85.00 psi	0.11	0.00
+1.20D+1.60Lr+0.50L	4.35 psi	4.35 psi	4.35 psi	4.35 psi	4.35 psi	85.00 psi	0.05	0.00
+1.20D+1.60Lr	3.99 psi	3.99 psi	3.99 psi	3.99 psi	3.99 psi	85.00 psi	0.05	0.00
+1.20D+0.50L+1.60S	22.58 psi	22.58 psi	22.58 psi	22.58 psi	22.58 psi	85.00 psi	0.27	0.00
+1.20D+1.60S	22.22 psi	22.22 psi	22.22 psi	22.22 psi	22.22 psi	85.00 psi	0.26	0.00
+1.20D+0.50Lr+0.50L	2.78 psi	2.78 psi	2.78 psi	2.78 psi	2.78 psi	85.00 psi	0.03	0.00
+1.20D+0.50L+0.50S	8.48 psi	8.48 psi	8.48 psi	8.48 psi	8.48 psi	85.00 psi	0.10	0.00
+1.20D+0.50L+0.20S	4.63 psi	4.63 psi	4.63 psi	4.63 psi	4.63 psi	85.00 psi	0.05	0.00
+0.90D	1.28 psi	1.28 psi	1.28 psi	1.28 psi	1.28 psi	85.00 psi	0.02	0.00

All units k

Two-Way "Punching" Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	7.37 psi	170.00psi	0.04337	OK
+1.20D+0.50Lr+1.60L	13.17 psi	170.00psi	0.07744	OK
+1.20D+1.60L+0.50S	34.23 psi	170.00psi	0.2013	OK
+1.20D+1.60Lr+0.50L	16.06 psi	170.00psi	0.09448	OK
+1.20D+1.60Lr	14.74 psi	170.00psi	0.08673	OK
+1.20D+0.50L+1.60S	83.46 psi	170.00psi	0.491	OK
+1.20D+1.60S	82.15 psi	170.00psi	0.4832	OK
+1.20D+0.50Lr+0.50L	10.27 psi	170.00psi	0.0604	OK
+1.20D+0.50L+0.50S	31.33 psi	170.00psi	0.1843	OK
+1.20D+0.50L+0.20S	17.11 psi	170.00psi	0.1007	OK
+0.90D	4.74 psi	170.00psi	0.02788	OK

Concrete Beam

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Description : FELDSPAR UVN LOWER FLOOR - B22 - CONCRETE

REPLACE STEEL W/ CONC BEAM

CODE REFERENCES

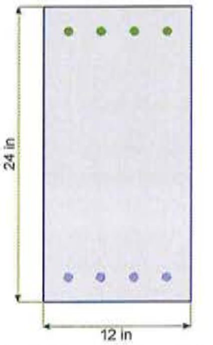
Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

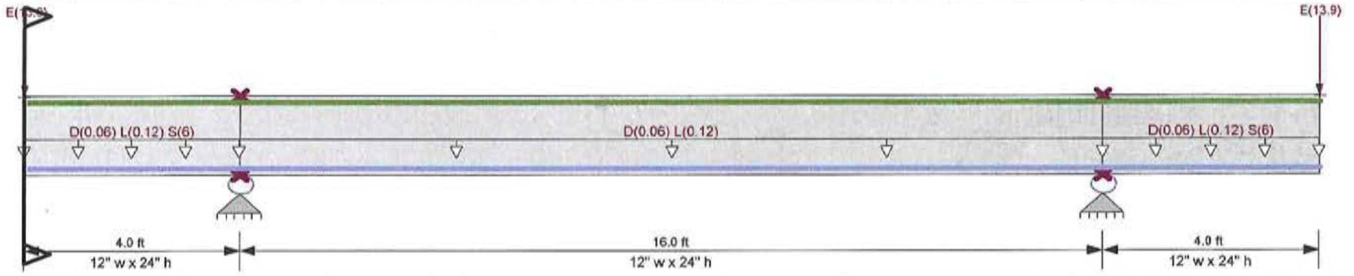
Material Properties

f'c = 2.50 ksi
fr = $f'c^{1/2}$ * 7.50 = 375.0 psi
ψ Density = 145.0 pcf
λ LtWt Factor = 1.0
Elastic Modulus = 3,122.0 ksi
fy - Main Rebar = 60.0 ksi
E - Main Rebar = 29,000.0 ksi

ϕ Phi Values Flexure : 0.90
Shear : 0.750
β_1 = 0.850

Fy - Stirrups 40.0 ksi
E - Stirrups = 29,000.0 ksi
Stirrup Bar Size # 3
Number of Resisting Legs Per Stirrup = 2





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 24.0 in
Span #1 Reinforcing....
4-#6 at 2.0 in from Bottom, from 0.0 to 4.0 ft in this span
4-#6 at 2.0 in from Top, from 0.0 to 4.0 ft in this span
Span #2 Reinforcing....
4-#6 at 2.0 in from Bottom, from 0.0 to 16.0 ft in this span
4-#6 at 2.0 in from Top, from 0.0 to 16.0 ft in this span
Span #3 Reinforcing....
4-#6 at 2.0 in from Bottom, from 0.0 to 4.0 ft in this span
4-#6 at 2.0 in from Top, from 0.0 to 4.0 ft in this span

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 = 0.020, L = 0.040, S = 2.0 ksf, Tributary Width = 3.0 ft, (FLOOR)
Point Load : E = 13.90 k @ 0.0 ft, (HOLD DOWN OVERSTRENGTH)
Load for Span Number 2
Uniform Load : D = 0.020, L = 0.040 ksf, Tributary Width = 3.0 ft, (FLOOR)
Load for Span Number 3
Uniform Load : D = 0.020, L = 0.040, S = 2.0 ksf, Tributary Width = 3.0 ft, (FLOOR)
Point Load : E = 13.90 k @ 4.0 ft, (HOLD DOWN OVERSTRENGTH)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.498** : 1
Section used for this span **Typical Section**
Mu : Applied -80.640 k-ft
Mn * Phi : Allowable 162.057 k-ft
Location of maximum on span 0.000 ft
Span # where maximum occurs Span # 2

Maximum Deflection			
Max Downward Transient Deflection	0.163 in	Ratio =	590>=36
Max Upward Transient Deflection	-0.145 in	Ratio =	1325>=36
Max Downward Total Deflection	0.184 in	Ratio =	520>=18
Max Upward Total Deflection	-0.152 in	Ratio =	1267>=18

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum		30.578	30.577	
Overall MINimum		1.440	1.440	
D Only		4.200	4.200	

Concrete Beam

Lic. # : KW-06002886 **Licensee : RICHMOND HOFFMAYER**

Description : FELDSPAR UVN LOWER FLOOR - B22 - CONCRETE

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
+D+L		5.640	5.640	
+D+S		28.200	28.200	
+D+0.750L		5.280	5.280	
+D+0.750L+0.750S		23.280	23.280	
+D+0.70E		13.930	13.930	
+D+0.750L+0.750S+0.5250E		30.578	30.577	
+0.60D		2.520	2.520	
+0.60D+0.70E		12.250	12.250	
L Only		1.440	1.440	
S Only		24.000	24.000	
E Only		13.900	13.900	

Shear Stirrup Requirements

Between 0.00 to 0.00 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 0.03 to 2.19 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 11.000 in
Between 2.21 to 3.97 ft, PhiVc < Vu, Req'd Vs = 18.196, use stirrups spaced at 7.000 in
Between 4.00 to 19.89 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 20.00 to 21.79 ft, PhiVc < Vu, Req'd Vs = 0.2004, use stirrups spaced at 7.000 in
Between 21.81 to 23.97 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 11.000 in

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S+0.5250E	1	0.1841	0.000	+D+0.750L+0.750S+0.5250E	-0.0124	4.320
+D+0.750L+0.750S+0.5250E	2	0.0033	16.080	+D+0.750L+0.750S+0.5250E	-0.1515	8.000
+D+0.750L+0.750S+0.5250E	3	0.1841	4.000		0.0000	8.000

2e1

Steel Base Plate

Lic. # : KW-06002886

Description : P.C. BASE PLATE

NARROW BASE PLATES

Code References

Calculations per AISC Design Guide # 1, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

General Information

Material Properties

AISC Design Method	Load Resistance Factor Design	Φ_c : LRFD Resistance Factor	0.60
Steel Plate Fy	= 36.0 ksi		
Concrete Support f'c	= 2.50 ksi		
Assumed Bearing Area :	Bearing Area = P / Fp	Allowable Bearing Fp per J8	4.017 ksi

Column & Plate

Column Properties

Steel Section : Pipe4STD

Depth	4.5 in	Area	2.96 in^2
Width	4.5 in	Ixx	in^4
Flange Thickness	0.221 in	Iyy	in^4
Web Thickness	in		

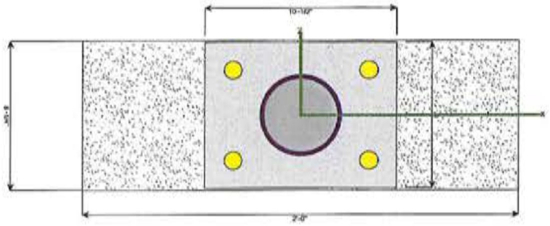


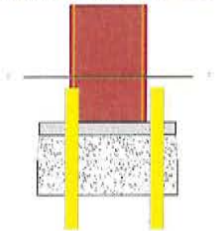
Plate Dimensions

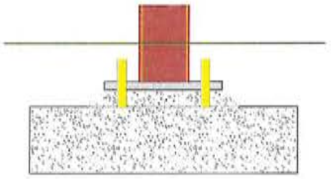
N : Length	8.0 in
B : Width	10.50 in
Thickness	0.750 in

Column assumed welded to base plate.

Support Dimensions

Width along "X"	8.250 in
Length along "Z"	24.0 in

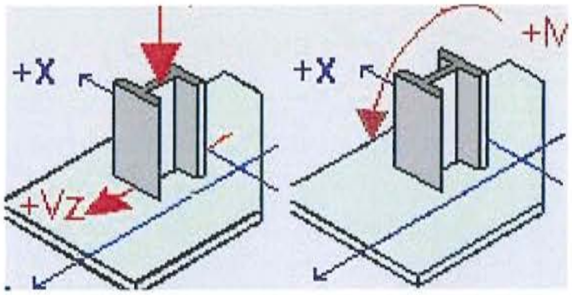




Applied Loads

	P-Y	V-Z	M-X
D : Dead Load	3.807 k	k	k-ft
L : Live	k	k	k-ft
Lr : Roof Live	4.015 k	k	k-ft
S : Snow	49.381 k	k	k-ft
W : Wind................	k	k	k-ft
E : Earthquake	k	k	k-ft
H : Lateral Earth	k	k	k-ft

" P " = Gravity load, "+" sign is downward.
"+" Moments create higher soil pressure at +Z edge.
"+" Shears push plate towards +Z edge.



GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method	Load Resistance Factor Design
Governing Load Combination	+1.20D+1.60S
Governing Load Case Type	Axial Load Only
Design Plate Size	8" x 10 -1/2" x 0 -3/4"
Pu : Axial	83.578 k
Mu : Moment	0.000 k-ft

Mu : Max. Moment	0.173 k-in
fb : Max. Bending Stress	1.228 ksi
Fb : Allowable : Fy * Phi	32.400 ksi
Bending Stress Ratio	0.038
	Bending Stress OK
fu : Max. Plate Bearing Stress	1.706 ksi
Fp : Allowable : min(0.85*f'c*sqrt(A2/A1), 1.7* f'c)*Phi	1.722 ksi
Bearing Stress Ratio	0.991
	Bearing Stress OK

Steel Base Plate

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Description : HSS BASE PLATE ON STEM

Code References

Calculations per AISC Design Guide # 1, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

General Information

Material Properties

AISC Design Method Load Resistance Factor Design
Steel Plate Fy = 36.0 ksi
Concrete Support f'c = 2.50 ksi
Assumed Bearing Area : Bearing Area = P / Fp

Φ_c : LRFD Resistance Factor 0.60

Allowable Bearing Fp per J8 3.814 ksi

Column & Plate

Column Properties

Steel Section : HSS6x6x3/8

Depth	6	in	Area	7.58	in^2
Width	6	in	Ixx		in^4
Flange Thickness	0.349	in	Iyy		in^4
Web Thickness		in			

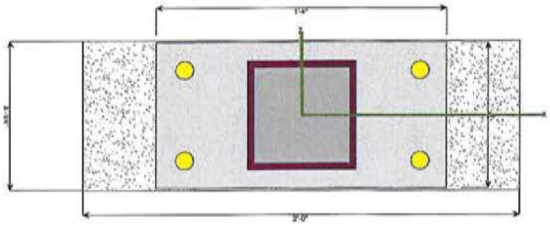
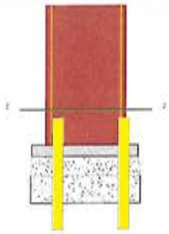


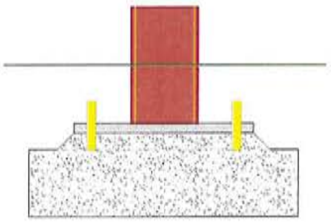
Plate Dimensions

N : Length 8.0 in
B : Width 16.0 in
Thickness 0.8750 in
Column assumed welded to base plate.

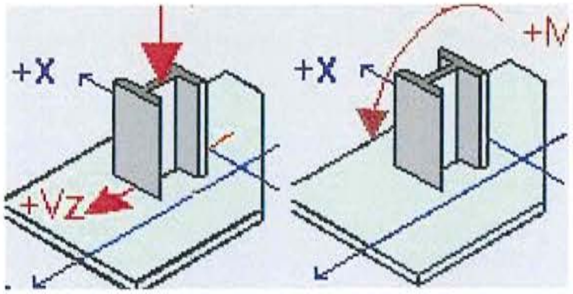
Support Dimensions

Width along "X" 8.250 in
Length along "Z" 24.0 in





Applied Loads



	P-Y		V-Z		M-X	
D : Dead Load	9.050	k		k		k-ft
L : Live	7.440	k		k		k-ft
Lr : Roof Live	4.028	k		k		k-ft
S : Snow	90.670	k		k		k-ft
W : Wind		k		k		k-ft
E : Earthquake		k		k		k-ft
H : Lateral Earth		k		k		k-ft

" P " = Gravity load, "+" sign is downward. "+" Moments create higher soil pressure at +Z edge.
"+" Shears push plate towards +Z edge.

GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method	Load Resistance Factor Design
Governing Load Combination	+1.20D+0.50L+1.60S
Governing Load Case Type	Axial Load Only
Design Plate Size	8" x 1'-4" x 0 -7/8"
Pu : Axial	159.652 k
Mu : Moment	0.000 k-ft

Mu : Max. Moment .... 3.033 k-in
fb : Max. Bending Stress .... 15.846 ksi
Fb : Allowable : 32.400 ksi
Fy * Phi
Bending Stress Ratio 0.489
Bending Stress OK

fu : Max. Plate Bearing Stress .... 1.385 ksi
Fp : Allowable : 1.386 ksi
min(0.85*f'c*sqrt(A2/A1), 1.7* f'c)*Phi
Bearing Stress Ratio 1.000
Bearing Stress OK

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Concrete Column

Description : HSS SUPPORT - INTEGRAL w/ WALL

Code References

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

General Information

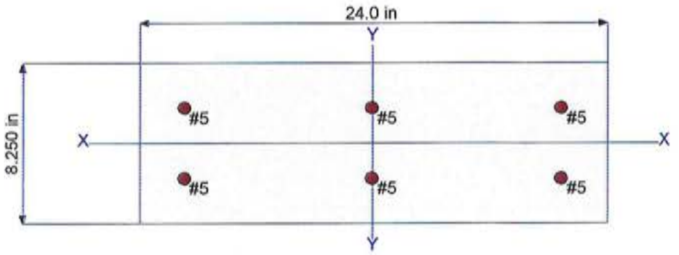
f'c : Concrete 28 day strength = 2.50 ksi
E = = 3,122.0 ksi
Density = 150.0 pcf
β = 0.850
fy - Main Rebar = 60.0 ksi
E - Main Rebar = 29,000.0 ksi
Allow. Reinforcing Limits ASTM A615 Bars Used
Min. Reinf. = 1.0 %
Max. Reinf. = 8.0 %

Overall Column Height = 6.0 ft
End Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 6.0 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for X-X Axis buckling = 6.0 ft, K = 1.0

Column Cross Section

Column Dimensions : 8.250in high x 24.0in Wide, Column Edge to Rebar Edge Cover = 2.0in

Column Reinforcing : 4 - #5 bars @ corners,, 1.0 - #5 bars top & bottom between corner bars



Applied Loads

Entered loads are factored per load combinations specified by user.

Column self weight included : 1,237.50 lbs * Dead Load Factor
AXIAL LOADS . . .
MAX: Axial Load at 6.0 ft above base, D = 10.0, LR = 10.0, L = 5.0, S = 90.0 k

DESIGN SUMMARY

Load Combination +1.20D+0.50L+1.60S
Location of max.above base 5.960 ft

Maximum Stress Ratio **0.582 : 1**
Ratio = (Pu^2+Mu^2)^.5 / (PhiPn^2+PhiMn^2)^.5
Pu = 159.985 k φ * Pn = 274.767 k
Mu-x = 0.0 k-ft φ * Mn-x = 0.0 k-ft
Mu-y = 0.0 k-ft φ * Mn-y = 0.0 k-ft
Mu Angle = 0.0 deg
Mu at Angle = 0.0 k-ft φMn at Angle = 0.0 k-ft

Pn & Mn values located at Pu-Mu vector intersection with capacity curve

Column Capacities . . .
Pnmax : Nominal Max. Compressive Axial Capacity 528.40 k
Pnmin : Nominal Min. Tension Axial Capacity -111.60 k
φ Pn, max : Usable Compressive Axial Capacity 274.767 k
φ Pn, min : Usable Tension Axial Capacity -72.540 k

Maximum SERVICE Load Reactions . .
Top along Y-Y 0.0 k Bottom along Y-Y 0.0 k
Top along X-X 0.0 k Bottom along X-X 0.0 k

Maximum SERVICE Load Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination :
Along X-X 0.0 in at 0.0 ft above base
for load combination :

General Section Information . φ = 0.650 β = 0.850 θ = 0.80
ρ : % Reinforcing 0.9394 % Rebar < Min of 1.0 %
Reinforcing Area 1.860 in^2
Concrete Area 198.0 in^2

Governing Load Combination Results

Governing Factored Load Combination	Moment		Dist. from	Axial Load k		Bending Analysis k-ft							Utilization
	X-X	Y-Y	base ft	Pu	φ * Pn	δx	δx * Mux	δy	δy * Muy	Alpha (deg)	δ Mu	φ Mn	Ratio
+1.40D			5.96	15.73	274.77					0.000			0.057
+1.20D+0.50Lr+1.60L			5.96	26.49	274.77					0.000			0.096
+1.20D+1.60L+0.50S			5.96	66.49	274.77					0.000			0.242

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Concrete Column

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Description : HSS SUPPORT - INTEGRAL w/ WALL

Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	Axial Load k φ * Pn	δ x	δ x * Mux	δ y	δy * Muy	Bending Analysis k-ft Alpha (deg)	δ Mu	φ Mn	Utilization Ratio
+1.20D+1.60Lr+0.50L			5.96	31.99	274.77					0.000			0.116
+1.20D+1.60Lr			5.96	29.49	274.77					0.000			0.107
+1.20D+0.50L+1.60S			5.96	159.99	274.77					0.000			0.582
+1.20D+1.60S			5.96	157.49	274.77					0.000			0.573
+1.20D+0.50Lr+0.50L			5.96	20.99	274.77					0.000			0.076
+1.20D+0.50L+0.50S			5.96	60.99	274.77					0.000			0.222
+1.20D+0.50L+0.20S			5.96	33.99	274.77					0.000			0.124
+0.90D			5.96	10.11	274.77					0.000			0.037

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	My - End Moments @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
D Only					11.238				
+D+L					16.238				
+D+Lr					21.238				
+D+S					101.238				
+D+0.750Lr+0.750L					22.488				
+D+0.750L+0.750S					82.488				
+0.60D					6.743				
Lr Only					10.000				
L Only					5.000				
S Only					90.000				

Maximum Moment Reactions

Note: Only non-zero reactions are listed.

Load Combination	Moment About X-X Axis @ Base	Moment About X-X Axis @ Top		Moment About Y-Y Axis @ Base	Moment About Y-Y Axis @ Top	
D Only			k-ft			k-ft
+D+L			k-ft			k-ft
+D+Lr			k-ft			k-ft
+D+S			k-ft			k-ft
+D+0.750Lr+0.750L			k-ft			k-ft
+D+0.750L+0.750S			k-ft			k-ft
+0.60D			k-ft			k-ft
Lr Only			k-ft			k-ft
L Only			k-ft			k-ft
S Only			k-ft			k-ft

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+L	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr+0.750L	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750L+0.750S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
Lr Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
L Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
S Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Concrete Column

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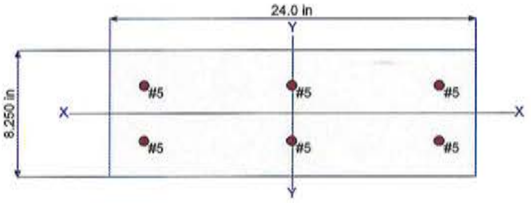
Sketches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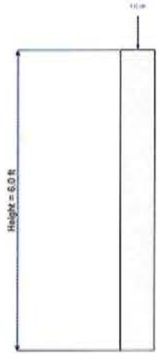
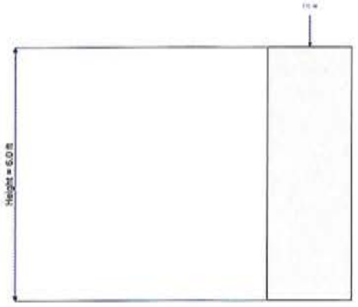
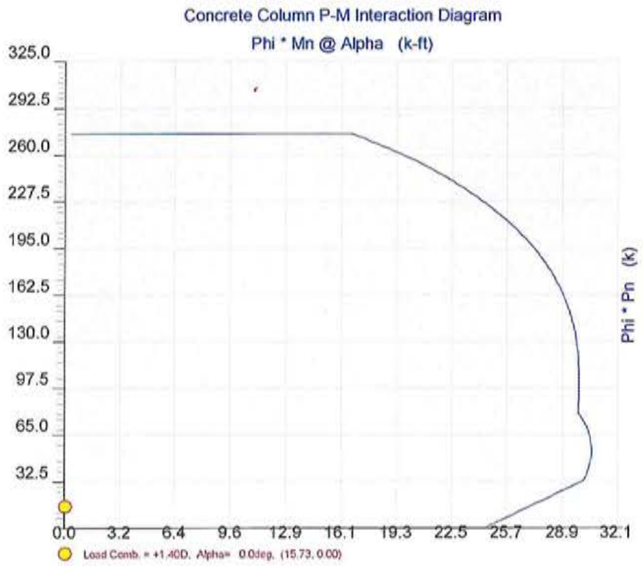

Height = 6.0 ft

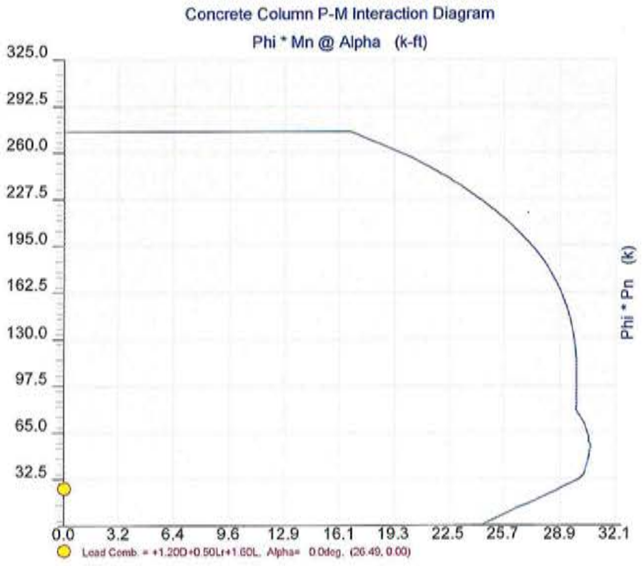
Looking along X-X Axis


Height = 6.0 ft

Looking along Y-Y Axis

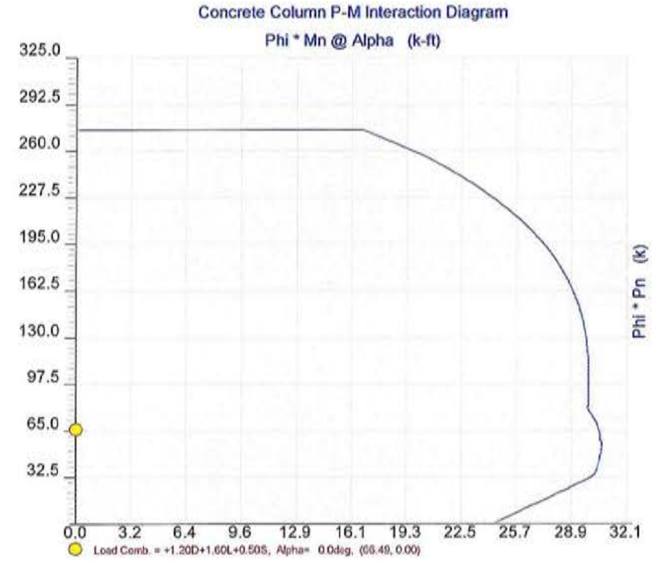
Interaction Diagrams

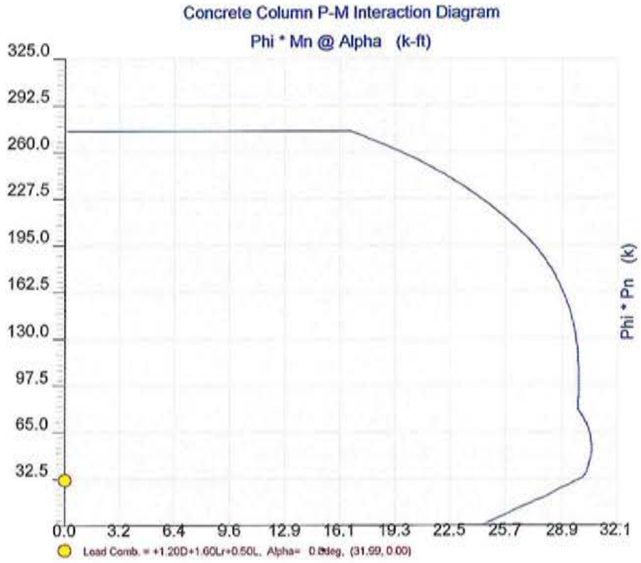


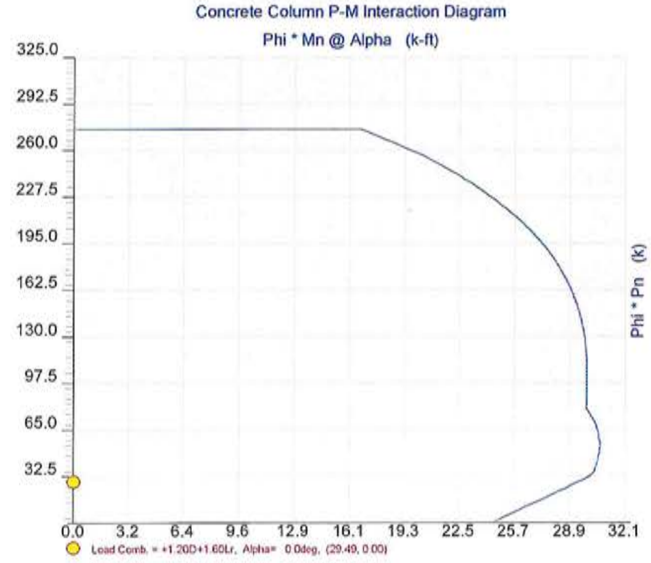


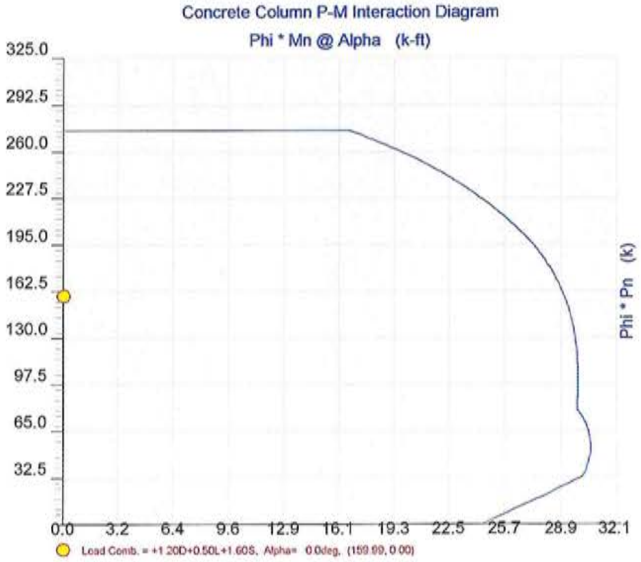
Concrete Column

Description : HSS SUPPORT - INTEGRAL w/ WALL









Concrete Column

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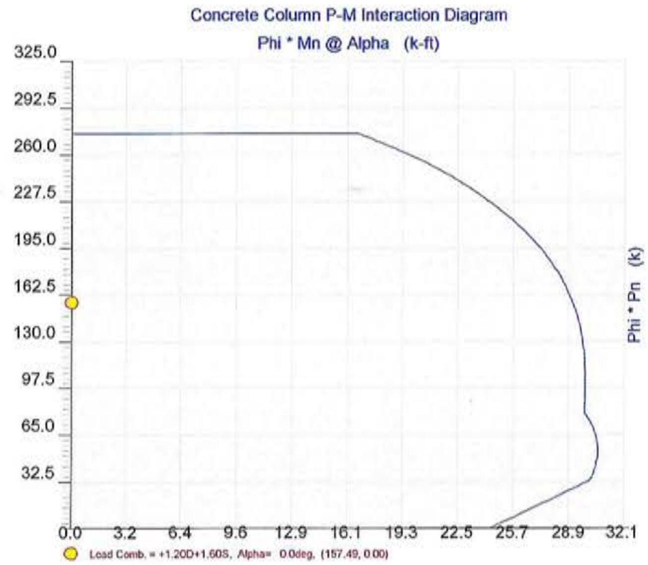

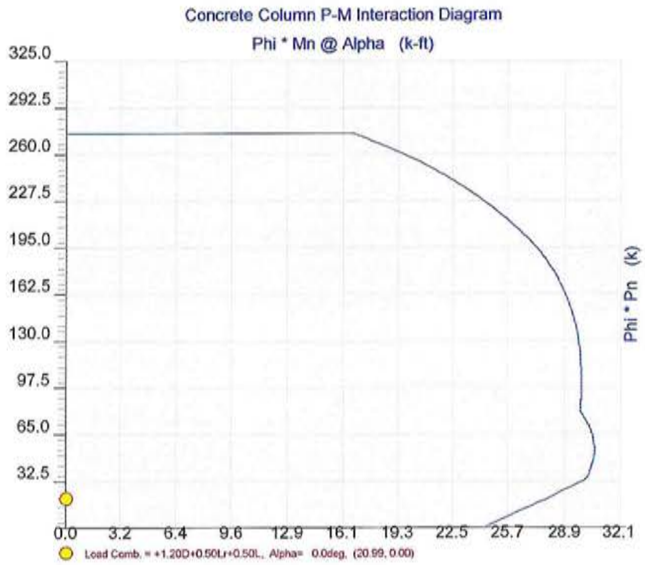

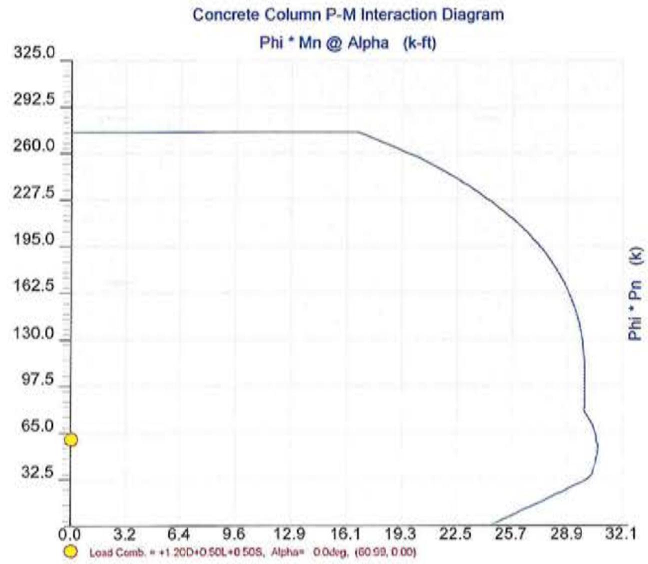

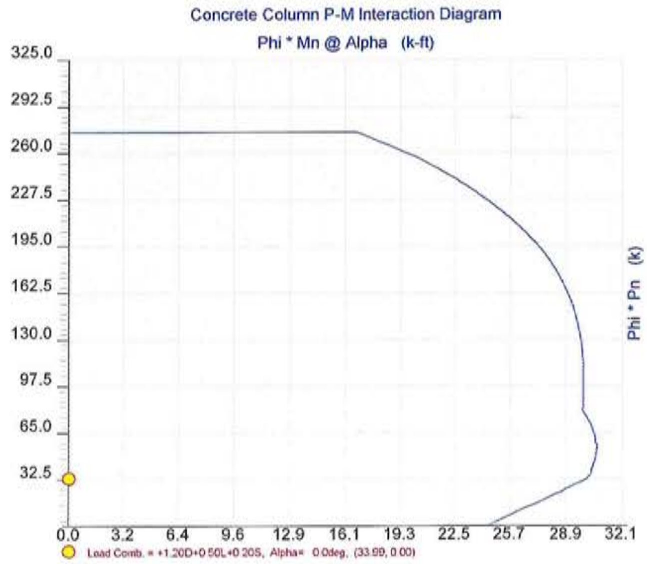

287/END

Concrete Column

Description : HSS SUPPORT - INTEGRAL w/ WALL