

## STRUCTURAL ENGINEERING CALCULATIONS

CLIENT: <u>HUGHES UMBANHOWAR ARCHITECTS</u>

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: <u>JULY 21, 2017</u>

ISSUE: BLDG. DEPT. SUBMITTAL #1



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



## 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	В		WE	IGHTS
DECKING				3.0 psf
FLOOR FRAMING				4.0 psf
79			-	psf
_			-	psf
4			*	psf
+			-	psf
t .			-	psf
H.			-	psf
7			7	psf
다. Market was a series of the control of the contr			+	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 LIV	E LOA Lo	20	_	60.0 psf
PITCH ADJUSTED DEAD LOAD	D'	=		7.0 psf

2nd FLOOR	C	Т	WEIGHTS	
HARDWOOD			4.0	psf
1-1/8" PLY			3.4	psf
MISC. MECH 3 WARMS	0220			psf
FLOOR FRAMING	7 4-			psf
5/8" PLY				psf
-			-	psf
8			2	psf
				psf
			2	psf
			8	psf
ADDITIONAL LOAD			0.0	
DEAD	D	88	14.3	
	2270		Date of the latest the	
LIVE RESIDENCE	Lo	=	40.0	psf
FRAMING PITCH			0.00	: 12
			1.00	
ADJUSTED LIVE RESIDENCE	Lo	=	40.0	psf
DITOU AD HISTED DEAD LOAD	DI	-	440	
PITCH ADJUSTED DEAD LOAD	D'	=	14.3	psi

DRIVEWAY LOADS	D		WEIGHTS	
6" CONCRETE	- Camara		75.0 psf	
1-1/8" PLY			3.4 psf	
FLOOR FRAMING			4.0 psf	
SIDING			3.0 psf	
		19	- psf	
		- 3	- psf	
-		9	- psf	
-		- 2	- psf	
		- 9	- psf	
-		9	- psf	
ADDITIONAL LOAD		416	0.0 psf	
DEAD	D	= =	85.4 psf	
LIVE STAIRS AND EXIT	L	= =	100.0 psf	
FRAMING PITCH			0.00 : 12	N.
			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
. 0			- psf
Pg = 264PSF			- psf
12-204121			- psf
			- psf
· 1-1=07(7(-4)			- psf
M-2.2(00+2)			- psf
· / W//.			- psf
H=0.2(264) SEISMIL W/SNOW			- psf
			- psf
ADDITIONAL LOAD			52.8 psf
	14	=	52.8 psf
*	*	=	0.0 psf
FRAMING PITCH			0.00 : 12
A ASSESSMENT OF STATE			1.00
ADJUSTED -	-	=	0.0 psf
PITCH ADJUSTED DEAD LOAD	D'	=	52.8 psf

GARAGE LOADS	F	$\top$	WEIGHTS
4" CONCRETE	7/4		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
Service Moderates			- 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



## **VERTICAL LOADS - WALLS:**

LOCATION:

5754 DAYBREAK RIDGE, EDEN, UTAH

101	WEI	GHTS
		3.0 psf
		2.0 psf
		1.5 psf
		2.1 psf
		1.0 psf
		1.0 psf
		1.5 psf
		3.1 psf
	(2)	psf
		psf
		psf
- 1		15.2 psf

	W	WEIG	HTS
			psf
		2	psf
(4)			psf
		2	psf
			psf
		- 2	psf
			psf
		-	psf
			psf
		2	psf
		-	psf
	-	=	0.0 psf

Х	WE	IGHTS
	-	psf
	2	psf
	7	psf
	-	psf
	-	psf
	_	psf
	7	psf
	-	psf
		0.0 psf

Y WEIGHTS
- psf
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Po.
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- ∞ 0.0 psf
<u>₹</u>

X	Z WEIGHTS
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	- ps
	- ps - ps - ps - ps
	- ps
	- ps
	- ps
	- DS
	- = 0.0 ps



## **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

· KIAILS

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

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						Panel S	pan (ft)		V		
Thickness (in)	Deflection Limit <sup>2</sup>	44	8	10	12	14	16	18	20	22	24
	L/360	100	43	29	21	16	10				
3.5	L/240	143	60	42	33	25	16				
	L/180	143*	61	57	46	34	22				
	L/360	105	52	39	30	24	18	15	11		
5.5	L/240	162	78	58	36	32	28	22	16		
	L/180	191*	80*	60*	46*	40	34	29	21		
	L/360	120	61	60	42	34	26	21	15	13	11
7.25	L/240	179*	85	75	61	50	39	31	23	21	18
M. Harris	L/180	179*	85	75	69	60	50	42	31	28	24
	L/360	131	80	66	52	43	33	28	22	20	18
9.25	L/240	168*	86	71	57°	51°	46	42*	34	30	26
	L/180	168*	86	71	57 <sup>*</sup>	51	46	42	39	37	34*
-	L/360	132	94	76	51	50	48	38	28	24	20
11.25	L/240	163*	94	76	59	55	51	45	39	36	31
	L/180	163*	94	76	59	55	51	45	39	36	33

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

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

the panel.

<sup>2</sup> Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted

building code.

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 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.

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.

For wall panel capacities utilizing a zero bearing configuration (Figure 2), the allowable load shall be determined using C<sub>v</sub>=0.86.





PROJECT	VILLAGE NEST		JOB	2017	-0610
CLIENT					
ADDRESS	EDEN, UT			1 77	
DESIGN	HUUM				
ENGINEER	J.H.		DATE	6/14/2	2017
	RICHMOND PE 44628 HOP	FMAYER SE 3495	PHEET	7	OF

SHOW LOAPS:
ETEN, UT CASIO ELEV. 8040-E
EXPOSURE C Pg = 264 PSF PEN WEBER COUNTY
ROOF PITCH X 1:12 -> CONSIDER FLAT
Ps = 0.7 Ce Ca Is Pg ASUE 7-10 (7.3-1)
Ce= 1.0 PARTIALLY EXPOSED
CE= 1.2 UPHENTED (UNUSED VACATION HOME)
I,=1.0 RISK CATEGORY II
Py = 0.7(1.0) 1.2(1.0) 2604 = 221.8 (SF -> 214 PSF -> USE )
5= 24485

OVERHANG: ASCE 7-10 7.4.6 Son = 28f = 2(221,8) = 443.6 858

# RICHMOND HOFFMAYER 16.



PROJECT	VILLAGE NEST	JOB	2017-0610
CLIENT	=		
<b>ADDRESS</b>	EDEN, UT		
DESIGN	HUUM		- 200
ENGINEER	J.H.	DATE	6/14/2017
	PICAGONO DE 40476 MOSENAVED SE 9095		//

	LOAD	COMBINATIONS
1	-	

1.20 + 1.6(Lr on 5)+L 1.20+1.0W+L+0.5(Lr on 5) 1.2P + LOE + L + 0,28 0.90 + 1.0 W 0,9D+1.0E

# EFFECTIVE SEISMIL WEIGHT:

ASLE 7-10 12.7.2

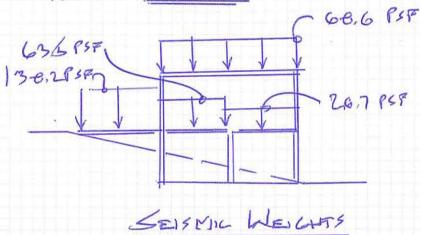
PS EXCEEDS 30 PSF : 20% OF UNIFORM DESIGN

WROOF = 14,8 + 0,2(264) = 68.615F

WFran = 26,7 857

WGARKE = 635 PSF

WORKE = 85,4 + 0.2(264) = 138,285F



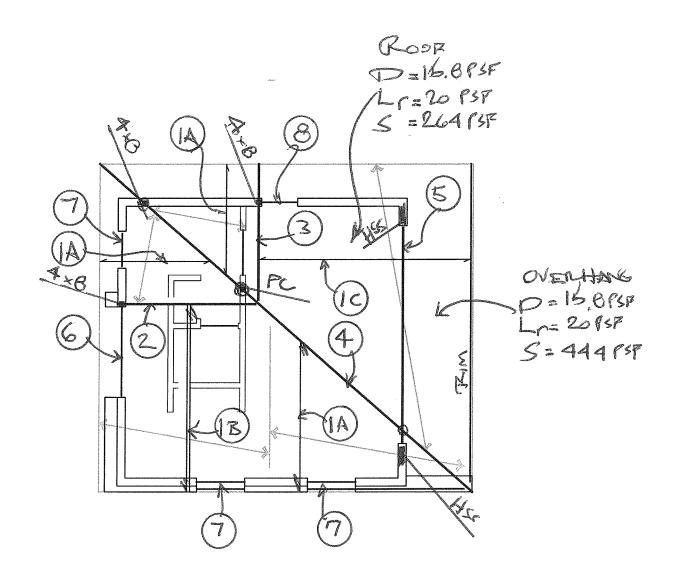
SEISMIL WEIGHTS





 PROJECT
 FELDSPAR LOWER
 JOB
 2017-0610

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 VILLAGE NEST
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ROF FRAMING TLAN
FELDIPAL - LOVER
N.T.S.

O: LERW 4



VILLAGE NEST

1,555.0 psi

MULTI UNIT PROJECT

Project ID: 2017-0610

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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**

E: Modulus of Elasticity Analysis Method: Allowable Stress Design 2,600.0 psi Fb - Tension Fb - Compr 1,900.0ksi Load Combination :ASCE 7-10 2,600.0 psi Ebend-xx 965.71 ksi Fc - Prll 2,510.0 psi Eminbend - xx 750.0 psi Fc - Perp Wood Species : Trus Joist 285.0 psi F۷ : MicroLam LVL 1.9 E Wood Grade

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

Density 42.0 pcf Repetitive Member Stress Increase

D(0.021014) Lr(0.0266) S(0.35112)

Span = 14.0 ft

## **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 Section used for this span	=	0.774 1 1.75x11.875	Maximum Shear Stress Ratio Section used for this span	=	0.494 : 1 1.75x11.875
fb : Actual	=	2,660.07psi	fv : Actual	=	161.95 psi
FB : Allowable	i = i	3,438.50psi	Fv : Allowable	=	327.75 psi
Load Combination Location of maximum on span Span # where maximum occurs	=	+D+S 7.000ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	=	+D+S 13.029 ft Span # 1
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection		0.658 in Ration   0.000 in Ration   0.697 in Ration   0.000 in Rat	0 = 0 < 240.0 0 = 240 > 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	



VILLAGE NEST JJH

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

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

Density 42.0 pcf Repetitive Member Stress Increase

D(0.021014) Lr(0.0266) S(0.35112)

2-1.75x11.87

Span = 15.0 ft

#### **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				4	Design OK
Maximum Bending Stress Ratio Section used for this span	=	0.444 1 2-1.75x11.87	Maximum Shear Stress Ratio Section used for this span	=	0.267 : 1 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 Location of maximum on span Span # where maximum occurs	=	+D+S 7.500ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	= =	+D+S 14.015 ft Span # 1
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection		0.434 in Rat 0.000 in Rat 0.459 in Rat 0.000 in Rat	io = 0<240.0 io = 391>=240.		

	Suppo	ort notation : Far left is #1	Values in KIPS	
Support 1	Support 2			
2.791	2.791			
0.095	0.095			
0.158	0.158			
0.357	0.357			
2.791	2.791			
0.307	0.307			
2.133	2.133			
0.095	0.095			
0.200	0.200			
2.633	2.633			
	2.791 0.095 0.158 0.357 2.791 0.307 2.133 0.095 0.200	Support 1 Support 2 2.791 2.791 0.095 0.095 0.158 0.158 0.357 0.357 2.791 2.791 0.307 0.307 2.133 2.133 0.095 0.095 0.200 0.200	2.791 2.791 0.095 0.095 0.158 0.158 0.357 0.357 2.791 2.791 0.307 0.307 2.133 2.133 0.095 0.095 0.200 0.200	Support 1 Support 2 2.791 2.791 0.095 0.095 0.158 0.158 0.357 0.357 2.791 2.791 0.307 0.307 2.133 2.133 0.095 0.095 0.200 0.200



VILLAGE NEST

MULTI UNIT PROJECT

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**Wood Beam** 

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Description: B1C - RAFTERS w OVERHANG

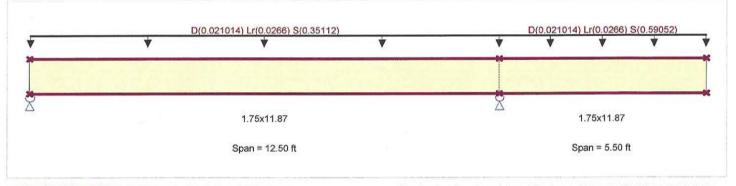
#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

Analysis Method : Allowable Stress Design	Fb - Tension	2,600.0 psi	E : Modulus of Elasti	city
Load Combination ASCE 7-10	Fb - Compr Fc - Prll	2,600.0 psi 2,510.0 psi	Ebend- xx Eminbend - xx	1,900.0ksi 965.71ksi
Wood Species : Trus Joist Wood Grade : MicroLam LVL 1.9 E	Fc - Perp Fv	750.0 psi 285.0 psi		1940 IDC 1944
	Ft	1,555.0 psi	Density	42.0pcf
Beam Bracing : Beam is Fully Braced against lateral-torsional b	uckling	1000	Repetitive Member	er Stress Increase



#### **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

Max Upward Total Deflection

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 Section used for this span	=	0.785 1 1.75x11.87	Maximum Shear Stress Ratio Section used for this span	=	0.608 : 1 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 Location of maximum on span	=	+D+S 12.500ft	Load Combination Location of maximum on span	=	+D+S 12.500 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection Max Downward Transient Deflection	tion	0.428 in Ratio	o = 308>=240.		14.18
Max Upward Transient Deflection	n	-0.044 in Ratio	>= 2976>=240.		
Max Downward Total Deflection		0.429 in Ratio	306>-240		

3046>=240.

-0.043 in Ratio =

Vertical Reactions			oport notation : Far left is #1	Values in KIPS	
Load Combination	Support 1	Support 2	Support 3		
Overall MAXimum	1.586	6.429	10000		
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			



VILLAGE NEST JJH

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**Wood Beam** 

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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			



VILLAGE NEST

2,025.0 psi

MULTI UNIT PROJECT

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45.050pcf

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Lic. #: KW-06002886

Description: B2 - ROOF BEAM

#### **CODE REFERENCES**

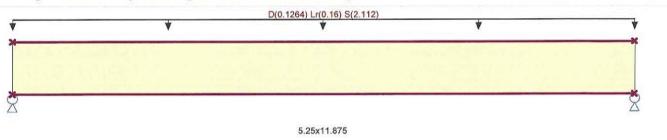
Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

Analysis Method: Allowable Stress Design 2,900.0 psi E: Modulus of Elasticity Fb - Tension 2,900.0 psi 2,000.0ksi Ebend-xx Load Combination :ASCE 7-10 Fb - Compr 1,016.54ksi Fc - Prll 2,900.0 psi Eminbend - xx Fc - Perp 625.0 psi : Trus Joist Wood Species 290.0 psi F۷ : Parallam PSL 2.0E Wood Grade

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



Span = 11.0 ft

## **Applied Loads**

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

Density

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 Section used for this span fb : Actual FB : Allowable	= = =	0.996 1 5.25x11.875 3,321.29psi 3,335.00psi	Maximum Shear Stress Ratio Section used for this span fv : Actual Fv : Allowable	=	0.739 : 1 5.25x11.875 246.45 psi 333.50 psi
Load Combination Location of maximum on span Span # where maximum occurs	=	+D+S 5.500ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	=	+D+S 10.036 ft Span # 1
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	in	0.478 in Ratio 0.000 in Ratio 0.511 in Ratio 0.000 in Ratio	= 0<240.0 = 258>=240.		

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		8	
Lr Only	0.880	0.880			
S Only	11.616	11.616			



VILLAGE NEST

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

11

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**Wood Beam** 

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Lic. #: KW-06002886

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**

E: Modulus of Elasticity Analysis Method: Allowable Stress Design 2,900.0 psi Fb - Tension 2,000.0ksi Load Combination :ASCE 7-10 Fb - Compr 2,900.0 psi Ebend-xx 2,900.0 psi Eminbend - xx 1,016.54ksi Fc - Prll 625.0 psi Fc - Perp Wood Species : Trus Joist 290.0 psi : Parallam PSL 2.0E Fv Wood Grade 2,025.0 psi 45.050pcf Ft Density 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 Section used for this span fb : Actual	=	0.521: 1 5.25x11.875	Maximum Shear Stress Ratio Section used for this span fv: Actual	=	0.469 : 1 5.25x11.875 156.50 psi
FB : Allowable	=	1,737.41psi 3,335.00psi	Fv : Allowable	=	333.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span Span # where maximum occurs	=	4.250ft Span # 1	Location of maximum on span Span # where maximum occurs	=	0.000 ft Span # 1
Maximum Deflection	ravan				200 minutes 19
Max Downward Transient Deflect Max Upward Transient Deflection		0.149 in Rat		4	
Max Downward Total Deflection		0.000 in Rat 0.159 in Rat	io = 639>=240.		
Max Upward Total Deflection		0.000 in Rat	io = 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	



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

12

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**Wood Beam** 

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Lic. # : KW-06002886

Description: B4 - RIDGE

#### **CODE REFERENCES**

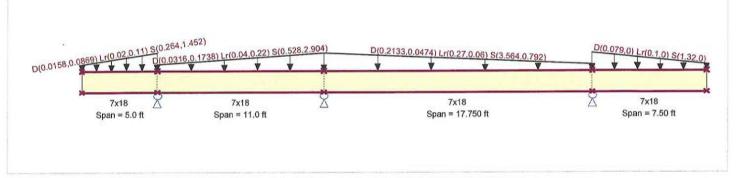
Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

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

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



## **Applied Loads**

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

V-I--- I- KIDO

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 Section used for this span	=	0.643 1 M 7x18	aximum Shear Stress Ratio Section used for this span	=	0.822 : 1 7x18
fb : Actual	=	2,049.11psi	fv : Actual	=	274.07 psi
FB : Allowable	=	3,188.23psi	Fv : Allowable	=	333.50 psi
Load Combination Location of maximum on span Span # where maximum occurs	=	+D+S 11.000ft Span # 2	Load Combination Location of maximum on span Span # where maximum occurs	= =	+D+S 11.000ft Span # 2
Maximum Deflection Max Downward Transient Deflect Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection		0.357 in Ratio = -0.412 in Ratio = 0.382 in Ratio = -0.433 in Ratio =	436 >= 360 558 >= 240.		

Vertical Reactions		Support notation : Far left is #1					
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			



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

13

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**Wood Beam** 

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Lic. # : KW-06002886 Description: B4 - RIDGE

/ertical Reactions		Sup	port notation	: Far left is #1		Values in KIPS	
oad Combination	Support 1 St	upport 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			
O+Lr, LL Comb Run (LLLL)		1.316	6.328	3.037			
+S		7.627	45.573	19.198			
0+0.750Lr, LL Comb Run (***L)		0.819	3.040	2.045			
TO A REPORT OF THE PROPERTY OF		0.514	4.904	2.401			
D+0.750Lr, LL Comb Run (**L*) D+0.750Lr, LL Comb Run (**LL)		0.534	4.833	2.734			
			3.858	1.679			
0+0.750Lr, LL Comb Run (*L**)		1.158	3.786				
0+0.750Lr, LL Comb Run (*L*L)		1.178		2.012			
0+0.750Lr, LL Comb Run (*LL*)	MINNESS	0.873	5.651	2.368			
0+0.750Lr, LL Comb Run (*LLL)	ONG	0.893	5.580	2.701			
0+0.750Lr, LL Comb Run (L***)	almices ong cuk	1.094	3.055	1.717			
0+0.750Lr, LL Comb Run (L**L)	CK	1.113	2.984	2.050			
740.730LI, LE COMB Run (L. L.)		0.809	4.848	2.406			
0+0.750Lr, LL Comb Run (L*LL)		0.828	4.777	2.739			
0+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			
0+0.750Lr, LL Comb Run (LLL*)		1.167	5.595	2.373			
0+0.750Lr, LL Comb Run (LLLL)		1.187	5.524	2.706			
0+0.750S		5.920	34.958	14.826			
0.60D		0.479	1.867	1.027			
Only, LL Comb Run (***L)		0.026	-0.095	0.444			
Only, LL Comb Run (**L*)		-0.380	2.391	0.918			
Only, LL Comb Run (**LL)		-0.354	2.295	1.362			
Only, LL Comb Run (*L**)		0.478	0.996	-0.044			
Only, LL Comb Run (*L*L)		0.505	0.900	0.400			
Only, LL Comb Run (*LL*)		0.098	3.387	0.874			
Only, LL Comb Run (*LLL)		0.125	3.291	1.318			
Only, LL Comb Run (L***)		0.393	-0.074	0.007			
Only, LL Comb Run (L**L)		0.419	-0.170	0.451			
Only, LL Comb Run (L*L*)		0.012	2.316	0.925			
Only, LL Comb Run (L*LL)		0.039	2.221	1.369			
Only, LL Comb Run (LL**)		0.871	0.921	-0.038			
Only, LL Comb Run (LL*L)		0.897	0.826	0.407			
Only, LL Comb Run (LLL*)		0.491	3.312	0.881			
Only, LL Comb Run (LLLL)		0.517	3.217	1.325			
Only		6.828	42.462	17.485			



VILLAGE NEST

MULTI UNIT PROJECT

Project ID: 2017-0610

7-0610 14

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#### **Wood Beam**

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Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886

Description: B5 - BIG OPENING

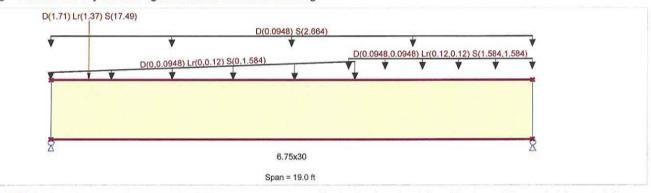
#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

Analysis Method : Allowable Stress Design		Fb - Tension	2,400.0 psi	E : Modulus of Elasti	city
Load Combinati	ion ASCE 7-10	Fb - Compr	1,850.0 psi	Ebend- xx	1,800.0ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0ksi
Wood Species	: DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0ksi
Wood Species Wood Grade	24F - V4	Fv	265.0 psi	Eminbend - yy	850.0ksi
Wood Grade	· And · Vi	Ft	1,100.0 psi	Density	31.20pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional b	uckling		20110119	Control of the Park



#### **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, (Runiform 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 SUMMARY					Design OK
Maximum Bending Stress Ratio	, =	CALL THE RESERVE THE PARTY OF T	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	=	16.504ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span #1
Maximum Deflection					
Max Downward Transient Deflect	tion	0.453 in Ratio	= 503>=360		
Max Upward Transient Deflection	1	0.000 in Ratio	= 0<360		
Max Downward Total Deflection		0.479 in Ratio	= 475>=240.		
Max Upward Total Deflection		0.000 in Ratio			

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	



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

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**Wood Beam** 

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Description: **B5 - BIG OPENING** 

Support notation : Far left is #1 **Vertical Reactions** 

Values in KIPS

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



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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Steel Beam

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Lic. #: KW-06002886

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

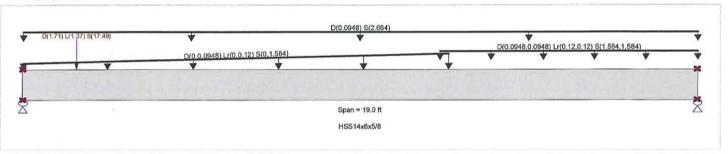
Beam Bracing : Bending Axis : Completely Unbraced
Major Axis Bending

Fy: Steel Yield:

46.0 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

 $\begin{aligned} & \text{Varying Uniform Load}: \ D(S,E) = 0.01580 -> 0.01580, \ Lr(S,E) = 0.020 -> 0.020, \ S(S,E) = 0.2640 -> 0.2640 \ ksf, \ Extent = 0.0 ->> 12.0 \ ft, \ Trib \ Width = 0.0 \ Varying Uniform Load: \ D(S,E) = 0.01580 -> 0.01580, \ Lr(S,E) = 0.020 -> 0.020, \ S(S,E) = 0.2640 -> 0.2640 \ ksf, \ Extent = 11.750 ->> 19.0 \ ft, \ Trib \ Width = 0.0 \ Varying Uniform Load: \ D(S,E) = 0.01580 -> 0.01580, \ Lr(S,E) = 0.020 -> 0.020, \ S(S,E) = 0.2640 -> 0.2640 \ ksf, \ Extent = 11.750 ->> 19.0 \ ft, \ Trib \ Width = 0.0 \ Varying Uniform Load: \ D(S,E) = 0.01580 -> 0.01580, \ Lr(S,E) = 0.020 -> 0.020, \ S(S,E) = 0.2640 -> 0.2640 \ ksf, \ Extent = 11.750 ->> 19.0 \ ft, \ Trib \ Width = 0.0 \ Varying Uniform Load: \ D(S,E) = 0.01580 -> 0.01580, \ Lr(S,E) = 0.020 -> 0.020, \ S(S,E) = 0.2640 -> 0.2640 \ ksf, \ Extent = 11.750 ->> 19.0 \ ft, \ Trib \ Width = 0.0 \ Varying Uniform Load: \ D(S,E) = 0.01580 -> 0.01580, \ Lr(S,E) = 0.020 -> 0.020, \ S(S,E) = 0.2640 -> 0.2640 \ ksf, \ Extent = 11.750 ->> 19.0 \ ft, \ Trib \ Width = 0.0 \ Varying Uniform Load: \ D(S,E) = 0.01580 -> 0.01580, \ Lr(S,E) = 0.020 -> 0.020, \ S(S,E) = 0.2640 -> 0.2640 \ ksf, \ Extent = 11.750 ->> 19.0 \ ft, \ Trib \ Width = 0.0 \ Lr(S,E) = 0.01580 -> 0.01580, \ Lr(S,E) = 0.020 -> 0.020, \ S(S,E) = 0.2640 -> 0.2640 \ ksf, \ Extent = 11.750 ->> 19.0 \ ft, \ Trib \ Width = 0.0 \ Lr(S,E) = 0.01580 -> 0.015$ 

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 OK **DESIGN SUMMARY** Maximum Bending Stress Ratio = Maximum Shear Stress Ratio = 0.224:1 0.981:1 Section used for this span Section used for this span HSS14x6x5/8 HSS14x6x5/8 Ma: Applied Va : Applied 52.772 k 199.816 k-ft Mn / Omega: Allowable Vn/Omega: Allowable 235.387 k 203.603 k-ft +D+S Load Combination +D+S Load Combination 0.000 ft 9.554ft Location of maximum on span Location of maximum on span 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</td>

 Max Downward Total Deflection
 0.951 in Ratio = 240 > =240.

 Max Upward Total Deflection
 0.000 in Ratio = 0 < 240.0</td>

Maximum Forces & Stresses for Load Combinations

Load Combina	ation		Max Stress	ax Stress Ratios Summary of Moment Values				Summary of Shear Values						
Segmen		Span #	M	٧	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only Dsgn, L = +D+Lr	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
The second secon	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
	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
Dsgn. L = +D+0.750S	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
Dsgn. L = +0.60D	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
	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	Max	imum	Def	lect	ions
---------	-----	------	-----	------	------

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	410	Max. "+" Defl	Location in Span
+D+S	1	0.9512	9.500			0.0000	0.000
A CONTRACTOR OF THE PROPERTY OF THE PROPERTY OF						Values in KIDO	

+D+8	10	0.9512	9.500	0.0000	0.000
<b>Vertical Reactions</b>			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			



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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**Steel Beam** 

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Lic. #: KW-06002886 Description: B5-BIG 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			



VILLAGE NEST

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 20 JUN 2017, 8:51AM

**Wood Beam** 

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Lic. #: KW-06002886 **B6-HEADER** Description:

## **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 Elastic	ity
Load Combination ASCE 7-10	Fb - Compr	2600 psi	Ebend- xx	1900ksi
	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		
Wood Glade , Milerozam EVE 110 E	Ft	1555 psi	Density	42 pcf



Span = 8.0 ft

## **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 Section used for this span fb : Actual FB : Allowable	=	0.895 1 2-1.75x7.25 2,628.13psi	Maximum Shear Stress Ratio Section used for this span fv : Actual Fv : Allowable	=	0.517 : 1 2-1.75x7.25 169.50 psi
2 372 500 American	=	2,936.44psi	2 2366 73 70 70 70 70 70 70 70 70 70 70 70 70 70	=	327.75 psi
Load Combination Location of maximum on span Span # where maximum occurs	=	+D+S 4.000ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	=	+D+S 7.416 ft Span # 1
Maximum Deflection Max Downward Transient Deflection	tion	0.348 in Rati	io = 276>=240.		
Max Upward Transient Deflection	n	0.000 in Rat	io = 0<240.0		
Max Downward Total Deflection Max Upward Total Deflection		0.368 in Rati 0.000 in Rati	io = 260>=240.		

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			



VILLAGE NEST JJH

Project ID: 2017-0610 **MULTI UNIT PROJECT** 

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**Wood Beam** 

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**B7 - BEARING HEADERS** Description:

#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

Wood Grade

Analysis Method: Allowable Stress Design Load Combination :ASCE 7-10 : Trus Joist Wood Species

: MicroLam LVL 1.9 E

Beam Bracing : Completely Unbraced

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

1,900.0ksi Ebend-xx Eminbend - xx

E: Modulus of Elasticity

Density

965.71 ksi

42.0pcf



Span = 4.50 ft

## **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 Section used for this span	=	0.749 1 2-1.75x7.25	Maximum Shear Stress Ratio Section used for this span	=	0.670 : 1 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 Location of maximum on span	=	+D+S 2.250ft	Load Combination Location of maximum on span	=	+D+S 3.909ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection Max Downward Transient Deflect	ion	0.093 in Ratio	= 581>=240.		

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

	 cal	_			
•		20	120	-	ne

Vertical Reactions		Support notation : Far left is #1	Values in KIPS
Load Combination	Support 1 Su	pport 2	

Load Combination	oupport i	Support z.	
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	



VILLAGE NEST

MULTI UNIT PROJECT

Project ID: 2017-0610

18

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## **Wood Beam**

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Description: B8 - BEARING HEADERS

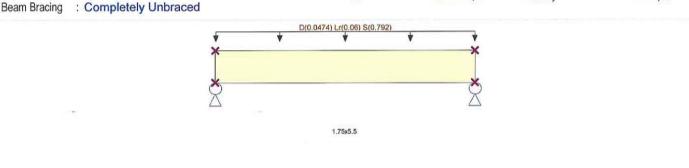
#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

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



## Span = 3.50 ft

#### **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)

Manifestore Danding Otrace Datie		0.000.4	Maximum Shear Stress Ratio	=	0.520 : 1
Maximum Bending Stress Ratio	=			.=	
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	i = i	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 Deflect	ion	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 =			

	Suppor	t notation : Far lett is #1	Values in KIPS	
Support 1	Support 2			
1.469	1.469			
0.050	0.050			
0.083	0.083			
0.188	0.188			
1.469	1.469			
0.162	0.162			
1.122	1.122			
0.050	0.050			
0.105	0.105			
1.386	1.386			
	1.469 0.050 0.083 0.188 1.469 0.162 1.122 0.050 0.105	Support 1 Support 2  1.469	1.469	Support 1 Support 2  1.469





PROJECT	VILLAGE NEST		JOB	2017-	0610	10
CLIENT	+					
<b>ADDRESS</b>	EDEN, UT					
DESIGN	HUUM					
ENGINEER	J.H.		DATE	6/14/2	017	
	RICHMOND PF 49628	HOFFMAYER SE 3495	CHIETE	2000	OF	

WIND ON EXTERIOR WALLS:
115 MPH - EXP"C" ASCE 7-10
WIMN = 16 PSF
9 = 0,00256 Kz Kz Kz Kz V2
Kz= 0.9 = 20 TABLE 30.3-1
KX= 0.65
9=0.00266(0.9X1X0.85>115 = 25.985F
P=26,9(64-642)
Asono= 10/x 1,23'= 13,362 -> 6/p= 1.0
GCP2 = 0.16
P = 25,9(1,++0,1+) = 51,2-0 PSP > 16 PSP
ALL WALL CLAPPING MUST RESIST 51,3151



**VILLAGE NEST** JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

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1.076 k 0.0 k

Tension

**Wood Column** 

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Lic. #: KW-06002886 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 End Fixities Overall Column F	Top & Bo	e Stress Des ottom Pinned	-	Wo	ood Section Name ood Grading/Manuf. ood Member Type		d Lumber	
Wood Species Wood Grade Fb + Fb - Fc - Prll Fc - Perp	Spruce - Pir No.1 850.0 psi 850.0 psi 700.0 psi 425.0 psi	re - Fir Fv Ft Density	125.0 psi 550.0 psi 26.210 pcf	Ex	act Width act Depth Area Ix Iy	3.50 in 7.250 in 25.375 in <sup>2</sup> 111.148 in <sup>4</sup> 25.904 in <sup>4</sup>	Cf or Cv for Tension	1.30 1.050 1.20 1.0 1.0
E: Modulus of El	asticity	x-x Bending	y-y Bending	Axial			Kf : Built-up columns	1.0 NDS 15.3.2
	Basic Minimum	1,300.0 470.0	1,300.0 470.0	1,300.0 ksi		effection (bucklin	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	185	Shear	Check	Results	
			m 15	01 0	2

PASS Max. Axial+Bending Stress Ratio = Load Combination Governing NDS Forumla	0.9256 : 1 +D+S Comp Only, fc/Fc'	Maximum SERV Top along Y-Y Top along X-X	/ICE Lateral Load 1.076 k 0.0 k	Reactions Bottom alon Bottom alon	ng Y-Y 1	1
Location of max.above base	0.0 ft	Maximum SERVICE	Load Lateral Deflec	tions		
At maximum location values are Applied Axial	16.549k	Along Y-Y for load co	0.3922 in atombination: W Only	5.285	ft above base	
Applied Mx Applied My Fc : Allowable	0.0 k-ft 0.0 k-ft 704.56 psi	Along X-X for load co	0.0 in at ombination : n/a	t 0.0	ft above base	
	701.00	Other Factors used	to calculate allowab	le stresses		
PASS Maximum Shear Stress Ratio = Load Combination	0.1272:1 +D+0.60W			Bending	Compression	
Location of max.above base	0.0 ft					

38.172 psi

200.0 psi

## **Load Combination Results**

Allowable Shear

Applied Design Shear

	22	429	Maximum Axial	+ Bending	Stress Ratios	Maximu	Maximum Shear Ratios			
Load Combination	CD	CP	Stress Ratio	Status	Location	Stress Ratio	Status	Location		
D Only	0.900	0.877	0.1052	PASS	0.0ft	0.0	PASS	10.50 ft		
+D+Lr	1.250	0.815	0.1604	PASS	O.Oft	0.0	PASS	10.50 ft		
+D+S	1.150	0.834	0.9256	PASS	O.Oft	0.0	PASS	10.50 ft		
+D+0.750Lr	1.250	0.815	0.1407	PASS	O.Oft	0.0	PASS	10.50 ft		
+D+0.750S	1.150	0.834	0.7159	PASS	O.Oft	0.0	PASS	10.50 ft		
+D+0.60W	1.600	0.750	0.3970	PASS	5.285ft	0.1272	PASS	0.0 ft		
+D+0.750Lr+0.450W	1.600	0.750	0.3184	PASS	5.285ft	0.09543	PASS	0.0 ft		
+D+0.750S+0.450W	1.600	0.750	0.7653	PASS	5.215ft	0.09543	PASS	0.0 ft		
+0.60D+0.60W	1.600	0.750	0.3869	PASS	5.285ft	0.1272	PASS	0.0 ft		



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

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**Wood Column** 

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Description:

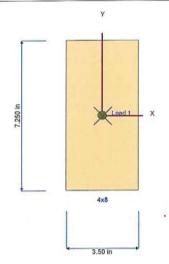
ROOF SUPPORT POSTS - EXTERIOR WALLS

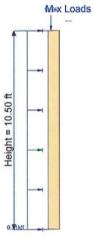
Load	Com	bination	Results
------	-----	----------	---------

				Maximum .	Axial +	- Bending	Stress Ratios		Maxim	um Sh	ear Ratio	OS
Load Combination	CD	CP		Stress R	atio	Status	Location	Stre	ess Ratio	Sta	atus L	ocation
+0.60D	1.600	0.750		0.041	53	PASS	0.0 ft		0.0	PA	ASS	10.50 ft
Maximum Reactions								Note: C	only non-	zero r	eactions	are listed
	X-X Axis R	teaction	k	Y-Y Axis	Reaction	n Ax	ial Reaction	My - End M	oments	k-ft	Mx - En	d Moments
Load Combination	@ Base	@ Тор		@ Base	@ To	p	@ Base	@ Base	@ Top		@ Base	@ Top
D Only	MIC. MAN.	2000 1000					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.



Project Title: Engineer:

Project Descr:

VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

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0.0 k

**Wood Column** 

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

## Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combinations Used: ASCE 7-10

#### **General Information**

Analysis Method : End Fixities		e Stress Des ottom Pinned	-	W	ood Section Name ood Grading/Manuf.	2-2x8 Gradeo	d Lumber	
Overall Column H			10.50 ft	W	ood Member Type	Sawn		
( Used for	non-slender cald			Ex	act Width	3.0 in	Allow Stress Modification Factor	S
Wood Species	Spruce - Pir	ne - Fir		Ex	act Depth	7.250 in	Cf or Cv for Bending	1.20
Wood Grade	No.1				Area	21.750 in^2	Cf or Cv for Compression	1.050
Fb+	850.0 psi	Fv	125.0 psi		lx	95.270 in^4	O( - O ( - T	1.20
Fb -	850.0 psi	Ft	550.0 psi		ly	16.313 in^4	and the second second	1.0
Fc - Prll	700.0 psi	Density	26.210 pc		3	10.010	Ct: Temperature Factor	1.0
Fc - Perp	425.0 psi						Cfu : Flat Use Factor	1.0
E: Modulus of Ela	asticity	x-x Bending	y-y Bending	Axial			Kf : Built-up columns	1.0 NDS 15.3.2
	Basic	1,300.0	1,300.0	1,300.0 ks	i		Use Cr : Repetitive ?	No
	Minimum	470.0	470.0	Bra	ace condition for def	flection (bucklir		

Fully braced against buckling along X-X Axis X-X (width) axis:

Unbraced Length for X-X Axis buckling = 10.50 ft, K = 1.0 Y-Y (depth) axis:

**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 = Load Combination	0.6013:1 +D+S
Governing NDS Forumla	Comp Only, fc/Fc'
Location of max.above base	0.0 ft
At maximum location values are	
Applied Axial	8.976k
Applied Mx	0.0 k-fi
Applied My	0.0 k-fi
Fc : Allowable	686.32 psi

0.07386:1 PASS Maximum Shear Stress Ratio = 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 . .

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

Maximum SERVICE Load Lateral Deflections . . .

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

for load combination: n/a

Other Factors used to calculate allowable stresses . . .

Tension Bending Compression

#### **Load Combination Results**

	122		Maximum Axial	+ Bending	Stress Ratios	Maximum Shear Ratios			
Load Combination	CD	CP	Stress Ratio	Status	Location	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	O.Oft	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	0.0ft	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 f	
+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	



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

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## **Wood Column**

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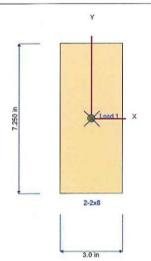
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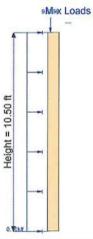
2ND FLOOR BEARING STUDS @ 48" o.c. - SIP SPLINES

#### **Load Combination Results**

	2		Max	mum Ax	ial + Bend	ding Stress Rat	tios		Maxim	um Sh	ear Ratio	S
Load Combination	CD	CP	Sti	ess Rat	io Sta	tus Locatio	n	Stre	ss Ratio	Sta	itus L	ocation
+0.60D	1.600	0.719	(	0.01779	9 PAS	S 0	.Oft		0.0	PA	SS	10.50 ft
Maximum Reactions								Note: O	nly non-	zero re	eactions	are listed
21 W ==: (A.13 755	X-X Axis Reaction k		X-X Axis Reaction k Y-Y Axis Reaction Axial Reaction		kis Reaction k Y-Y Axis Reaction Axial Reaction			My - End Mo	ments	k-ft	Mx - En	d Moments
Load Combination	@ Base	@ Тор	@	Base (	@ Тор	@ 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			1	0.321	0.321	0.327						
+0.60D						0.327						
Lr Only						0.639						
S Only						8.430						
W Only			1	0.536	0.535							

oad 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	





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



VILLAGE NEST

JJH **MULTI UNIT PROJECT**  Project ID: 2017-0610

Printed: 20 JUN 2017, 9:24AM

0.0ft above base

Steel Column

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

Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886

RIDGE SUPPORT Description:

#### 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 E: Elastic Bending Modulus

36.0 ksi 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**

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

## Maximum SERVICE Load Reactions

midili ofice ford itedonolis	
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 . . .

0.0 in at 0.0ft above base Along Y-Y

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

for load combination:

0.0 in at Along X-X

for load combination:

#### **Load Combination Results**

	Maximum Axial +	Bending S	tress Ratios	Maximu	atios .	
Load Combination	Stress Ratio 3		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	@ Тор		@ Base	@ Top	@ Base	@ Top		@ Base	@ Top
D Only	3.219					SAIR BOTTON					
+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										



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

Steel Column

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Lic. #: KW-06002886 Description:

RIDGE SUPPORT

Extreme Reactions	

	AND DESCRIPTION OF THE PROPERTY OF THE PROPERT	Axial Reaction	X-X Axis	Reaction	k	Y-Y Axis	Reaction	Mx - End M	oments k-ft	My - End	Moments
Item	Extreme Value	@ Base	@ Base	@ Тор		@ Base	@ Тор	@ Base	@ Тор	@ 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				.5					
•	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									
н	Minimum	3.219									
Moment, Y-Y Axis Ba	Maximum	3.219									
п	Minimum	3.219									
Moment, X-X Axis To	Maximum	3.219									
	Minimum	3.219									
Moment, Y-Y Axis To	Maximum	3.219									
K AND STATE OF STATE	Minimum	3.219									

## **Maximum Deflections for Load Combinations**

Load Combination	Max. X-X Defle	ction	Distance		Max. Y-Y Defi	ection	Distanc	e
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

Stool	Section	Properties	Pipe4 Std
oreer	Decuon	FIUDEILIES	LIDEA OIL

orgon populari								
Depth	=	4.500 in	l 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	l yy	=	6.820 in^4			
Weight = 10.858 plf	S yy	=	3.030 in^3					
			R yy	=	1.510 in			

Ycg

0.000 in



VILLAGE NEST JJH MULTI UNIT PROJECT

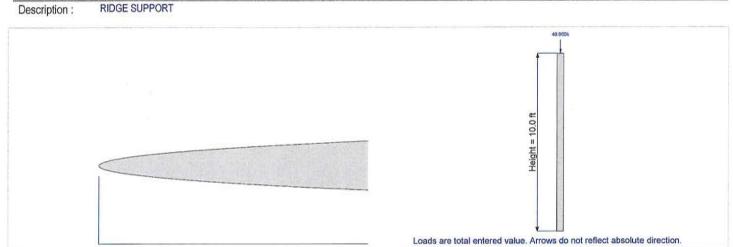
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Steel Column

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Lic. #: KW-06002886

RIDGE SUPPORT





VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

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#### Steel Column

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**B5 SUPPORT** Description:

#### 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: Analysis Method:

HSS12x6x5/8

Steel Stress Grade

Fy: Steel Yield E: Elastic Bending Modulus 29,000.0 ksi

Allowable Strength

46.0 ksi

Overall Column Height Top & Bottom Fixity

10.0 ft 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**

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

# **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 . . .

> 53.136 k Pa: Axial 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

> > Axial Reaction

PASS Maximum Shear Stress Ratio = Load Combination

At maximum location values are . . .

Maximum SERVICE Load Reactions . .

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

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

Maximum SERVICE Load Deflections . . .

0.0 in at Along Y-Y

above base O.Oft for load combination:

0.0ft above base

My - End Moments

@ Top

@ Base

Along X-X 0.0 in at

for load combination:

Location of max.above base

Va : Applied Vn / Omega : Allowable

0.0 k 0.0 k

X-X Axis Reaction

@ Top

@ Base

#### Load Combination Results

	Maximum Axial +	Bending S	tress 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	

Y-Y Axis Reaction

@ Top

@ Base

0.0:1

0.0 ft

#### **Maximum Reactions**

Note: Only non-zero reactions are listed.

Mx - End Moments

@ Top

@ Base

Load Combination	@ Base		
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		



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

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Steel Column

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Lic. # : KW-06002886

Description: **B5 SUPPORT** 

Extreme	Reactions
---------	-----------

		Axial Reaction	X-X Axis Reaction			Y-Y Axis	Reaction	Mx - End Moments		k-ft	My - End	Moments
Item	Extreme Value	@ Base	@ Base	@ Тор		@ Base	@ Тор	@ Base	@ Top		@ Base	@ Тор
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										
TOTAL STREET,	Minimum	4.026										

## **Maximum Deflections for Load Combinations**

Load Combination	Max. X-X Deflect	ction	Distance		Max. Y-Y Defi	ection	Distanc	е
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	l 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	l yy	=	107.000 in^4	С	=	71.100 in^3
Weight	=	67.618	plf	S yy	=	35.500 in^3			
				R yy	=	2.390 in			
7,200				Zy	=	42.100 in^3			

Ycg 0.000 in



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

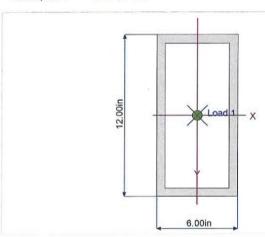
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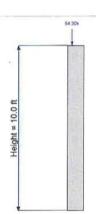
Steel Column

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Description: **B5 SUPPORT** 

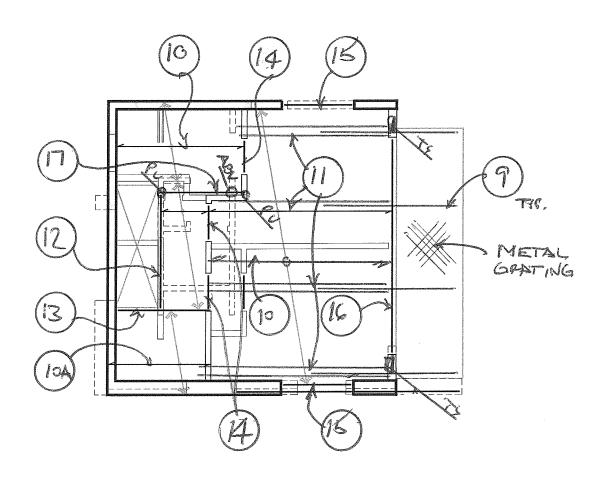




Loads are total entered value. Arrows do not reflect absolute direction.



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	S OF



PROFESSAL LOVER
N.T.S.

O: BEAM #

# RICHMOND HOFFMAYER INC. STRUCTURAL ENGINEERING



PROJECT	VILLAGE NEST	_ JOB	2017-0610	
CLIENT				
ADDRESS	EDEN, UT			
DESIGN	ниим			
ENGINEER	J.H.	DATE	6/14/2017	27/40
	ENGLOSON DE ANADO MOREO ANTO AN THOU	000	g 2 :	-

MEJAL GRATING:
D = 12-PSF L = 60 PSP S = 269PSF - (OPENED GRATING - POINT DBL SHOW)
LOAD COMSONASTIONS:
D+L = 72837
P+3 = 27683F & GOVERNS
P+0.76L+0.765 = 266858
SPAN M = W9 (2- SPAN OF)
32" 2944 W- W
46" 6624 11-11
64" 1177612-12
MAT. ALDMINUM STEEL
E = 10000 km 29000 km SAN IN MAES F = 12000 psi 16000 psi MIN (SAN IN MAES FSK>M & EIK > 5 US/4000 × 240
FSK>M & EIK > 5 WS/460 = x 240
C BARA/FE LZTROSF
K=12/(1/6)=274
12000 (S)274 > 2944 -> 5= 0.009 1, (AL
12000 (S)27,4 > 2944 -> 5 = 0.009 12 AL. 12000 (S)27,4 > 6614 -> 5 = 0.020 12 S 12000 (S)27,4 > 11776 -> 5 = 0.035 12 S

# RICHMOND HOFFMAYER INC. STRUCTURAL ENGINEERING



 PROJECT
 VILLAGE NEST
 JOB
 2017-0610

 CLIENT

 ADDRESS
 EDEN, UT

 DESIGN
 HUUM

 ENGINEER
 J.H.
 DATE
 6/14/2017

 $|\frac{2}{16}(0,000) = 0.00611^{3} | \frac{1}{16}(0.02) = 0.013311^{3} | \frac{1}{16}(0.036) = 0.023311^{3} | \frac{1}{16}(0.036) = 0.023311^{3} | \frac{1}{16}(0.036) = 0.023311^{3} | \frac{1}{16}(0.036) = 0.023311^{3} | \frac{1}{16}(0.036) = 0.000111^{4} | \frac{1}{16}(0.0109(32^{3})^{2})| \frac{1}{10} = 0.0001111^{4} | \frac{1}{16}(0.0109(64^{3})^{2})| \frac{1}{10} = 0.00031111^{4} | \frac{1}{16}(0.0003) = 0.00031311^{4} | \frac{1}{16}(0.0003) = 0.00031511^{4} | \frac{1}{16}(0.0003) = 0.0003111^{4} | \frac{1}{16}(0.0003) = 0.0003$ 

| DEED X 3/6" AL = S = 0.03/3/10"

HOSE UP TO 4-6" SPANK U/ BARUE MODILAND HILLS CA 91364

425 CANOGA AVENUE WOODLAND HILLS CA 91364 PHONE (818) 347-7008 FAX (818) 883-8869 Info@RichmondHoffmayer.com



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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Steel Beam

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**B9 - DECK SUPPORT** Description:

#### **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

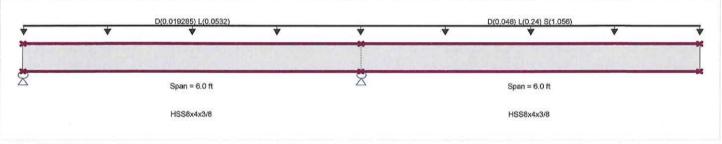
Beam Bracing: Beam is Fully Braced against lateral-torsional buckling

Major Axis Bending Bending Axis:

Fy: Steel Yield:

50.0 ksi

29,000.0 ksi E: Modulus:



## **Applied Loads**

Load Combination

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

Max. "+" Defl

-0.0430

Location in Span

3.504

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

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

Span

1

Max. "-" Defl

0.0000

Load Combin	ation		Max Stress	Ratios		Summary of Moment Values			es Summary of Shear		ear Values			
Segmen	it Length	Span #	M	V	Mmax +	Mmax -	Ма Мах	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	Maximu	m Deflec	tions											

Load Combination

+D+0.750L+0.750S

Location in Span

0.000



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

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Steel Beam

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Lic. # : KW-06002886

Description:

B9 - DECK SUPPORT

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	N-90000000			
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				



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 20 JUN 2017. 1:50PM

**Wood Beam** 

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Lic. # : KW-06002886

Description:

B10 - FLOOR JOISTS w/ PARTITION

#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

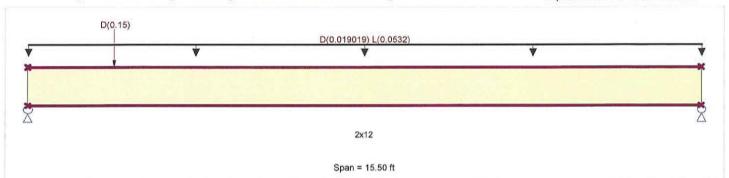
Load Combination Set: ASCE 7-10

#### **Material Properties**

Analysis Method: Allowable Stress Design 875.0 psi E: Modulus of Elasticity Fb - Tension 1,400.0ksi 875.0 psi Ebend-xx Load Combination ASCE 7-10 Fb - Compr 510.0ksi Fc - Prll 1,150.0 psi Eminbend - xx Fc - Perp 425.0 psi : Spruce - Pine - Fir Wood Species 135.0 psi F۷ Wood Grade : No. 1/No. 2 450.0 psi

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

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)

I Offit Load . D Office it (a) Lie it, (	* * / 11/				
DESIGN SUMMARY					Design OK
Maximum Bending Stress Ratio Section used for this span	=	0.875 1 P 2x12	Maximum Shear Stress Ratio Section used for this span	=	0.412 : 1 2x12
fb : Actual	=	880.42psi	fv : Actual	=	55.55 psi
FB : Allowable	=	1,006.25psi	Fv : Allowable	=	135.00 psi
Load Combination Location of maximum on span Span # where maximum occurs	=	+D+L 7.467ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	= =	+D+L 0.000 ft Span # 1
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	n	0.279 in Ratio 0.000 in Ratio 0.409 in Ratio 0.000 in Ratio	= 0<360.0 = 454>=240.		

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	



VILLAGE NEST JJH MULTI UNIT PROJECT Project ID: 2017-0610

Printed: 7 JUL 2017, 10:46AM

Wood Beam Lic. # : KW-06002886 File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29 Licensee: RICHMOND HOFFMAYE

Description:

B10A - FLOOR JOISTS ABV. LANDING

#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

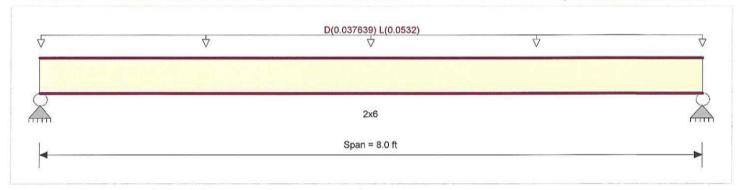
Load Combination Set: ASCE 7-10

## **Material Properties**

E: Modulus of Elasticity Analysis Method: Allowable Stress Design 875.0 psi Fb+ 875.0 psi 1,150.0 psi 1,400.0ksi Load Combination :ASCE 7-10 Fb -Ebend-xx 510.0ksi Fc - Prll Eminbend - xx 425.0 psi Fc - Perp Spruce - Pine - Fir Wood Species 135.0 psi F۷ Wood Grade : No. 1/No. 2 450.0 psi

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

Density 26.210pcf Repetitive Member Stress Increase



## **Applied Loads**

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

Values in KIPS

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

DESIGN SUMMARY					Design OK
Maximum Bending Stress Ratio Section used for this span	=	0.882 1 N 2x6	Maximum Shear Stress Ratio Section used for this span	=	0.436 : 1 2x6
fb : Actual	=	1,153.13psi	fv : Actual	=	58.83 psi
FB : Allowable	=	1,308.13psi	Fv : Allowable	=	135.00 psi
Load Combination Location of maximum on span Span # where maximum occurs	=	+D+L 4.000ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	=	+D+L 7.562ft Span # 1
Maximum Deflection Max Downward Transient Deflect Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection		0.169 in Ratio = 0.000 in Ratio = 0.289 in Ratio = 0.000 in Ratio =	= 0<360.0 = 331>=240.		*

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·v			Red		113

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	

Support notation: Far left is #1



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 21 JUN 2017, 9:06AM

Design OK

**Wood Beam** 

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Lic. # : KW-06002886

Description: B11 - FLOOR JOISTS w/ DECK SUPPORT

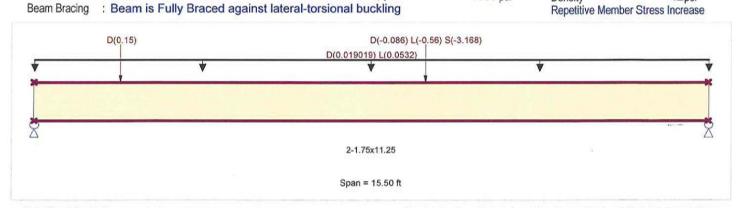
#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

Analysis Method : Allowable Stress Design	Fb - Tension	2600 psi	E : Modulus of Elastic	city
Load Combination ASCE 7-10	Fb - Compr	2600 psi	Ebend-xx	1900ksi
	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		
Hood Glado , Illiandeann EVE 110 E	Ft.	1555 psi	Density	42 ncf



## **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)

		D	Е	S	IG	N	S	U	M	N		4/	₹	Υ	
--	--	---	---	---	----	---	---	---	---	---	--	----	---	---	--

DECICIT COMMITTIE					
Maximum Bending Stress Ratio Section used for this span fb : Actual FB : Allowable	= =	0.532 1 A 2-1.75x11.25 1,829.45psi 3,438.50psi	Maximum Shear Stress Ratio Section used for this span fv : Actual Fv : Allowable	= = =	0.213 : 1 2-1.75x11.25 69.73 psi 327.75 psi
Load Combination Location of maximum on span Span # where maximum occurs	=	+D+S 8.995ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	=	+D+S 9.051 ft Span # 1
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection		0.002 in Ratio -0.005 in Ratio -0.046 in Ratio -0.350 in Ratio	= 33909>=360. = 4026>=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	

# RICHMOND HOFFMAYER INC. STRUCTURAL ENGINEERING



PROJECT VILLAGE NEST JOB 2017-0610

CLIENT 
ADDRESS EDEN, UT

HUUM

ENGINEER J.H. DATE 6/14/2017

RICHARD PE 49628 HOFFMAYER SE 5435 SHEFT 27 OF

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VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID:

2017-0610 3*C* 

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Design OK

**Wood Beam** 

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Description:

B11 - FLOOR JOISTS w/ DECK SUPPORT

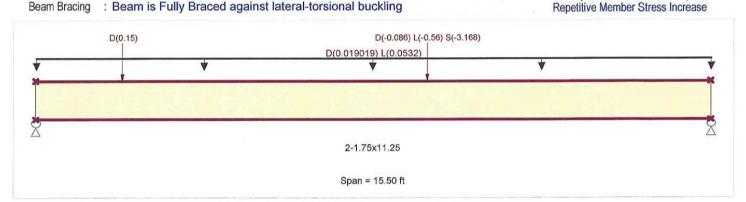
### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

E: Modulus of Elasticity Analysis Method: Allowable Stress Design 2600 psi Fb - Tension 1900ksi Load Combination ASCE 7-10 Fb - Compr 2600 psi Ebend-xx 2510 psi Fc - Prll Eminbend - xx 965.71ksi 750 psi Fc - Perp Wood Species : Trus Joist 285 psi Fv : MicroLam LVL 1.9 E Wood Grade 42pcf Ft 1555 psi Density



#### **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**

Max Upward Total Deflection

Maximum Bending Stress Ratio	=	0.532 1 N	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 Deflect	ion	0.002 in Ratio	= 92810>=360.		
Max Upward Transient Deflection	1000	-0.005 in Ratio	= 33909>=360.		
Max Downward Total Deflection		0.046 in Ratio			

531>=240.

-0.350 in Ratio =

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	



VILLAGE NEST JJH MULTI UNIT PROJECT

1555 psi

Project ID: 2017-0610

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42pcf

**Wood Beam** 

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Lic. #: KW-06002886

Description : B12 - FLOOR SUPPORT

#### **CODE REFERENCES**

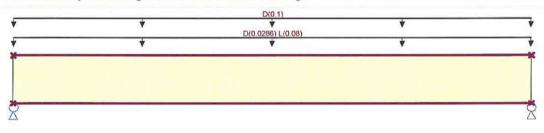
Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set : ASCE 7-10

#### **Material Properties**

Analysis Method: Allowable Stress Design Fb - Tension 2600 psi E: Modulus of Elasticity 2600 psi 1900ksi Load Combination :ASCE 7-10 Ebend-xx Fb - Compr Fc - Prll 2510 psi Eminbend - xx 965.71ksi Fc - Perp 750 psi Wood Species : Trus Joist 285 psi : MicroLam LVL 1.9 E F٧ Wood Grade

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



2-1.75x11.87

Span = 10.0 ft

#### **Applied Loads**

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

Density

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

6150>=360
0<360
2229>=240.
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	
I. Only	0.400	0.400	



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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**Wood Beam** 

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Description:

B13 - FLOOR SUPPORT ABV. STAIR

#### **CODE REFERENCES**

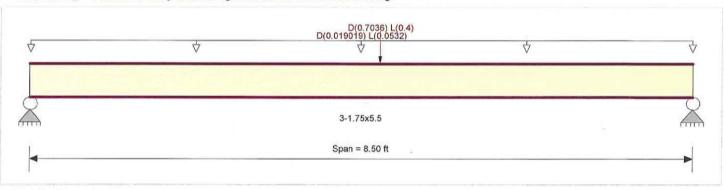
Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

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

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)

	(0, 1,00 11)	(-,-/			
DESIGN SUMMARY				200	Design OK
Maximum Bending Stress Ratio Section used for this span	=	0.534 1 3-1.75x5.5	Maximum Shear Stress Ratio Section used for this span	=	0.163 : 1 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 Location of maximum on span Span # where maximum occurs	=	+D+L 4.498ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	=	+D+L 8.066 ft Span # 1
Maximum Deflection Max Downward Transient Deflect Max Library Transient Deflection		0.109 in Ratio	= 932>=360		-pairir 1

0.109 in	Ratio =	932>=360
0.000 in	Ratio =	0<360
0.245 in	Ratio =	415>=240.
0.000 in	Ratio =	0<240.0
	0.000 in 0.245 in	0.109 in Ratio = 0.000 in Ratio = 0.245 in Ratio = 0.000 in Ratio =

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	¥



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

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**Wood Beam** 

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Lic. # : KW-06002886

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 Wood Grade

: Spruce - Pine - Fir : No. 1/No. 2

Beam Bracing : Completely Unbraced

Fb - Tension

875 psi 875 psi Fb - Compr 1150 psi Fc - Prll Fc - Perp 425 psi Fv

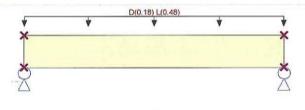
135 psi 450 psi Ft

E: Modulus of Elasticity

Ebend-xx Eminbend - xx

1400ksi 510ksi

26.21 pcf Density



4x6

Span = 3.50 ft

### **Applied Loads**

Beam self weight calculated and added to loads Uniform Load: D = 0.0150, L = 0.040 ksf, Tributary Width = 12.0 ft, (FLOOR)

DESIGN	SUMM	ARY
Maximum	Bending	Stress

s Ratio 0.610 1 Section used for this span 4x6 fb: Actual 690.92psi FB: Allowable 1,133.19psi +D+L

1.750ft Location of maximum on span Span # where maximum occurs Span #1

Maximum Shear Stress Ratio Section used for this span fv : Actual

Fv : Allowable Load Combination Location of maximum on span Span # where maximum occurs

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

135.00 psi +D+L 3.053ft Span #1

Design OK

0.499:1

4x6

67.36 psi

Maximum Deflection

Load Combination

Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection

0.024 in Ratio = 0.000 in Ratio = 0.033 in Ratio = 0.000 in Ratio =

1750>=360 0<360 1266>=240. 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



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

92

Printed; 21 JUN 2017, 10:11AM

Design OK

**Wood Beam** 

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Lic. #: KW-06002886

Description:

B15 - HEADERS

#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set : ASCE 7-10

#### **Material Properties**

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

Beam Bracing : Completely Unbraced



2-1.75x9.5

Span = 6.50 ft

## **Applied Loads**

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

Beam self weight calculated and added to loads

Max Downward Total Deflection

Max Upward Total Deflection

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**

DECIGIT COMMINATOR					The state of the s
Maximum Bending Stress Ratio Section used for this span	=	0.872 1 2-1.75x9.5	Maximum Shear Stress Ratio Section used for this span	=	0.719 : 1 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 Location of maximum on span	=	+D+S 3.250ft	Load Combination Location of maximum on span	. =	+D+S 5.717 ft
Span # where maximum occurs	=	Span #1	Span # where maximum occurs	=	Span # 1
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection		0.157 in Ratio			

433>=240.

0<240.0

0.180 in Ratio =

0.000 in Ratio =

Vertical Reactions		Support notation : Far	r 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			
I r Only	0.455	0.455			



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

**Wood Beam** 

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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	



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

94

Steel Beam

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Design OK

Lic. #: KW-06002886

Description: B16

**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

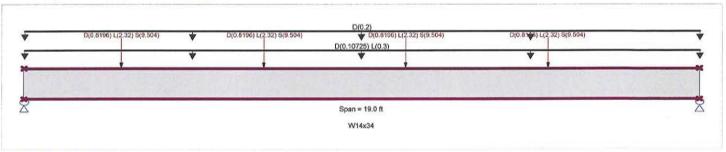
Bending Axis: Major Axis Bending

Fy: Steel Yield:

E: Modulus :

50.0 ksi

29,000.0 ksi



## **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**

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 Location of maximum on span	+D+0.750L+0.750S 10.749ft	Load Combination Location of maximum on span	+D+0.750L+0.750S 0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

#### Maximum Deflection

naximum Deliection		
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	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	100		A ST IDAN ST AT			U LOUTE AL TRACT	0.000						
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
<b>Vertical Reactions</b>			Support	notation : Far left is #1	Values in KIPS	

Load Combination	Support 1	Support 2
Overall MAXimum	26.284	23.225



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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Steel Beam

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Lic. #: KW-06002886 Description: B16 - BA

**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		



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID:

2017-0610

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Design OK

W10x22

0.801:1

39.237 k

48.960 k

+D+S

Span #1

7.000 ft

Steel Beam

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Lic. # : KW-06002886 Description : B17 -

**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

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

Bending Axis: Major Axis Bending

Fy: Steel Yield: E: Modulus: 50.0 ksi

29,000.0 ksi

D(0.0288) L(0.08)

D(0.0288) L(0.08)

W10x22

#### **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**

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

Maximum Deflection

 Max Downward Transient Deflection
 0.005 in Ratio = 16,041 >= 360

 Max Upward Transient Deflection
 0.000 in Ratio = 0 < 360</td>

 Max Downward Total Deflection
 0.011 in Ratio = 0 < 240.0</td>

 Max Upward Total Deflection
 0.000 in Ratio = 0 < 240.0</td>

#### **Maximum Forces & Stresses for Load Combinations**

Load Combina	ation		Max Stress	Ratios		8	Summary of M	loment Valu	es			Summa	ary of Sh	ear Values
Segment	t Length	Span #	M	V	Mmax +	Mmax -	Ма Мах	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only	A 100 (A 100 (A))(A 100 (A 100 (A 100 (A 100 (A 100 (A))(A 100 (A))(A 100 (A))(A 100 (A 100 (A))(A 100	0.00	0.003.000000	1030900000000	UWWWW		00e/0/00	0.907.0000.000.000	#1000F00000	(SAC) 9 82 4044		1007/00/1940	, www.com	25000 0 00000
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+0.1	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	t notation : Far left is #1	Values in KIPS	
Load Combination	Support 1	Support 2	1016			

moud communication	- approx.	- arte to - c - m
Overall MAXimum	6.687	39.237
Overall MINimum	0.280	0.280
D Only	0.621	2.843



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

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Steel Beam

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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		



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

2017-0610 Project ID:

Printed: 21 JUN 2017, 11:03AM

Steel Column

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Lic. #: KW-06002886

**B17 SUPPORT** Description:

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** 

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 7.275 k-ft

Mn-x / Omega: Allowable Ma-y: Applied Mn-y / Omega: Allowable

PASS Maximum Shear Stress Ratio = Load Combination

Location of max.above base At maximum location values are . . . Va : Applied

Vn / Omega: Allowable

Maximum SERVICE Load Reactions . .

0.0 k 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

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

Maximum SERVICE Load Deflections . . .

Along Y-Y 0.0 in at

for load combination: Along X-X 0.0 in at

0.0ft above base

0.0ft above base

for load combination:

0.000

PASS

**Load Combination Results** 

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

PASS

0.039

0.0 k-ft

7.275 k-ft

0.0:1

0.0 ft

0.0 k 0.0 k

Maximum Reactions

+0.60D

0.00 ft Note: Only non-zero reactions are listed.

illustrillarii i todotiono								Annual State of the state of th			
	Axial Reaction	al Reaction X-X Axis Reaction		k	k Y-Y Axis Reaction		Mx - End M	oments k-ft	My - End	My - End Moments	
Load Combination	@ Base	@ Base	@ Тор		@ Base	@ Тор	@ Base	@ Top	@ Base	@ Тор	
D Only	2.951	50 5 W. J. J. F.	10.000.00								
+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										

0.00 ft



VILLAGE NEST

JJH Pri MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 21 JUN 2017, 11:03AM

13.600 in^4

File = d:\ENERCALC Projects\2017-0610.ec6 Steel Column ENERCALC, INC. 1983-2017, Build:6.17.3.17, Ver:6.17.3.17 Lic. #: KW-06002886 Licensee: RICHMOND HOFFMAYER **B17 SUPPORT** Description: **Maximum Reactions** Note: Only non-zero reactions are listed. My - End Moments Axial Reaction X-X Axis Reaction Y-Y Axis Reaction Mx - End Moments k-ft @ Base @ Base @ Top Load Combination @ Base @ Base @ Тор @ Base @ Top @ Top 2.906 Lr Only L Only 0.280 36.394 S Only **Extreme Reactions** Axial Reaction X-X Axis Reaction Y-Y Axis Reaction Mx - End Moments My - End Moments Item Extreme Value @ Base @ Base @ Top @ Base @ Top @ Base @ Top @ Base @ Top Axial @ Base Maximum 39.345 0.280 Minimum 2.951 Reaction, X-X Axis Maximum Minimum 2.951 2.951 Reaction, Y-Y Axis Maximum 2.951 Minimum Reaction, X-X Axis Maximum 2.951 2.951 Minimum 2.951 Reaction, Y-Y Axis Maximum Minimum 2.951 2.951 Moment, X-X Axis Ba Maximum Minimum 2.951 Moment, Y-Y Axis Ba Maximum 2.951 2.951 Minimum 2.951 Moment, X-X Axis To Maximum Minimum 2.951 2.951 Moment, Y-Y Axis To Maximum Minimum 2.951 **Maximum Deflections for Load Combinations** 

Load Combination	Max. X-X Defle	ection	Distance		Max. Y-Y Def	lection	Distanc	е	
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	
	Accessor of the Company of the Compa								

Steel Section Properties	s: Pi	pe4STD
--------------------------	-------	--------

OLCOL GOOGICH	100011100	and the second	.,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,				
Depth	=	4.500	in	l xx	=	6.82 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	l yy	=	6.820 in^4	
Weight	=	10.800	plf	S yy	=	3.030 in^3	
				R yy	=	1.510 in	

Ycg

=

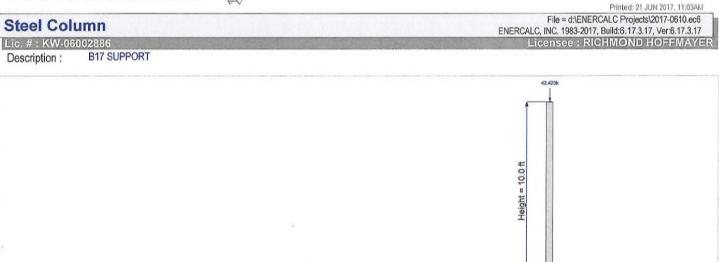
0.000 in



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

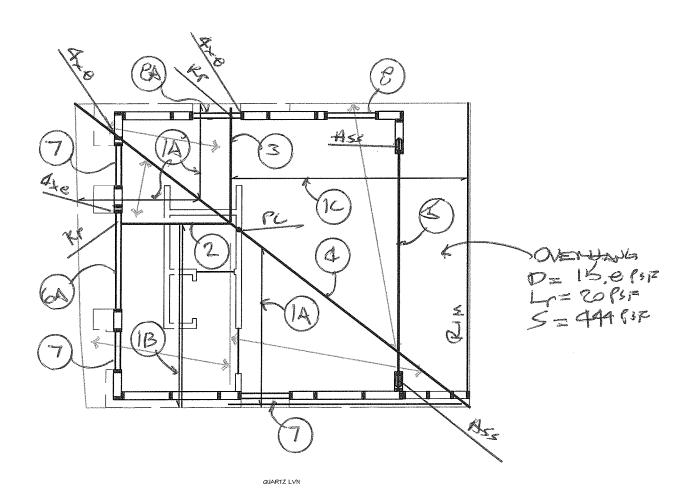
Loads are total entered value. Arrows do not reflect absolute direction.

Project ID: 2017-0610





FELDSPAR LOWER	JOB	2017-0610
VILLAGE NEST		
EDEN, UTAH		
HUUM		
J.H.	DATE	6/16/2017
RICHMOND PE 49628 HOFFMAYER SE 3935	SHEET	51_ OF
	VILLAGE NEST EDEN, UTAH HUUM J.H.	VILLAGE NEST EDEN, UTAH HUUM J.H. DATE



ROOF FRAMING PLAN BUALTZ - LOVER N.T.S.





VILLAGE NEST JJH

1,555.0 psi

MULTI UNIT PROJECT

Project ID: 2017-0610

52

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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

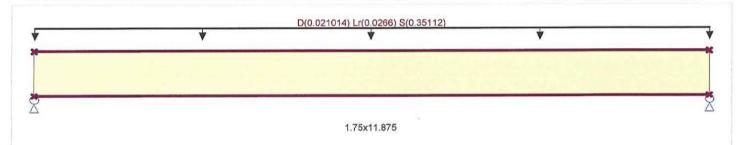
Load Combination Set: ASCE 7-10

#### **Material Properties**

E: Modulus of Elasticity Analysis Method: Allowable Stress Design 2,600.0 psi Fb - Tension Fb - Compr 1,900.0ksi Load Combination ASCE 7-10 2,600.0 psi Ebend-xx 965.71 ksi 2,510.0 psi Eminbend - xx Fc - Prll 750.0 psi Fc - Perp Wood Species : Trus Joist 285.0 psi F۷ MicroLam LVL 1.9 E Wood Grade

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

Density 42.0pcf Repetitive Member Stress Increase



Span = 14.0 ft

## **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 Section used for this span	=	0.774 1 1.75x11.875	Maximum Shear Stress Ratio Section used for this span	=	0.494 : 1 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 Location of maximum on span	=	+D+S 7.000ft	Load Combination Location of maximum on span	=	+D+S 13.029ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection Max Downward Transient Deflect	ion	0.658 in Rat	io = 255>=240.		
Max Upward Transient Deflection	ı	0.000 in Rat			
Max Downward Total Deflection		0.697 in Rat	io = 240>=240.		
Max Upward Total Deflection		0.000 in Rat	io = 0<240.0		

Vertical Reactions		Support n	otation : 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			



**VILLAGE NEST** JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

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**Wood Beam** 

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Lic. #: KW-06002886

Description: **B1B-RAFTERS** 

#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

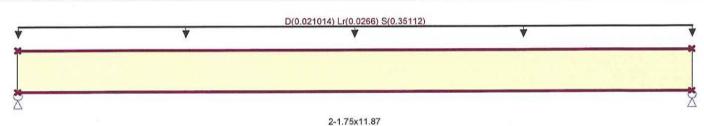
Load Combination Set: ASCE 7-10

## **Material Properties**

Analysis Method: Allowable Stress Design	Fb - Tension	2,600.0 psi	E: Modulus of Elasti	icity
Load Combination ASCE 7-10	Fb - Compr Fc - Prll	2,600.0 psi 2,510.0 psi	Ebend- xx Eminbend - xx	1,900.0ksi 965.71ksi
Wood Species : Trus Joist Wood Grade : MicroLam LVL 1.9 E	Fc - Perp Fv	750.0 psi 285.0 psi	5	42.0
	- Ft	1,555.0 psi	Density	42.0 pcf

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

42.0 pcf Density Repetitive Member Stress Increase



Span = 15.0 ft

#### **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)

Official Edda . D G.G.GGG, E. G		DIMED TO THOSE THEOREM ! THE COLOR	and the same of th		
DESIGN SUMMARY					Design OK
Maximum Bending Stress Ratio Section used for this span fb : Actual	=	0.444 1 2-1.75x11.87 1,526.82psi	Maximum Shear Stress Ratio Section used for this span fy: Actual	=	0.267 : 1 2-1.75x11.87 87.49 psi
FB : Allowable	=	3,438.50psi	Fv : Allowable	=	327.75 psi
Load Combination Location of maximum on span Span # where maximum occurs	=	+D+S 7.500ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	=	+D+S 14.015 ft Span # 1
Maximum Deflection Max Downward Transient Deflect Max Upward Transient Deflectio Max Downward Total Deflection Max Upward Total Deflection	n	0.434 in Ratio 0.000 in Ratio 0.459 in Ratio 0.000 in Ratio	0 = 0 < 240.0 0 = 391 >= 240.		

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 Ook	0.159	0.159	

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



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID:

2017-0610

Printed: 26 JUN 2017, 2:05PM

**Wood Beam** 

Description:

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Lic. #: KW-06002886

**B1C - RAFTERS w OVERHANG** 

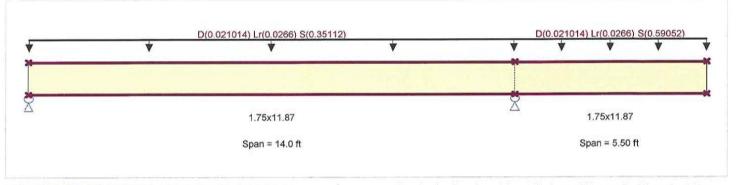
#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

Analysis Method: Allowable Stress Design	Fb - Tension	2,600.0 psi	E: Modulus of Elasti	city
Load Combination ASCE 7-10	Fb - Compr Fc - Prll	2,600.0 psi 2,510.0 psi	Ebend- xx Eminbend - xx	1,900.0ksi 965.71ksi
Wood Species : Trus Joist Wood Grade : MicroLam LVL 1.9 E	Fc - Perp Fv	750.0 psi 285.0 psi		100 100 100
	Ft	1,555.0 psi	Density	42.0pcf
Beam Bracing : Beam is Fully Braced against lateral-torsi	onal buckling	PARKETS SAFET SAFET	Repetitive Member	er Stress Increase



## **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)

-	 	 		
		 JIVIN	<i>n /</i> \	$\mathbf{D}_{\mathbf{V}}$

DESIGN SUMMARY					Design OK
Maximum Bending Stress Ratio Section used for this span	=	0.785 1 1.75x11.87	Maximum Shear Stress Ratio Section used for this span	=	0.642 : 1 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 Location of maximum on span Span # where maximum occurs	=	+D+S 14.000ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	= =	+D+S 13.061 ft Span # 1
Maximum Deflection Max Downward Transient Defle Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	on	0.282 in Ratio -0.062 in Ratio 0.272 in Ratio -0.072 in Ratio	0 = 2118>=240. 0 = 484>=240.		3

Vertical Reactions		Sup	oport notation : Far left is #1	Values in KIPS
Load Combination	Support 1	Support 2	Support 3	
Overall MAXimum	1.944	6.629	1000 000 000 000 000 000 000 000 000 00	
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		



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

**Wood Beam** 

Printed: 25 JUN 2017, 2:05PM

File = d:\ENERCALC Projects\2017-0610.ec6

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

Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886 Description: B1C - RA

B1C - RAFTERS w OVERHANG

Vertical Reactions		Sup	pport 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			



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610



Printed: 26 JUN 2017, 2:06PM

Wood 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 **B2 - ROOF BEAM** Description:

#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

Wood Species

Wood Grade

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

: Trus Joist

Parallam PSL 2.0E

: Beam is Fully Braced against lateral-torsional buckling

E: Modulus of Elasticity 2,900.0 psi Fb - Tension Ebend-xx

2,900.0 psi Fb - Compr Fc - Prll 2,900.0 psi 625.0 psi Fc - Perp

290.0 psi 2,025.0 psi

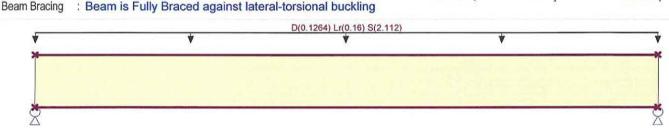
Density

Eminbend - xx

45.050pcf

2,000.0ksi

1,016.54ksi



F۷

5.25x11.875

Span = 11.0 ft

## **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)

DES	ICA	101	! N // N //	ARY	
DLO	101	00	uuuu	MILL	

DESIGN SUMMARY					Design OK
Maximum Bending Stress Ratio Section used for this span	=	0.996 1 5.25x11.875	Maximum Shear Stress Ratio Section used for this span	=	0.739 : 1 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 Location of maximum on span Span # where maximum occurs	=	+D+S 5.500ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	=	+D+S 10.036 ft Span # 1

#### Maximum Deflection

0.
0
0.
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	



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 26 JUN 2017, 2:06PM

**Wood Beam** 

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Lic. #: KW-06002886 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**

Wood Species

Wood Grade

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

: Trus Joist : Parallam PSL 2.0E

Fb - Compr 2,900.0 psi Fc - Prll 2,900.0 psi Fc - Perp 625.0 psi 290.0 psi

Fb - Tension

2,900.0 psi

2,025.0 psi

Density

Ebend-xx

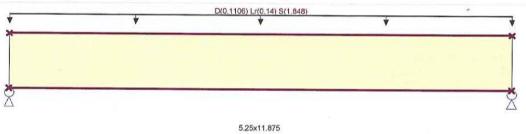
Eminbend - xx

E: Modulus of Elasticity

2,000.0ksi 1,016.54ksi

45.050 pcf

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



F٧

Span = 8.50 ft

#### **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 SHMMARY

DESIGN SUMMARY					Design OK
Maximum Bending Stress Ratio Section used for this span	=	0.521: 1 5.25x11.875	Maximum Shear Stress Ratio Section used for this span	=	0.469 : 1 5.25x11.875
fb : Actual	=	1,737.41 psi	fv : Actual	=	156.50 psi
FB : Allowable	=	3,335.00psi	Fv : Allowable	=	333.50 psi
Load Combination Location of maximum on span	=	+D+S 4.250ft	Load Combination Location of maximum on span	=	+D+S 0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection					C100/2 (000/100/1)

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 Passions

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	



VILLAGE NEST

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

2017-0610

Printed: 26 JUN 2017, 2:07PM

**Wood Beam** 

File = d:\(\text{LNERCALC}\) Projects\(\text{2017-0610.ec6}\)
ENERCALC, INC. 1983-2017, Build:\(\text{6.17.3.17}\), Ver:\(\text{6.17.3.17}\)
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Lic. # : KW-06002886

Description: B4 - RIDGE

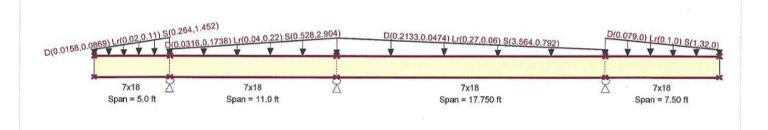
#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

### **Material Properties**

Analysis Method : A	llowable Stress Design	Fb - Tension	2,900.0 psi	E: Modulus of Elast	ticity	
Load Combination :A	SCE 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 : T	rus Joist	Fc - Perp	625.0 psi			
The same because the same of t	arallam PSL 2.0E	Fv .	290.0 psi			
rrood Orddo		Ft	2,025.0 psi	Density	45.050pcf	
Beam Bracing : B	eam is Fully Braced against lateral-torsional bu	ckling	Company of the Company of the Company		AND CONTROL OF STREET, BUCKEY	



#### **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, (and for Son Number 2)

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



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 26 JUN 2017, 2:07PM

59

**Wood Beam** 

Description:

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Lic. #: KW-06002886

Lr Only, LL Comb Run (LL\*\*)

Lr Only, LL Comb Run (LL\*L)

Lr Only, LL Comb Run (LLL\*)

Lr Only, LL Comb Run (LLLL)

S Only

B4 - RIDGE

Support notation: Far left is #1 Values in KIPS Vertical Reactions Support 5 Support 4 Load Combination Support 1 Support 2 Support 3 5.502 2.630 +D+Lr, LL Comb Run (\*\*L\*) 0.419 3.074 +D+Lr, LL Comb Run (\*\*LL) 0.445 5.407 +D+Lr, LL Comb Run (\*L\*\*) 1.278 4.107 1.668 2.112 +D+Lr, LL Comb Run (\*L\*L) 1.304 4.012 0.897 2.586 +D+Lr, LL Comb Run (\*LL\*) 6.498 0.924 3.030 +D+Lr, LL Comb Run (\*LLL) 6.402 +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 2.637 +D+Lr, LL Comb Run (L\*L\*) 0.812 5.428 +D+Lr, LL Comb Run (L\*LL) 0.838 5.332 3.081 1.670 4.033 1.675 +D+Lr, LL Comb Run (LL\*\*) +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) 6.328 3.037 1.316 19.198 7.627 45.573 +D+S +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 2.012 +D+0.750Lr, LL Comb Run (\*L\*L) 1.178 3.786 0.873 5.651 2.368 +D+0.750Lr, LL Comb Run (\*LL\*) +D+0.750Lr, LL Comb Run (\*LLL) 0.893 5.580 2.701 1.717 3.055 +D+0.750Lr, LL Comb Run (L\*\*\*) 1.094 +D+0.750Lr, LL Comb Run (L\*\*L) 1.113 2.984 2.050 0.809 4.848 2.406 +D+0.750Lr, LL Comb Run (L\*L\*) 2.739 +D+0.750Lr, LL Comb Run (L\*LL) 0.828 4.777 3.802 1.684 +D+0.750Lr, LL Comb Run (LL\*\*) 1.452 3.731 2.017 +D+0.750Lr, LL Comb Run (LL\*L) 1.472 +D+0.750Lr, LL Comb Run (LLL\*) 1.167 5.595 2.373 5.524 2.706 1.187 +D+0.750Lr, LL Comb Run (LLLL) 5.920 34.958 14.826 +D+0.750S +0.60D 0.479 1.867 1.027 Lr Only, LL Comb Run (\*\*\*L) 0.026 -0.0950.444 Lr Only, LL Comb Run (\*\*L\*) -0.3802.391 0.918 Lr Only, LL Comb Run (\*\*LL) -0.3542.295 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 0.125 3.291 1.318 Lr Only, LL Comb Run (\*LLL) 0.007 Lr Only, LL Comb Run (L\*\*\*) 0.393 -0.074Lr Only, LL Comb Run (L\*\*L) 0.419 -0.1700.451 2.316 0.925 Lr Only, LL Comb Run (L\*L\*) 0.012 Lr Only, LL Comb Run (L\*LL) 0.039 2.221 1.369

0.871

0.897

0.491

0.517

6.828

0.921

0.826

3.312

3.217

42.462

-0.038

0.407

0.881

1.325

17.485



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID:

2017-0610

Printed: 26 JUN 2017, 2:08PM

**Wood Beam** 

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

Lic. #: KW-06002886 Description:

**B5 - BIG OPENING** 

### **CODE REFERENCES**

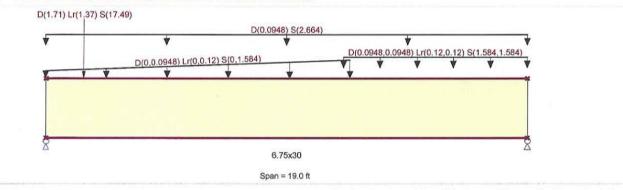
Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

Analysis Method: Allowable Stress Design E: Modulus of Elasticity 2,400.0 psi Fb - Tension 1,800.0ksi Load Combination :ASCE 7-10 1,850.0 psi Ebend-xx Fb - Compr Fc - Prll 1,650.0 psi Eminbend - xx 950.0ksi 650.0 psi 1,600.0ksi Fc - Perp Ebend-yy DF/DF Wood Species Fv 265.0 psi Eminbend - yy 850.0ksi : 24F - V4 Wood Grade 1,100.0 psi 31.20pcf Density

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 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				- 100	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 0.000 in Ratio = Max Upward Transient Deflection 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	

52.463	42.232
1.845	1.115
3.353	2.248
5.198	3.363
52.463	42.232
4.736	3.085
40.186	32.236
2.012	1.349
	1.845 3.353 5.198 52.463 4.736 40.186



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

**Wood Beam** 

Lic. #: KW-06002886

Description: B5 - BIG OPENING

Printed: 26 JUN 2017, 2:08PM

File = d:\ENERCALC Projects\2017-0610.ec6

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Description:

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		



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

7

Printed: 26 JUN 2017, 2:09PM

Steel Beam

Lic. #: KW-06002886

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

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

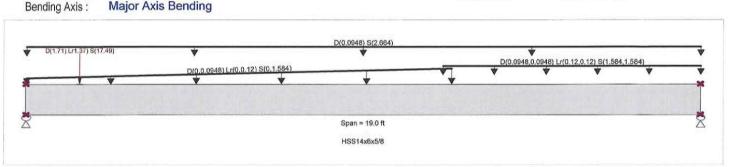
Beam Bracing:

Completely Unbraced Major Axis Bending Fy: Steel Yield:

46.0 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

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

Maximum Forces & Stresses for Load Combinations

Load Combin	ation		Max Stress	Ratios		S	Summary of M	oment Valu	ies			Summ	ary of Sh	ear Values
Segmen	it Length	Span #	М	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only														
Dsgn. L = +D+Lr	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
Dsgn. L = +D+S	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
THE RESERVE OF THE PARTY OF THE	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
Dsgn. L = +D+0.750S	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
Dsgn. L = +0.60D	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
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. "-" Defi	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.9512	9.500		0.0000	0.000
Vertical Reactions			Support	t 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



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 26 JUN 2017, 2:09PM

Steel Beam

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

Lic. # : KW-06002886 Description:

B5 - BIG OPENING - STEEL OPTION

Values in KIPS

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		
DATE OF THE PARTY				



**VILLAGE NEST** JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 26 JUN 2017, 2:10PM

#### **Wood Beam**

Lic. #: KW-06002886

**B6A - HEADER** Description:

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

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#### **CODE REFERENCES**

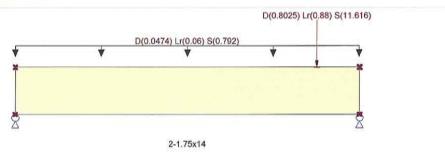
Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

Analysis Method: Allowable Stress Design 2,600.0 psi E: Modulus of Elasticity Fb - Tension 1,900.0ksi 2,600.0 psi Load Combination ASCE 7-10 Fb - Compr Ebend- xx Fc - Prll 2,510.0 psi Eminbend - xx 965.71 ksi 750.0 psi Fc - Perp : Trus Joist Wood Species 285.0 psi F۷ : MicroLam LVL 1.9 E Wood Grade 42.0pcf Ft 1,555.0 psi Density

Beam Bracing : Completely Unbraced



Span = 8.0 ft

## **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)

DES	ICA	ICI	INAR	IAL	VC
DES	UIV	30	JIVII	HAI	11

Max Upward Total Deflection

DESIGN SUMMARY					Design OK
Maximum Bending Stress Ratio Section used for this span	=	0.545 1 N 2-1.75x14	Maximum Shear Stress Ratio Section used for this span	=	0.369 : 1 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 Location of maximum on span Span # where maximum occurs	= =	+D+S 5.839ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	= =	+D+S 0.000 ft Span # 1
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection	n	0.101 in Ratio = 0.000 in Ratio = 0.107 in Ratio =	= <mark>0</mark> <240.0		

0<240.0

0.000 in Ratio =

Vertical Reactions		Support notation : Far left is #1	Values in KIPS
Load Combination	Support 1	Support 2	1000 MM (A-10)
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	



VILLAGE NEST JJH

1,555.0 psi

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

<u>6500</u>

Printed: 26 JUN 2017, 2:11PM

42.0 pcf

**Wood Beam** 

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Lic. # : KW-06002886

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 2,600.0 psi E: Modulus of Elasticity Fb - Tension Load Combination ASCE 7-10 1,900.0ksi 2,600.0 psi Fb - Compr Ebend- xx Fc - Prll 2,510.0 psi Eminbend - xx 965.71 ksi Fc - Perp 750.0 psi Wood Species : Trus Joist Fv 285.0 psi Wood Grade : MicroLam LVL 1.9 E

Beam Bracing : Completely Unbraced

D(0.1264) Lr(0.16) S(2.112)

Ft

2-1.75x7.25

Span = 4.50 ft

## **Applied Loads**

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

Density

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

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 Deflect	ion	0.093 in Rati	o = 581>=240.		
Max Upward Transient Deflection		0.000 in Rati	o = 0<240.0		
Max Downward Total Deflection		0.098 in Rati			
Max Upward Total Deflection		0.000 in Rati	o = 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	



VILLAGE NEST

MULTI UNIT PROJECT

Project ID: 2017-0610

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Printed: 26 JUN 2017, 2:11PM

**Wood Beam** 

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Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886

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		Fb - Tension	2,600.0 psi	E : Modulus of Elastic	city
Load Combination ASCE 7-10		Fb - Compr	2,600.0 psi	Ebend-xx	1,900.0ksi
	7	Fc - Prll	2,510.0 psi	Eminbend - xx	965.71ksi
Wood Species : Trus Joist		Fc - Perp	750.0 psi		
Wood Grade : MicroLam LVL 1.9 E		Fv	285.0 psi		
A STATE OF STATES AND		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 = 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	=	2,577.66psi	fv : Actual	=	219.14 psi
FB : Allowable	=	2,875.23psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	2.125ft	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 Deflect	on	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		

Span = 4.250 ft

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	



VILLAGE NEST

MULTI UNIT PROJECT

Project ID: 2017-0610

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Printed: 26 JUN 2017, 12:34PM

## **Wood Beam**

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Lic. #: KW-06002886

Description: B8A - BEARING HEADERS

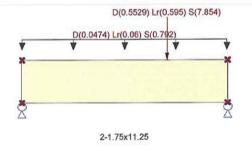
## **CODE REFERENCES**

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 Elasti	city	
Load Combination ASCE 7-10	Fb - Compr Fc - Prll	2,600.0 psi 2,510.0 psi	Ebend- xx Eminbend - xx	1,900.0ksi 965.71ksi	
Wood Species : Trus Joist Wood Grade : MicroLam LVL 1.9 E	Fc - Perp Fv Ft	750.0 psi 285.0 psi 1,555.0 psi	Density	42.0pcf	
Beam Bracing : Completely Unbraced	108	т,осо.оры	Density	.2.000	



Span = 4.250 ft

## **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 Section used for this span fb : Actual FB : Allowable	=	0.496 1 2-1.75x11.25 1,459.71psi 2,942.27psi	Maximum Shear Stress Ratio Section used for this span fv: Actual Fv: Allowable	=	0.806 : 1 2-1.75x11.25 264.26 psi 327.75 psi
Load Combination		+D+S	Load Combination	=	+D+S 3.319ft
Location of maximum on span Span # where maximum occurs	=	2.994ft Span # 1	Location of maximum on span Span # where maximum occurs	=	Span # 1
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection		0.029 in Rati 0.000 in Rati 0.031 in Rati 0.000 in Rati	o = 0<240.0 o = 1635>=240.		

Vertical Reactions		Suppor	t 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			

# RICHMOND HOFFMAYER INC.



WIND ON EXTERIOR WALLS: 115 MPH - EXP"C" ASCE 7-10 WMN = 16 PSF 9 = 0,00256 K2 Kx Kx Kx V2 Kz= 0.9 e 20' TABLE 30.3-1 K2= 1 KA= 0.85 9=0.00256(0.9X1X0.85)115 = 25.9 85F P= 26,9(64-642) A STUD = 10' × 1.33' = 13.362 -> 6Cp = 1.6 GCP2 = 0.18 P=26.9(1.0+0.10)=51,2-0157 > 169512 ALL WALL CLAPPING MUST RESIST \$1,313F



**VILLAGE NEST** JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 26 JUN 2017, 2:14PM

**Wood Column** 

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Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886 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 End Fixities Overall Column F	Top & Bolleight	e Stress Designation Pinned	O 100	Wo	ood Section Name ood Grading/Manuf. ood Member Type	The state of the s	d Lumber	
Wood Species Wood Grade Fb + Fb - Fc - Prll Fc - Perp	Spruce - Pir No.1 850.0 psi 850.0 psi 700.0 psi 425.0 psi	re - Fir Fv Ft Density	125.0 psi 550.0 psi 26.210 pci	Ex	act Width act Depth Area Ix Iy	3.50 in 7.250 in 25.375 in <sup>2</sup> 111.148 in <sup>4</sup> 25.904 in <sup>4</sup>	Cf or Cv for Tension	1.30 1.050 1.20 1.0 1.0
E : Modulus of Ela	asticity	x-x Bending	y-y Bending	Axial			Kf : Built-up columns	1.0 NDS 15.3.2
	Basic Minimum	1,300.0 470.0	1,300.0 470.0	1,300.0 ksi		eflection (bucklin	Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns:

X-X (width) axis:

Fully braced against buckling along X-X Axis Unbraced Length for X-X Axis buckling = 10 ft, K = 1.0

Y-Y (depth) axis:

# **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**

Bendin	g & Shear Check Results	
PASS	Max. Axial+Bending Stress Ratio = Load Combination	0.9256:1 +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 = Load Combination	0.1272 : 1 +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 Top along X-X 0.0 k

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

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.3922 in at

Along X-X

5.285 ft above base

for load combination: W Only

0.0 in

0.0 ft above base

for load combination: n/a

Other Factors used to calculate allowable stresses . . .

Bending

Compression

Tension

## **Load Combination Results**

	925	927	Maximum Axial	+ Bending	Stress Ratios	Maximum Shear Ratios			
Load Combination	CD	CP	Stress Ratio	Status	Location	Stress Ratio	Status	Location	
D Only	0.900	0.877	0.1052	PASS	0.0ft	0.0	PASS	10.50 f	
+D+Lr	1.250	0.815	0.1604	PASS	0.0 ft	0.0	PASS	10.50 f	
+D+S	1.150	0.834	0.9256	PASS	O.Oft	0.0	PASS	10.50 f	
+D+0.750Lr	1.250	0.815	0.1407	PASS	0.0ft	0.0	PASS	10.50 f	
+D+0.750S	1.150	0.834	0.7159	PASS	O.Oft	0.0	PASS	10.50 f	
+D+0.60W	1.600	0.750	0.3970	PASS	5.285ft	0.1272	PASS	0.0 f	
+D+0.750Lr+0.450W	1.600	0.750	0.3184	PASS	5.285ft	0.09543	PASS	0.0 f	
+D+0.750S+0.450W	1.600	0.750	0.7653	PASS	5.215ft	0.09543	PASS	0.0 f	
+0.60D+0.60W	1.600	0.750	0.3869	PASS	5.285ft	0.1272	PASS	0.0 f	



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 26 JUN 2017, 2:14PM File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:6.17.3.17, Ver:6.17.3.17

Licensee: RICHMOND HOFFMAYER

# **Wood Column**

Lic. #: KW-06002886

Description:

**ROOF SUPPORT POSTS - EXTERIOR WALLS** 

0.0000 in

0.000 ft

# **Load Combination Results**

Edda Combination Recalls												
		0		aximum Axia			atios			-	ear Ratios	3
Load Combination	CD	CP		Stress Ratio	Status	Locati	on	Stre	ess Ratio	Sta	atus Lo	cation
+0.60D	1.600	0.750		0.04153	PASS	(	0.0 ft		0.0	PA	SS	10.50 f
Maximum Reactions								Note: C	Only non-	zero re	actions a	are liste
The state of the s	X-X Axis R	eaction	k	Y-Y Axis Rea	ction A	xial Reaction	n My	y - End M	oments	k-ft	Mx - End	Moments
Load Combination	@ Base	@ Top	(	Base @	Тор	@ 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 Lo	oad Combinations											
Load Combination	Max. X-X Defle	ction [	Distance	N	ax. Y-Y Defi	lection I	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					
	272222		2002020	200	200200							

# S Only W Only **Sketches**

+0.60D

Lr Only

+D+0.750Lr

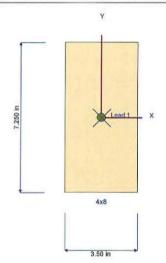
+D+0.750S

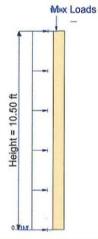
+D+0.60W

+D+0.750Lr+0.450W

+D+0.750S+0.450W

+0.60D+0.60W





0.000 ft

0.000 ft

5.285 ft

5.285 ft

5.285 ft

5.285 ft

0.000 ft

0.000 ft

0.000 ft

5.285 ft

0.000 in

0.000 in

0.235 in

0.176 in

0.176 in

0.235 in

0.000 in

0.000 in

0.000 in

0.392 in

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



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 26 JUN 2017, 2:15PM

# **Wood Column**

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

Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886 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 End Fixities Overall Column F	Top & Bo	e Stress Desi ottom Pinned		Wo	ood Section Name ood Grading/Manuf. ood Member Type	2-2x8 Graded Sawn	Lumber	
Wood Species Wood Grade Fb + Fb - Fc - Prll Fc - Perp	Spruce - Pir No.1 . 850.0 psi 850.0 psi 700.0 psi 425.0 psi	ne - Fir Fv Ft Density	125.0 psi 550.0 psi 26.210 pci	Ex	act Width act Depth Area Ix Iy	3.0 in 7.250 in 21.750 in <sup>2</sup> 95.270 in <sup>4</sup> 16.313 in <sup>4</sup>	Allow Stress Modification Factor Cr for Cv for Bending Cf or Cv for Compression Cf or Cv for Tension Cm: Wet Use Factor Ct: Temperature Factor Cfu: Flat Use Factor	1.20 1.050 1.20 1.0 1.0
E: Modulus of El	asticity	x-x Bending	y-y Bending	Axial			Kf : Built-up columns	1.0 NDS 15.3.2
	Basic Minimum	1,300.0 470.0	1,300.0 470.0	1,300.0 ksi	ace condition for de	flection (bucklin	Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns:

Fully braced against buckling along X-X Axis X-X (width) axis:

Unbraced Length for X-X Axis buckling = 10.50 ft, K = 1.0 Y-Y (depth) axis:

# **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 = Load Combination Governing NDS Forumla Location of max.above base	0.6013 : 1 +D+S Comp Only, fc/Fc' 0.0 ft
	At maximum location values are	0.0 %
	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 = Load Combination	0.07386 : 1 +D+0.60W

Maximum SERVICE Lateral Load Reactions . .

Bottom along Y-Y 0.5355 k Top 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 0.0 ft above base Along X-X 0.0 in

for load combination: n/a

Other Factors used to calculate allowable stresses . . .

Tension Bending Compression

# Allowable Shear **Load Combination Results**

Location of max.above base

Applied Design Shear

	12.1	CP	Maximum Axial	+ Bending	Stress Ratios	Maximum Shear Ratios			
oad Combination	CD		Stress Ratio	Status	Location	Stress Ratio	Status	Location	
D Only	0.900	0.862	0.04401	PASS	O.Oft	0.0	PASS	10.50 f	
+D+Lr	1.250	0.791	0.07490	PASS	O.Oft	0.0	PASS	10.50 f	
+D+S	1.150	0.812	0.6013	PASS	O.Oft	0.0	PASS	10.50 f	
+D+0.750Lr	1.250	0.791	0.06480	PASS	O.Oft	0.0	PASS	10.50 f	
+D+0.750S	1.150	0.812	0.4601	PASS	O.Oft	0.0	PASS	10.50 f	
+D+0.60W	1.600	0.719	0.2416	PASS	5.285ft	0.07386	PASS	0.0 f	
+D+0.750Lr+0.450W	1.600	0.719	0.1868	PASS	5.215ft	0.05540	PASS	0.0 f	
+D+0.750S+0.450W	1.600	0.719	0.3744	PASS	5.215ft	0.05540	PASS	0.01	
+0.60D+0.60W	1.600	0.719	0.2391	PASS	5.285ft	0.07386	PASS	0.0 f	

0.0 ft 22.159 psi

200.0 psi



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

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# **Wood Column**

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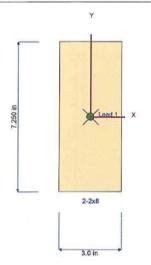
Lic. #: KW-06002886

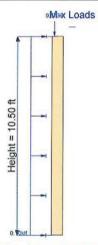
2ND FLOOR BEARING STUDS @ 48" o.c. - SIP SPLINES

# **Load Combination Results**

				Maximum	Axial	+ Bendin	g Stress Ratios		Maxim	um Sh	ear Rati	os
Load Combination	CD	CP		Stress R	atio	Status	Location	Stre	ess Ratio	Sta	atus	Location
+0.60D	1.600	0.719		0.017	79	PASS	0.0 ft		0.0	PA	ASS	10.50 f
Maximum Reactions								Note: C	only non-	zero r	eactions	s are liste
2 X 80 X A0 X	X-X Axis F	Reaction	k	Y-Y Axis	React	ion A	xial Reaction	My - End M	oments	k-ft	Mx - E	nd Moment
Load Combination	@ Base	@ Тор		@ Base	@ T	ор	@ Base	@ Base	@ Top		@ Base	@ Top
D Only							0.546				11,000,000	
+D+Lr							1.185					
+D+S							8.976					
+D+0.750Lr							1.025					
+D+0.750S							6.868					
+D+0.60W				0.321	(	).321	0.546					
+D+0.750Lr+0.450W				0.241	(	).241	1.025					
+D+0.750S+0.450W				0.241	(	.241	6.868					
+0.60D+0.60W				0.321	(	.321	0.327					
+0.60D							0.327					
Lr Only							0.639					
S Only							8.430					
W Only				0.536	(	).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.



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID:

2017-0610

Printed: 26 JUN 2017, 2:15PM

Steel Column

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Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886

RIDGE SUPPORT Description:

# 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 Fy: Steel Yield

A-36, Carbon Steel, Fy = 36 ksi

E: Elastic Bending Modulus

36.0 ksi 29,000.0 ksi Overall Column Height Top & Bottom Fixity

10.0 ft 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**

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**

Bendi	ng a	& Shea	r Chec	k Res	ults
0400		A. dal	Dandlan	Channe	Datie

PASS N	Max. Axial+Bending Stress Ratio = Load Combination	0.7135 +D+S	: 1
	Location of max.above base At maximum location values are	0.0	ft
	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 At maximum location values are . . . Va : Applied Vn / Omega : Allowable

Maximum SERVICE Load Reactions . .

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

0.0 in at

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

Maximum SERVICE Load Deflections . . .

0.0 in at Along Y-Y

0.0ft above base

for load combination:

Along X-X

0.0ft above base

for load combination:

# **Load Combination Results**

	Maximum Axial +	Bending S	tress Ratios	Maximu	atios	
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

0.0 ft

0.0 k 0.0 k

#### Maximum Reactions

Note: Only non-zero reactions are listed.

Axial Reaction	X-X Axis	Reaction	k	Y-Y Axis	Reaction	Mx - End M	oments k-ft	My - End	Moments
@ Base	@ Base	@ Top		@ Base	@ Top	@ Base	@ Тор	@ Base	@ Тор
3.219									
6.609									
45.679									
5.761									
35.064									
1.931									
3.390									
42.460									
	@ Base  3.219 6.609 45.679 5.761 35.064 1.931 3.390	@ Base @ Base  3.219 6.609 45.679 5.761 35.064 1.931 3.390	@ Base @ Base @ Top  3.219 6.609 45.679 5.761 35.064 1.931 3.390	@ Base @ Base @ Top  3.219 6.609 45.679 5.761 35.064 1.931 3.390	@ Base	@ Base	@ Base @ Base @ Top @ Base @ Top @ Base  3.219 6.609 45.679 5.761 35.064 1.931 3.390	@ Base @ Base @ Top @ Base @ Top @ Base @ Top  3.219 6.609 45.679 5.761 35.064 1.931 3.390	@ Base @ Base @ Top @ Base @ Top @ Base @ Top @ Base 3.219 6.609 45.679 5.761 35.064 1.931 3.390



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 26 JUN 2017, 2:15PM

Steel Column

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ENERCALC, INC. 1983-2017, Build:6.17.3.17, Ver:6.17.3.17
Licensee: RICHIMOND HOFFMAYER

13.600 in^4

Lic. #: KW-06002886

Description: RIDGE SUPPORT

	77 ( 70
E-America -	Reactions
EVITORIO	Reactions

	A DOMESTIC OF THE PARTY OF THE	Axial Reaction	X-X Axis	Reaction	k	Y-Y Axis	Reaction	Mx - End M	oments 1	k-ft	My - End	Moments
Item	Extreme Value	@ Base	@ Base	@ Тор		@ Base	@ Top	@ Base	@ Тор		@ Base	@ Тор
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										
M.	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										
wardenessen was researched	Minimum	3.219										

# Maximum Deflections for Load Combinations

Load Combination	Max. X-X Defle	ection	Distance		Max. Y-Y Def	lection	Distanc	e
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
Office Decrion	LIODGILIGS '	I IDE TOLU

Depth	=	4.500	in	l xx	=	6.82	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	l yy	=	6.820	in^4
Weight	=	10.858	plf	S yy	=	3.030	in^3
				R yy	=	1.510	in

Ycg

0.000 in



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

Steel Column

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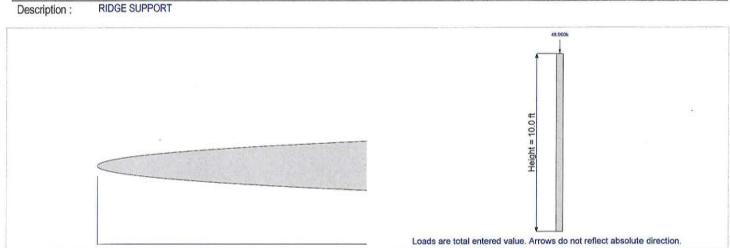
File = d:\ENERCALC Projects\2017-0610.ec6

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

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RIDGE SUPPORT





VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

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Steel Column

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Lic. #: KW-06002886

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: Analysis Method:

HSS12x6x5/8 Allowable Strength

Steel Stress Grade

Fy: Steel Yield E: Elastic Bending Modulus

46.0 ksi 29,000.0 ksi Overall Column Height Top & Bottom Fixity

10.0 ft 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**

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

# 115E FOR ATTEMPL OFTION

### **DESIGN SUMMARY**

Bending & Shear Check Results

0.1324 : 1 PASS Max. Axial+Bending Stress Ratio = 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 = Load Combination Location of max.above base At maximum location values are . . . Va: Applied

Vn / Omega : Allowable

Maximum SERVICE Load Reactions . .

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

Maximum SERVICE Load Deflections . . .

0.0 in at Along Y-Y 0.0ft above base

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

for load combination:

Along X-X 0.0 in at 0.0ft above base

for load combination:

#### Load Combination Results

	Maximum Axial +	Bending S	tress Ratios	Maximu	atios	
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

0.0:1

0.0 ft

0.0 k 0.0 k

#### **Maximum Reactions**

Note: Only non-zero reactions are listed. k-ft

My - End Moments

@ Top

@ Base

	Axial Reaction	X-X Axis	Reaction	k	Y-Y Axis	Reaction	Mx - End M	oments
Load Combination	@ Base	@ Base	@ Top		@ Base	@ Top	@ Base	@ Тор
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							



0.000 in

Project Title: Engineer: Project Descr:

VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

Steel Column

Ycg

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Lic. # : KW-06002886 Description: **B5 SUPPORT** 

		Ax	ial Reaction	1	X-X Axis React		k Y-Y Axis			Mx - Er			k-ft		nd Momen
Item	Extreme Valu	ue	@ Base		@ Base @	Top	@ Base	@ Top		@ Bas	е	@ Top		@ Base	@ To
Axial @ Base	Maximum		53.136	K											
	Minimum		1.840	0											
Reaction, X-X Axis	Maximum		4.026	li.											
	Minimum		4.026	R.											
Reaction, Y-Y Axis	Maximum		4.026	Ŕ											
•	Minimum		4.026	8											
Reaction, X-X Axis	Maximum		4.026												
•	Minimum		4.026												
Reaction, Y-Y Axis	Maximum		4.026											4	
	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												
ALTERNATION AS ALTERNATION OF ST.	N Aliminou uno		4.026												
	Minimum		4.020												
Maximum Doflac	Minimum	d Combin													
Maximum Deflect		THE RESERVE AND ADDRESS OF THE PARTY OF THE	nations		Distance	t ju	May V-V D	eflection		Distanc	e				
Load Combination		THE RESERVE AND ADDRESS OF THE PARTY OF THE	nations ex. X-X Def	ection	Distance	ft	Max. Y-Y D			Distanc	CFC				
Load Combination D Only		THE RESERVE AND ADDRESS OF THE PARTY OF THE	nations ex. X-X Def 0.0000	ection in	0.000	ft	0.000	) in		0.000	ft				
Load Combination  D Only +D+Lr		THE RESERVE AND ADDRESS OF THE PARTY OF THE	nations ax. X-X Def 0.0000 0.0000	ection in in	0.000 0.000	ft	0.000	) in ) in		0.000	ft ft				
D Only +D+Lr +D+S		THE RESERVE AND ADDRESS OF THE PARTY OF THE	nations ex. X-X Defi 0.0000 0.0000 0.0000	ection in in in	0.000 0.000 0.000	ft ft	0.000 0.000 0.000	) in ) in ) in		0.000 0.000 0.000	ft ft ft				
Load Combination  D Only +D+Lr +D+S +D+0.750Lr		THE RESERVE AND ADDRESS OF THE PARTY OF THE	nations ax. X-X Defi 0.0000 0.0000 0.0000 0.0000	ection in in in in	0.000 0.000 0.000 0.000	ft ft ft	0.000 0.000 0.000 0.000	in in in in in in in		0.000 0.000 0.000 0.000	ft ft ft ft				
D Only +D+Lr +D+S +D+0.750Lr +D+0.750S		THE RESERVE AND ADDRESS OF THE PARTY OF THE	0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	ection in in in in	0.000 0.000 0.000 0.000 0.000	ft ft ft	0.000 0.000 0.000 0.000	in i		0.000 0.000 0.000 0.000 0.000	ft ft ft ft				
D Only +D+Lr +D+S +D+0.750Lr +D+0.750S +0.60D		THE RESERVE AND ADDRESS OF THE PARTY OF THE	0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	ection in in in in in	0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft	0.000 0.000 0.000 0.000 0.000	in i		0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft				
Load Combination  D Only +D+Lr +D+S +D+0.750Lr +D+0.750S +0.60D Lr Only		THE RESERVE AND ADDRESS OF THE PARTY OF THE	0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	ection in in in in in in	0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft	0.000 0.000 0.000 0.000 0.000 0.000	in i		0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft ft				
Load Combination  D Only +D+Lr +D+S +D+0.750Lr +D+0.750S +0.60D Lr Only S Only	tions for Loa	Ма	0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	ection in	0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft	0.000 0.000 0.000 0.000 0.000	in i		0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft				
Load Combination  D Only +D+Lr +D+S +D+0.750Lr +D+0.750S +0.60D Lr Only S Only  Steel Section Pro	operties :	Ma	0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	ection in	0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft	0.000 0.000 0.000 0.000 0.000 0.000 0.000	in i		0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft ft				
Load Combination  D Only +D+Lr +D+S +D+0.750Lr +D+0.750S +0.60D Lr Only S Only  Steel Section Properth	operties :	HSS 12.000 in	0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 12x6x5/8	ection in ix ix ix	0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft	0.000 0.000 0.000 0.000 0.000 0.000 0.000	in i		0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft ft	-		271.000 in	
Load Combination  D Only +D+Lr +D+S +D+0.750Lr +D+0.750S +0.60D Lr Only S Only  Steel Section Properth	operties :	Ma	0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 12x6x5/8	ection in	0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	in i		0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft ft	= =		271.000 in 71.10 in	
Load Combination  D Only +D+Lr +D+S +D+0.750Lr +D+0.750S +0.60D Lr Only S Only  Steel Section Propetth Design Thick	operties :	HSS 12.000 in	0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	ection in ix ix ix	0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft	0.000 0.000 0.000 0.000 0.000 0.000 0.000	in i		0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft ft				
Load Combination  D Only +D+Lr +D+S +D+0.750Lr +D+0.750S +0.60D Lr Only S Only  Steel Section Properth Design Thick Width	operties :	HSS 12.000 in 0.581 in	0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	ection in in in in in in in in ix XX S xx	0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	in i		0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft ft				
Load Combination  D Only +D+Lr +D+S +D+0.750Lr +D+0.750S +0.60D Lr Only S Only  Steel Section Properth Design Thick Width Wall Thick	operties :	HSS 12.000 in 0.581 in 6.000 in	0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 12x6x5/8	ection in R xx S xx R xx Zx	0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 421.00 in^4 53.40 in^3 4.140 in	in i		0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 J	ft ft ft ft ft ft ft				^6
Load Combination  D Only +D+Lr +D+S +D+0.750Lr +D+0.750S +0.60D Lr Only S Only  Steel Section Properth Design Thick Width Wall Thick Area	operties :	HSS 12.000 in 0.581 in 6.000 in 0.624 in 18.700 in^2	0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 12x6x5/8	ection in ix XX S xx R xx Zx yy	0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft f	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 4.140 in 18.800 in 18.	in i	7	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft ft	=		71.10 in	^6
Load Combination  D Only +D+Lr +D+S +D+0.750Lr +D+0.750S +0.60D Lr Only S Only	operties :	HSS 12.000 in 0.581 in 6.000 in 0.624 in	nations 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 12x6x5/8	ection in R xx S xx R xx Zx	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft f	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 421.00 in^4 53.40 in^3 4.140 in 8.800 in^3	in i	7	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 J	ft ft ft ft ft ft ft	=		71.10 in	^6



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

Steel Column

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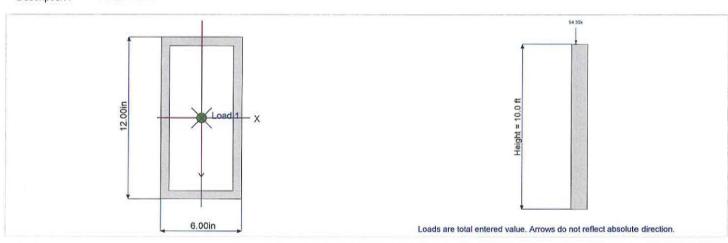
File = d:\ENERCALC Projects\2017-0610.ec6

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Lic. #: KW-06002886

Description: **B5 SUPPORT** 







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

SHEET 79 OF

13 12 13 13 10 13	MTC. GRATING
GUARTZ LVN	
QUARTZ LVN	



# RICHMOND HOFFMAYER



PROJECT	VILLAGE NEST		JOB	2017-0610
CLIENT	-	***		
<b>ADDRESS</b>	EDEN, UT			
DESIGN	HUUM			
ENGINEER	J.H.		DATE	6/14/2017
	PICAMONO DE 46428	HOPEMAYED SE BORS		0 05

METAL GRATING:
D = 12 PSF L = 60 PS 12 S = 264PSF - (OPENERS GRATING - POINT DBL SHOW)
LOAD COMSINASTIONS:
$D+L = 7LP37$ $D+3 = 276P37 \leftarrow GOVERNS$
P+0.76L+0.768 = 26685
SPAN M = W 9% (1- SPAN OR)
32" 2944 N-1L 40" 6624 N-1L 64" 11776 N-1L
MAJ. ALIMINUM STEEL
E = 10000 K/SL 29000 K/C F = 12000 PSi 18000 PSi MIN (3 SPAN IN MURES FSK > M & EIK > 5 W 1/460 0 x 240
CBA4/FE
CBR4/FI (76)= 27.4
$12000(5)27.4 > 2944 \longrightarrow 5 = 0.009 \text{ in}^3$ $12000(5)27.4 > 6614 \longrightarrow 5 = 0.020 \text{ in}^3$ $12000(5)27.4 > 11776 \longrightarrow 5 = 0.035 \text{ in}^3$

# RICHMOND HOFFMAYER INC. STRUCTURAL ENGINEERING



PROJECT	VILLAGE NEST	JOB	2017-0610	
CLIENT				
<b>ADDRESS</b>	EDEN, UT			
DESIGN	HUUM			
<b>ENGINEER</b>	J.H.	DATE	6/14/2017	
	RICHMOND PE 49628 HOFFMAYER SE 345	5 CHEET	O LOF	

Info@RichmondHoffmayer.com

12/1e(0,009) = 0,006123)
12/18(0,020) = 0.00(12) / 5TED.
12/18(0.035)=0.0233 IN3)
IK > 5 (276) \$460BE
T > 0.0109 l/
0.0109(323 / 10E6 = 0.0000 + INT ) 0.0109(403)/10E6 = 0.0001 INT (AL
0,0109(323)/10E6 = 0,0000 + IN4 0,0109(403)/10E6 = 0,0001 IN4 0,0109(643)/10E6 = 0,0003 IN4 0,0109(643)/10E6
10/(0,0000A) = 0,000013 1NA)
10/29 (0,000) = 0,000035 INA STEEL
10/29(0,0003)= 0,0001 1N4

| DEER X 3/6" AL = = 0.0313 IN3

I = 0.0156 IN4

V SE UP TO 4-8" SPANK U/ RARGE E7/6"

I DEER X 3/6" STEEL => = 0.0313 IN3

I = 0.0156 IN4

V SE UP TO 64" SPANK U/ RARGE E 7/6"

I X 3/6" AL. e 7/6" = 6 FASTING

UI SPANK e 4-6" = 425 CANOGA AVENUE WOODLAND HILLS CA 91364
PHONE (818) 347-7008 FAX (818) 883-8869



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

82

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Steel Beam

Lic. #: KW-06002886

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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 :

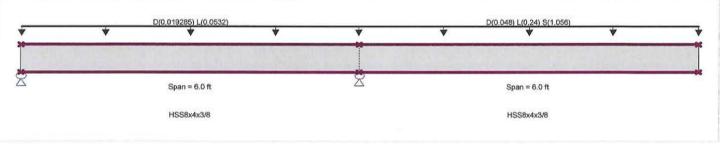
29,000.0 ksi

Max. "+" Defl

-0.0430

Location in Span

3.504



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

Maximum Deflection

Load Combination

 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.

Max. "-" Defl

0.0000

Span

1

# **Maximum Forces & Stresses for Load Combinations**

Load Combin	ation		Max Stress	Ratios		S	Summary of Me	oment Valu	ies		- 75	Summ	ary of Sh	ear Values
Segmen	nt Length	Span #	М	٧	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only				.acre.com		5.0110.600	WI MODES				100100000			
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						2102	(5157)	MEARE	4545		1337.5			150,500
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						1170705151	917787-09-0-101	1/2/27/27/2000	A 45-0 (CD)(C)	10000000	1000000	100 0000	0.0000000000000000000000000000000000000	00000000
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							2000000		1875	70/07/97/0				
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	Maximu	m Deflec	tions											

Load Combination

+D+0.750L+0.750S

Location in Span

0.000



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

03

Steel Beam

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Lic. # : KW-06002886

B9 - DECK SUPPORT

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				



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

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**Wood Beam** 

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**B10 - FLOOR JOISTS W/ PARTITION** 

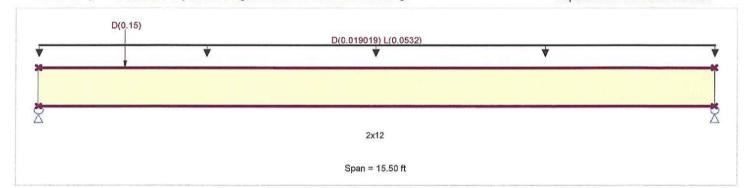
# **CODE REFERENCES**

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 Elasti	city
Load Combination :ASCE 7-10	Fb - Compr	875.0 psi	Ebend- xx	1,400.0ksi
	Fc - Prll	1,150.0 psi	Eminbend - xx	510.0ksi
Wood Species : Spruce - Pine - Fir	Fc - Perp	425.0 psi		
Wood Grade : No. 1/No. 2	Fv	135.0 psi		
2V - AS - S - CD - S - VS - S - CD	Ft	450.0 psi	Density	26.210pcf
Beam Bracing : Beam is Fully Braced against lateral-ton	sional buckling	WASHING AREA	Repetitive Member	er 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 Section used for this span	=	0.875 1 N 2x12	Maximum Shear Stress Ratio Section used for this span	=	0.412 : 1 2x12
fb : Actual	=	880.42psi	fv : Actual	=	55.55 psi
FB : Allowable	=	1,006.25psi	Fv : Allowable	=	135.00 psi
Load Combination Location of maximum on span Span # where maximum occurs	=	+D+L 7.467ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	= =	+D+L 0.000 ft Span # 1
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	70 TO 10 TO	0.279 in Ratio = 0.000 in Ratio = 0.409 in Ratio = 0.000 in Ratio =	= 0<360.0 = 454>=240.		

Vertical Reactions		Support notation : Far le	t 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			



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

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**Wood Beam** 

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B10A - FLOOR JOISTS ABV. LANDING

# **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

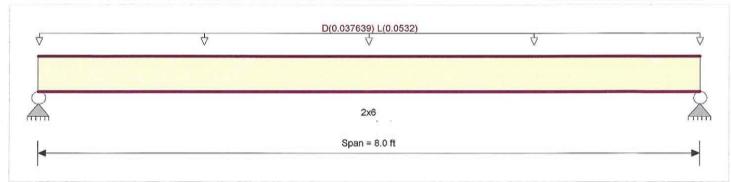
Load Combination Set: ASCE 7-10

# **Material Properties**

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

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

26.210 pcf Density 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 Section used for this span	=	0.882 1 P	Maximum Shear Stress Ratio Section used for this span	=	0.436 : 1 2x6
fb : Actual	=	1,153.13psi	fv : Actual	=	58.83 psi
FB : Allowable	=	1,308.13psi	Fv : Allowable	=	135.00 psi
Load Combination  Location of maximum on span  Span # where maximum occurs	=	+D+L 4.000ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	=	+D+L 7.562 ft Span # 1
Maximum Deflection Max Downward Transient Deflect Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	27,010	0.169 in Ratio 0.000 in Ratio 0.289 in Ratio 0.000 in Ratio	= 0<360.0 = 331>=240.		

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	



VILLAGE NEST

MULTI UNIT PROJECT

Project ID: 2017-0610

96

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**Wood Beam** 

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Description: B11 - FLOOR JOISTS w/ DECK SUPPORT

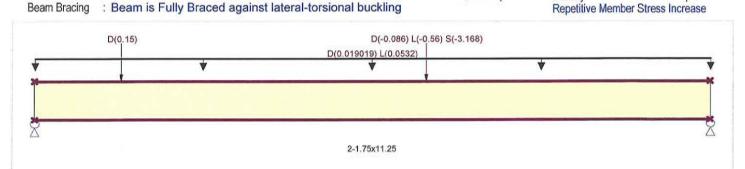
### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

# **Material Properties**

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



Span = 15.50 ft

# **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 Section used for this span	=	0.532 1 2-1.75x11.25	Maximum Shear Stress Ratio Section used for this span	=	0.213 : 1 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 Location of maximum on span Span # where maximum occurs	=	+D+S 8.995ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	=	+D+S 9.051 ft Span # 1
Maximum Deflection Max Downward Transient Deflect Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	30/7/04	0.002 in Ratio -0.005 in Ratio 0.046 in Ratio -0.350 in Ratio	9 = 33909 >= 360. 4026 >= 240.		

Vertical Reactions		Suppo	rt notation : Far left is #1	Values in KIPS	
Load Combination	Support 1	Support 2			
Overall MAXimum	-1.329	-1.839	4		
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			



VILLAGE NEST JJH

JJH MULTI UNIT PROJECT Project ID: 2017-0610

87

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**Wood Beam** 

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Description: B12 - FLOOR SUPPORT

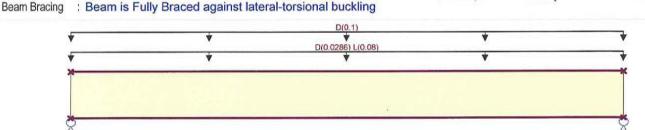
# **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

# **Material Properties**

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



2-1.75x11.87

Span = 11.0 ft

# **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 Section used for this span	=	0.187. 1 2-1.75x11.87	Maximum Shear Stress Ratio Section used for this span	=	0.127 : 1 2-1.75x11.87
fb : Actual	=	487.01psi	fv : Actual	=	36.14 psi
FB : Allowable	=	2,600.00psi	Fv : Allowable	=	285.00 psi
Load Combination Location of maximum on span Span # where maximum occurs	= =	+D+L 5.500ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	= =	+D+L 10.036 ft Span # 1
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	n	0.029 in Rat 0.000 in Rat 0.079 in Rat 0.000 in Rat	tio = 0<360 tio = 1674>=240.		

	Support notation : Far left is #1	Values in KIPS
Support 1	Support 2	
1.214	1.214	
0.440	0.440	
0.774	0.774	
1.214	1.214	
1.104	1.104	
0.464	0.464	
0.440	0.440	
	1.214 0.440 0.774 1.214 1.104 0.464	1.214 1.214 0.440 0.440 0.774 0.774 1.214 1.214 1.104 1.104 0.464 0.464



tle: VILLAGE NEST r: JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

4. C

Printed: 7 JUL 2017, 10:48AM

**Wood Beam** 

Description:

File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886

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**

Beam Bracing

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

D(0.019019) L(0.0532)

3-1.75x5.5

Span = 8.50 ft

# **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)

: Beam is Fully Braced against lateral-torsional buckling

Point Load: D = 0.7036, L = 0.40 k @ 4.50 ft, (B12)

#### Design OK DESIGN SUMMARY Maximum Shear Stress Ratio 0.163:1 Maximum Bending Stress Ratio . 0.534 1 Section used for this span Section used for this span 3-1.75x5.5 3-1.75x5.5 fv : Actual fb : Actual 46.34 psi 1,388,15psi Fv: Allowable 285.00 psi FB: Allowable 2,600.00psi +D+L Load Combination +D+L Load Combination 8.066 ft Location of maximum on span Location of maximum on span 4.498ft == Span #1 Span # where maximum occurs Span #1 Span # where maximum occurs Maximum Deflection Max Downward Transient Deflection 0.109 in Ratio = 932>=360 0.000 in Ratio = Max Upward Transient Deflection 0<360 0.245 in Ratio = Max Downward Total Deflection 415>=240. 0.000 in Ratio = Max Upward Total Deflection 0<240.0

Support notation : Far left is #1			Values in KIPS	
Support 1	Support 2			
0.862	0.927			
0.269	0.293			
0.448	0.489			
0.862	0.927			
0.758	0.818			
0.269	0.293			
0.414	0.438			
	0.862 0.269 0.448 0.862 0.758 0.269	Support 1 Support 2  0.862 0.927  0.269 0.293  0.448 0.489  0.862 0.927  0.758 0.818  0.269 0.293	0.862 0.927 0.269 0.293 0.448 0.489 0.862 0.927 0.758 0.818 0.269 0.293	Support 1 Support 2  0.862



**VILLAGE NEST** JJH

Project ID: 2017-0610 MULTI UNIT PROJECT

Printed: 26 JUN 2017, 2:27PM

**Wood 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

B14 - HEADERS Description:

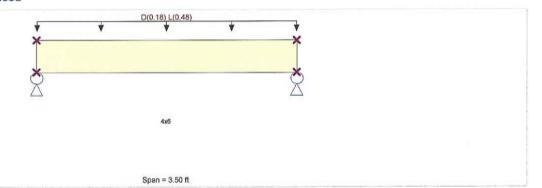
# **CODE REFERENCES**

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 Elasti	city
Load Combination ASCE 7-10	Fb - Compr	875.0 psi	Ebend- xx	1,400.0ksi
	Fc - Prll	1,150.0 psi	Eminbend - xx	510.0ksi
Wood Species : Spruce - Pine - Fir	Fc - Perp	425.0 psi		
Wood Grade : No. 1/No. 2	Fv	135.0 psi		
Wood Grade (1101 miles =	Ft	450.0 psi	Density	26.210pcf
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 = 12.0 ft, (FLOOR)

DESIGN SUMMARY					Design OK
Maximum Bending Stress Ratio Section used for this span	=	4x6	Maximum Shear Stress Ratio Section used for this span	=	0.499 : 1 4x6
fb : Actual	=	690.92psi	fv : Actual	=	67.36 psi
FB : Allowable	=	1,133.19psi	Fv : Allowable	=	135.00 psi
Load Combination Location of maximum on span	=	+D+L 1.750ft	Load Combination Location of maximum on span	=	+D+L 3.053 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=:	Span #1
Maximum Deflection Max Downward Transient Deflect Max Haward Transient Deflection	200727	0.024 in Ratio			
Max Upward Transient Deflection	п	0.000 in Ratio =			
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	



VILLAGE NEST

MULTI UNIT PROJECT

Project ID: 2017-0610

2

Printed: 26 JUN 2017. 2:51PM

**Wood Beam** 

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

Licensee: RICHMOND HOFFMAYER

Lic. # : KW-06002886

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	2,600.0 psi	E: Modulus of Elasti	city
Load Combination :ASCE 7-10	Fb - Compr	2,600.0 psi	Ebend-xx	1,900.0ksi
	Fc - Prll	2,510.0 psi	Eminbend - xx	965.71ksi
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	42.0pcf
Beam Bracing : Completely Unbraced			11/42/07/95/22/4/#5/7	DES COCKSTRACE



Span = 6,50 ft

# **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 Maximum Shear Stress Ratio 0.719:1 0.872 1 Section used for this span Section used for this span 2-1.75x9.5 2-1.75x9.5 fv : Actual fb : Actual 2,550.00psi 235.77 psi Fv : Allowable FB: Allowable 2,925.88psi 327.75 psi Load Combination Load Combination +D+S +D+S Location of maximum on span 3.250ft Location of maximum on span 5.717ft Span # where maximum occurs Span # where maximum occurs Span #1 Span #1 Maximum Deflection 0.157 in Ratio = Max Downward Transient Deflection 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		Suppo	ort 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			



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

**Wood Beam** 

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Values in KIPS

Lic. # : KW-06002886 Description: **B15 - HEADERS** 

Support notation: Far left is #1 **Vertical Reactions** Support 1 Support 2 Load Combination 0.260 0.260 L Only 6.006 6.006 S Only



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

32

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Steel Beam

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Licensee: RICHMOND HOFFMAYE

Lic. #: KW-06002886

**B16 - BALCONY SUPPORT** Description:

### **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

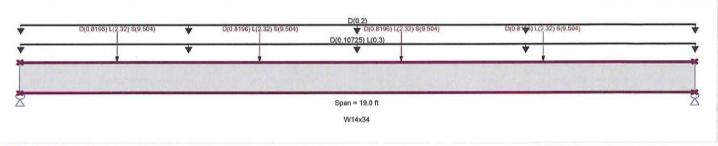
Major Axis Bending Bending Axis:

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

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

Maximum Bending Stress Ratio = 0.981:1 Section used for this span W14x34 Ma: Applied 133.584 k-ft Mn / Omega: Allowable

136.228 k-ft +D+0.750L+0.750S

10.749ft Span #1

Maximum Shear Stress Ratio = Section used for this span Va: Applied

Vn/Omega: Allowable Load Combination

Location of maximum on span Span # where maximum occurs

0.329:1 W14x34 26.284 k 79.80 k +D+0.750L+0.750S

Design OK

0.000 ft Span #1

Maximum Deflection

Load Combination

Location of maximum on span

Span # where maximum occurs

Max Downward Transient Deflection 0.259 in Ratio = 878>=360 0.000 in Ratio = Max Upward Transient Deflection 0 < 360 Max Downward Total Deflection 0.421 in Ratio = 541 >= 240. 0.000 in Ratio = Max Upward Total Deflection 0 < 240.0

# Maximum Forces & Stresses for Load Combinations

26.284

23.225

Load Combination		Max Stress	Ratios		Summary of Moment Values				Summary of Shear Values				
Segment Length	Span#	M	٧	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 +D+0.750L	1	0.960	0.320	130.73		130.73	227.50	136.23	1.00	1.00	25.52	119.70	79.80
Dsgn. L = 19.00 ft +D+0.750L+0.750S	1	0.395	0.137	53.85		53.85	227.50	136.23	1.00	1.00	10.90	119.70	79.80
Dsgn. L = 19.00 ft +0.60D	1	0.981	0.329	133.58		133.58	227.50	136.23	1.00	1.00	26.28	119.70	79.80
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**

Overall MAXimum

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				



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

Steel Beam

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ENERCALC, INC. 1983-2017, Build:6.17.3.17, Ver:6.17.3.17
Ligensee: RIGHWOND HOFFWAYER

Lic. # : KW-06002886

B16 - BALCONY SUPPORT Description:

Vertical Positions Values in KIPS Cupport potation : For loft in #1

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		



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 26 JUN 2017, 12:17PM

Steel Beam

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

Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886

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

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

Bending Axis:

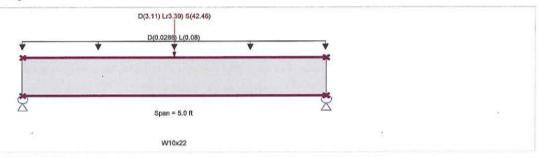
Major Axis Bending

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

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 = Section used for this span Ma : Applied Mn / Omega : Allowable	0.881 : 1 <b>W10x22</b> 57.121 k-ft 64.870 k-ft	Maximum Shear Stress Ratio = Section used for this span Va : Applied Vn/Omega : Allowable	0.468 : 1 W10x22 22.912 k 48.960 k
Load Combination Location of maximum on span Span # where maximum occurs	+D+S 2.500ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	+D+S 5.000 ft Span # 1
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	0.004 in Rati 0.000 in Rati 0.009 in Rati 0.000 in Rati	o = 0 <360 o = 6824 >=240.	

Maximum Forces & Stresses for Load Combinations

Load Combina	ation		Max Stress	Ratios		8	Summary of M	loment Valu	ies			Summa	ary of Sh	ear Values
Segmen		Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only	0.000.000		10000000	8331/83860	378430		14.000	A-17-5 (	20205330	NAMES OF	30.565627	188232	20170	700100
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
			A 7 A A A A A A A A A A A A A A A A A A	(CT/S/F/2T/1)	/ <del>-2</del> 40, 0.35)		A-10.0(E)							

	Overa	<b>Max</b>	imum	Def	lect	ions
--	-------	------------	------	-----	------	------

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	t notation : Far left is #1	Values in KIPS	
Load Combination	Support 1	Support 2				

Edda Combination	oupport i	Cabbone
Overall MAXimum	22.912	22.912
Overall MINimum	0.200	0.200
D Only	1.682	1.682



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 26 JUN 2017, 12:17PM

Steel Beam

File = d:\text{ENERCALC Projects\text{2017-0610.ec6}} ENERCALC, INC. 1983-2017, Build:6.17.3.17, Ver:6.17.3.17} Licensee: RICHMOND HOFFMAYER

Lic. # : KW-06002886

**B17 - RIDGE SUPPORT** Description:

> port notation : Far left is #1 Values in KIPS

Vertical Reactions			Support notation : Far left is #1	values in NPS
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		



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 26 JUN 2017. 2:54PM

Steel Column

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

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**B17 SUPPORT** Description:

# 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

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

# **Applied Loads**

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**

Banding & Cheer Cheek Beaulte

Bending	& Shear Check Results							
PASS	/lax. Axial+Bending Stress Ratio =	0.5031	: 1	Maximum SERVIC	E Load Reaction	ns		
	Load Combination	+D+S		Top along X	-X		0.0 k	
	Location of max.above base	0.0	ft	Bottom alon			0.0 k	
	At maximum location values are			Top along Y	0		0.0 k	
	Pa : Axial	23.020	k	Bottom alon			0.0 k	
	Pn / Omega : Allowable	45.760	k		Ala cas sum o			
	Ma-x : Applied	0.0	k-ft	Maximum SERVIC	E Load Deflect	ions		
	Mn-x / Omega : Allowable	7.275		Along Y-Y for load comb	0.0 in	at	0.0ft	above base
	Ma-y: Applied	0.0	k-ft	TOT TOAU COTTID				
	Mn-y / Omega : Allowable	7.275	k-ft	Along X-X	0.0 in	at	O.Oft	above base
				for load combination :				
PASS	Maximum Shear Stress Ratio = Load Combination	0.0	: 1					
	Location of max.above base At maximum location values are	0.0	ft					(4)
	Va : Applied Vn / Omega : Allowable	0.0						

#### **Load Combination Results**

	Maximum Axial +	-Bending S	tress Ratios	Maximu	ım Shear Ra	atios
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				1	Note: Only	non-zero reactions are listed.

# Maximum Reactions

	Axial Reaction	X-X Axis	Reaction	k	Y-Y Axis	Reaction	Mx - End M	loments k-ft	My - End	Moments
Load Combination	@ Base	@ Base	@ Тор		@ Base	@ Тор	@ Base	@ Top	@ Base	@ Top
D Only	1.790									
+D+L	1.990									
	72, 202									

+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



Ycg

0.000 in

Project Title: Engineer: Project Descr:

VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 25 JUN 2017. 2:54PM

Steel Colum						- North					LC, INC. 1983-	2017, Bui		/er:6.17.3.17
Lic. # : KW-06002		41 994			Trans I de	1000	411	Long La		WEST S	Licensee :	RICHI	MOND HO	FFMAY
Description:	17 SUPPORT							10						
Maximum React	ions				U.					Not	e: Only no	n-zero	reactions	are listed
Load Combination	Ĺ	A	xial Reaction @ Base	'n	X-X Axis Re @ Base	eaction @ Top	k	Y-Y Axis @ Base	Reaction @ Top	Mx - Ei @ Bas	nd Moments e @ T	k-ft op	My - End @ Base	d Moments @ Top
Lr Only			1.69	5										
L Only			0.200	)										
S Only			21.230	)										
<b>Extreme Reaction</b>	ns													
			xial Reactio	n	X-X Axis Re		k	Y-Y Axis			d Moments	k-ft	7.55 C. L. S.	Moments
tem	Extreme Valu	ie	@ Base		@ Base	@ Тор		@ Base	@ Top	@ Bas	е @Т	op	@ Base	@ Top
Axial @ Base	Maximum		23.020											
Danatian V V Auto	Minimum		0.200											
Reaction, X-X Axis	Maximum		1.790											
5	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											
			1.790	I .										
	Minimum													
		THE RESERVE OF THE PERSON NAMED IN	THE RESERVE OF THE PERSON NAMED IN COLUMN											
Load Combination		THE RESERVE OF THE PERSON NAMED IN	ax. X-X Def	ection	Distan	ce		Max. Y-Y D	Annual Invitation and an arrange	Distanc	е			
Load Combination D Only		THE RESERVE OF THE PERSON NAMED IN	ax. X-X Def 0.0000	in	0.00	0 ft		0.000	in	0.000	ft			
Load Combination  D Only +D+L		THE RESERVE OF THE PERSON NAMED IN	0.0000 0.0000	in in	0.00 0.00	0 ft 0 ft		0.000	in in	0.000 0.000	ft ft			
Load Combination  D Only +D+L +D+Lr		THE RESERVE OF THE PERSON NAMED IN	0.0000 0.0000 0.0000 0.0000	in in in	0.00 0.00 0.00	0 ft 0 ft 0 ft		0.000 0.000 0.000	in in in	0.000 0.000 0.000	ft ft ft			
Load Combination  D Only +D+L +D+Lr +D+S		THE RESERVE OF THE PERSON NAMED IN	0.0000 0.0000 0.0000 0.0000 0.0000	in in in in	0.00 0.00 0.00 0.00	0 ft 0 ft 0 ft 0 ft		0.000 0.000 0.000 0.000	in in in in	0.000 0.000 0.000 0.000	ft ft ft ft			
Load Combination  D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L		THE RESERVE OF THE PERSON NAMED IN	0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	in in in in	0.00 0.00 0.00 0.00 0.00	0 ft 0 ft 0 ft 0 ft 0 ft		0.000 0.000 0.000 0.000	in in in in in	0.000 0.000 0.000 0.000 0.000	ft ft ft ft			
Load Combination  D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750S		THE RESERVE OF THE PERSON NAMED IN	0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	in in in in in	0.00 0.00 0.00 0.00 0.00 0.00	0 ft 0 ft 0 ft 0 ft 0 ft 0 ft		0.000 0.000 0.000 0.000 0.000	in in in in in	0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft			
Load Combination  D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D		THE RESERVE OF THE PERSON NAMED IN	0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	in in in in in	0.00 0.00 0.00 0.00 0.00 0.00	0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft		0.000 0.000 0.000 0.000 0.000 0.000	in in in in in in	0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft			
D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only		THE RESERVE OF THE PERSON NAMED IN	ax. X-X Def 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	in in in in in in	0.00 0.00 0.00 0.00 0.00 0.00 0.00	0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft		0.000 0.000 0.000 0.000 0.000 0.000	in in in in in in in	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft			
Load Combination  D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only		THE RESERVE OF THE PERSON NAMED IN	ax. X-X Def 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	in in in in in in in in	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft		0.000 0.000 0.000 0.000 0.000 0.000 0.000	in in in in in in in in	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft			
D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only S Only	tions for Loa	Ma	ax. X-X Def 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	in in in in in in	0.00 0.00 0.00 0.00 0.00 0.00 0.00	0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft		0.000 0.000 0.000 0.000 0.000 0.000	in in in in in in in in	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft			
Load Combination  D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only S Only  Steel Section Pro	tions for Loa	Pipe	ax. X-X Def 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	in in in in in in in	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft	N	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	in in in in in in in in	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft			
+D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only	tions for Loa	Ma	ax. X-X Def 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	in in in in in in in	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft	6.82	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	in in in in in in in in	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft		13.600 in^4	ı
Load Combination  D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750S +D.60D Lr Only L Only S Only  Steel Section Procepth	perties :	Pipe 4.500 in	ax. X-X Def 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	in in in in in in in in in	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft	6.82 3.03	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	in in in in in in in in	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft		13.600 in^4	ı
Load Combination  D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only S Only  Steel Section Procepth  Diameter	perties :	Pipe 4.500 in 4.500 in	ax. X-X Def 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	in i	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft	6.82	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	in in in in in in in in	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft		13.600 in^4	
Load Combination  D Only +D+L +D+L +D+S +D+0.750Lr+0.750S +D.60D Lr Only L Only S Only  Steel Section Pro Depth  Diameter Vall Thick	perties :	Pipe 4.500 in 4.500 in 0.237 in	ax. X-X Def 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	in R xx R xx Zx	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft	6.82 3.03	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	in in in in in in in in	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft		13.600 in^4	
Load Combination  D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only S Only  Steel Section Procepth  Diameter Vall Thick	perties :	Pipe 4.500 in 4.500 in	ax. X-X Def 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	in i	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft	6.82 3.03 1.510	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	in in in in in in in in	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft		13.600 in^4	ı
Load Combination  D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only S Only  Steel Section Pro	perties :	Pipe 4.500 in 4.500 in 0.237 in	ax. X-X Def 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	in R xx R xx Zx	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft 0 ft	6.82 3.03 1.510 4.050	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	in in in in in in in in	0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft ft		13.600 in^4	



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

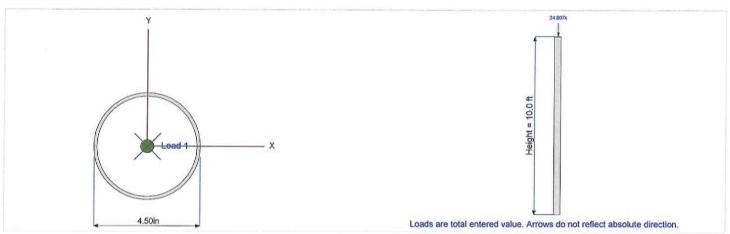
Steel Column

Printed: 26 JUN 2017, 2:54PM

File = d:ENERCALC Projects/2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:6.17.3.17, Ver.6.17.3.17 Ligensee : RICHMOND HOFFMAYER

Lic. # : KW-06002886 **B17 SUPPORT** 

Description:



# RICHMOND HOFFMAYER



PROJECT	VILLAGE NEST	JOB	2017-0610	
CLIENT				
ADDRESS	EDEN, UT			
DESIGN	HUUM			
<b>ENGINEER</b>	J.H.	DATE	6/14/2017	
	RICHMOND PE 44628 HOFFMAYER SE 5455	SHEET	QQ OF	

01	0	1414	INFORMATION	-
9	-	MAL	11410101 121120	-

TRY TYPE "S" PANELS

SEISMIL DESIGN CATEGORY D -FERIOPIL SPECIAL INSPECTION RELOVIRED FOR FASTENING AND ANCHORNING OF SHEAR WALLS.

TRY 7-4" TANEL CORE THICKNESS.

->MAX HEIGHT = 10-6"

WONIF = (16.8p+20L,+264,) 14 = 2099 PLF

ROST

WUNIES (14.30+40L) 8' = 217 PLF

WMX = 2099 + 217 = 2316 PLF

SEE TABLE 5 NEXT PAGE, CAR = 4326 PLF/

- MAX HEADEN SPAN W/O P.INT LOAD = 45'

WMANE 2099 PLF

SEE TABLE 14, PG #101, CASP = 2620PLF /

DBL TRAMPER, PREMIER INSUL-BERN 11

TABLE 4—UNIFORM TRANSVERSE LOADS FOR FACE SUPPORTED PREMIER TYPE L PANELS<sup>1,2,3</sup> (psf)

PANEL CORE						PANE	L SPAN				
THICKNESS (inches)	DEFLECTION	4 ft⁴	8 ft	10 ft	12 ft	14 ft	16 ft	18 ft	20 ft	22 ft	24 ft
	L/ <sub>360</sub>	98	45	32	24	16	11				
3 <sup>1</sup> / <sub>2</sub> <sup>5</sup>	L/ <sub>240</sub>	215	67	47	34	24	16				
	L/ <sub>180</sub>	298*	90	61	44	34	22				
	L/ <sub>360</sub>	241	128	57	41	33	25	20	15		
5 <sup>1</sup> / <sub>2</sub> <sup>6</sup>	L/ <sub>240</sub>	288*	182*	86	60	49	37	29	22		
	L/ <sub>180</sub>	288*	182*	112*	. 79	65	49	39	29		
	L/ <sub>360</sub>	241	168	80	65	54	42	33	24		
71/47	L/ <sub>240</sub>	288*	188*	126	99	81	61	49	34		
	<sup>L</sup> / <sub>180</sub>	288*	188*	133*	117*	105	80	62	44		
	<sup>L</sup> / <sub>360</sub>	274	188*	116	100	80	62	47	35	32	28
91/48	L/ <sub>240</sub>	326*	188*	147*	134*	120	92	70	52	46	41
	· <sup>L</sup> / <sub>180</sub>	326*	188*	147*	134*	121*	108*	93	68	61	53
	<sup>L</sup> / <sub>360</sub>	327*	188*	167*	140	116	90	75	57	47	36
11 <sup>1</sup> / <sub>4</sub> <sup>8</sup>	L/ <sub>240</sub>	327*	188*	167*	153*	132*	110*	97*	83*	69	53
	L/ <sub>180</sub>	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.

TABLE 5-ALLOWABLE UNIFORM AXIAL LOADS FOR PREMIER TYPE S PANELS (plf) 1.2.3.4

PANEL CORE	PANEL SPAN									
THICKNESS (inches)	8 ft	10 ft	12 ft	16 ft	20 ft	24 ft				
3 <sup>1</sup> / <sub>2</sub>	3,500	2,555	2,450	2,120						
5 <sup>1</sup> / <sub>2</sub>	4,250	4,040	3,375	3,920	2,815					
71/4	4,915	4,325	4,475	4,195	3,495	3,065				
9 <sup>1</sup> / <sub>4</sub>	4,200	4.200	4,200	4,200	3,389	3,247				
11 <sup>1</sup> / <sub>4</sub>	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.

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

PANEL CORE	PANEL SPAN										
THICKNESS (inches)	8 ft	10 ft	12 ft	16 ft	20 ft	24 ft					
31/2	4,725	3,905	3,095	2,350							
5 <sup>1</sup> / <sub>2</sub>	5,850	5,890	4,280	4,310	2,933						
71/4	6,850	6,110	5,555	5,180	4,835	4,080					
91/4	5,470	5,470	5,470	5,470	5,470	4,250					
11 <sup>1</sup> / <sub>4</sub>	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.

<sup>&</sup>lt;sup>1</sup>Floor panels must have a minimum <sup>3</sup>/<sub>4</sub>-inch-thick top skin or a minimum <sup>7</sup>/<sub>16</sub>-inch-thick top skin overlaid with minimum <sup>7</sup>/<sub>16</sub>-inch-thick finish flooring perpendicular to the panels.

<sup>&</sup>lt;sup>2</sup>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<sup>1</sup>/<sub>2</sub>-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.

<sup>&</sup>lt;sup>3</sup>Allowable loads with an asterisk, \*, indicates a capacity based on the average peak test load divided by 3.

<sup>&</sup>lt;sup>4</sup>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

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

<sup>&</sup>lt;sup>6</sup>5<sup>1</sup>/<sub>2</sub>-inch thick core panels must be limited to a maximum span of 14 feet when used in roof applications.

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

<sup>&</sup>lt;sup>8</sup>9<sup>1</sup>/<sub>4</sub> and 11<sup>1</sup>/<sub>4</sub>-inch thick core panels shall be limited to a maximum span of 20 feet when used in roof applications.

<sup>&</sup>lt;sup>1</sup>For the allowable axial load on the fire-resistance-rated assembly, see Section 4.2.6.

<sup>&</sup>lt;sup>2</sup>For combined loads; requirements in Section 4.1 must be applied.

<sup>&</sup>lt;sup>3</sup>The tabulated loads are uniform axial loads applied concentrically to the full thickness of the panels, including panel facings.

<sup>&</sup>lt;sup>4</sup>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.

<sup>&</sup>lt;sup>1</sup>For the allowable axial load on fire-resistance-rated assembly, see Section 4.2.7.

<sup>&</sup>lt;sup>2</sup>For combined loads; requirements in Section 4.1 must be applied.

<sup>&</sup>lt;sup>3</sup>The tabulated loads are uniform axial loads applied concentrically to the full thickness of the panels, including panel facings.

<sup>&</sup>lt;sup>4</sup>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 (ps	f) - End Zon	e (Zone 5)	for 100sf t	o 500 sf ef	fective win	d area			
Mean Roof	90 MPH				100 MPH			110 MPH			120 MPH	
Height (ft)	Exp B	Exp C	Exp D	Ехр В	Exp C	Exp D	Ехр В	Exp C	Exp D	Ехр В	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			

		Wal	I Loads (ps	f) - End Zor	ne (Zone 5)	for 100sf	to 500sf ef	fective win	d area			
Mean Roof	Mean Roof 130 MPH				140 MPH			150 MPH			170 MPH	
Height (ft)	Ехр В	Exp C	Exp D	Ехр В	Exp C	Exp D	Ехр В	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	d -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)

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

No. of	Deflection		Header Span (ft.)								
Trimmer Studs		9'	10'	11'	12'	13'	14'	15'	16'		
	L/480	700	630	573	458	360	288	234	193		
1	L/360	700	630	573	525	480	384	313	257		
	L/240	700	630	573	525	485	450	420	386		
	L/480	1085	791	594	458	360	288	234	193		
2	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 PLANTE W/ CAM

Top Plate

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

•	O. Maximum Anowabic Axial C	compression remit bee	(QB (103) — Type O Tai
	Top Plate	1.5" Minimum	/ 3" Minimum
	Configuration	Bearing Width /	Bosrina-Width
	Single 2x4 #2 or Better	2040	
	Hem-Fir Plate	2040	2450
	Single 2x4 #2 or Better		
	Hem-Fir Plate	4030	4678
	with 1-1/8 in. wide,	4030	4070
	1.3E Rim Board Cap Plate	CONTRACTOR OF THE PARTY OF THE	
	- 1 · · · · · ·		

Top plate secured to facings as required in Section 6.3

<sup>2</sup> Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.

<sup>3</sup> Concentrated loads shall be applied concentrically to the top of the panel.

<sup>4</sup> Tabulated values are based on the strong-axis of the facing material oriented parallel to the span direction.

2099 PLF × 16/12

= 2799 LOAD FACE



Top Plate

Cap Plate

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

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

Continuous over opening

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.

Tabulated road.

Tabulated values are based on the strong-axis of the facing material oriented perpendicular to the direction of header span.

<sup>3</sup> Minimum depth of facing above opening.

<sup>4</sup> 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<sup>1,2</sup>

Header	Deflection		Header	Span (ft)	
Depth <sup>3</sup> (in)	Limit⁴	4	6	8	10
	L/480	345	243	156	99
12	L/360	450	295	190	125
	L/240	630	382	236	153
	L/480	705	388	254	235
18	L/360	750	482	302	281
	L/240	750	482	302	281
	L/480	698	556	368	350
24 [	L/360	896	556	368	350
	L/240	896	556	368	350

SIPs wall panel spline minimum 6" from edge of opening

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.

<sup>2</sup> Tabulated values are based on the strong-axis of the facing material oriented perpendicular to the direction of header span.

<sup>3</sup> Minimum depth of facing above opening.

<sup>4</sup> Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code.

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PROJECT FELDSPAR LOWER JOB 2017-0610

CLIENT VILLAGE NEST

ADDRESS EDEN, UTAH

DESIGN HUUM

ENGINEER J.H. DATE 6/16/2017

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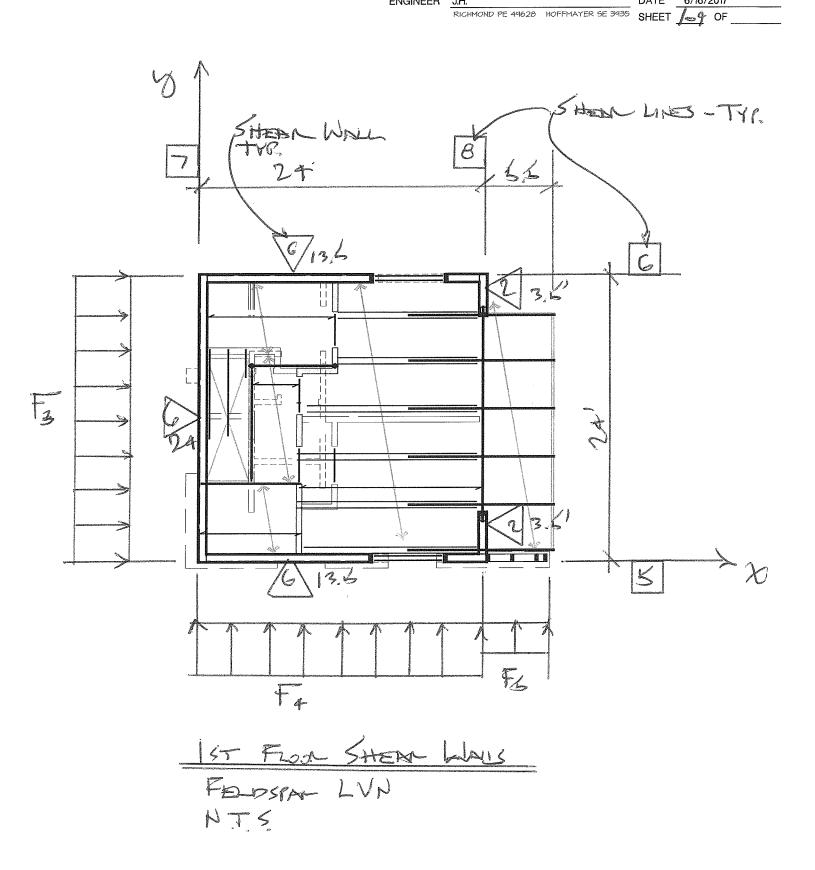
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PROJECT	FELDSPAR LOWER	JOB	2017-0610
CLIENT	VILLAGE NEST		
ADDRESS	EDEN, UTAH		
DESIGN	HUUM		
ENGINEER	J.H.	DATE	6/16/2017







PROJECT	FELDSPAR LOWER	JOB	2017-0610	
CLIENT	VILLAGE NEST			
ADDRESS	EDEN, UTAH			
DESIGN	HUUM			
ENGINEER	J.H.	DATE	6/16/2017	
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4425 CANOGA AVENUE WOODLAND HILLS CA 91364 PHONE (818) 347-7008 FAX (818) 883-8869 Info@RichmondHoffmayer.com





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## **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  $S_{\text{s}}$ ) and 1.3 (to obtain  $S_{\text{i}}$ ). 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

$\overline{V}_{s}$	$\overline{N}$ or $\overline{N}_{ch}$	- s <sub>u</sub>
>5,000 ft/s	N/A	N/A
2,500 to 5,000 ft/s	N/A	N/A
1,200 to 2,500 ft/s	>50	>2,000 psf
600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
<600 ft/s	<15	<1,000 psf
characteristics: • Plasticity inde • Moisture cont	x <i>PI</i> > 20, ent <i>w</i> ≥ 409	∕₀, and
	>5,000 ft/s 2,500 to 5,000 ft/s 1,200 to 2,500 ft/s 600 to 1,200 ft/s <600 ft/s Any profile with more characteristics: • Plasticity inde • Moisture cont	>5,000 ft/s N/A 2,500 to 5,000 ft/s N/A 1,200 to 2,500 ft/s >50 600 to 1,200 ft/s 15 to 50  <600 ft/s <15 Any profile with more than 10 ft

F. Soils requiring site response analysis in accordance with Section 21.1

See Section 20.3.1

For SI:  $1ft/s = 0.3048 \text{ m/s} 1lb/ft^2 = 0.0479 \text{ kN/m}^2$ 

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<sub>a</sub>

Site Class	Mapped Spectral Response Acceleration at Short Period											
	S <sub>s</sub> ≤ 0.25	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	S <sub>s</sub> ≥ 1.25							
А	0.8	0.8	0.8	0.8	0.8							
В	1.0	1.0	1.0	1.0	1.0							
С	1.2	1.2	1.1	1.0	1.0							
D	1.6	1.4	1.2	1.1	1.0							
Е	2.5	1.7	1.2	0.9	0.9							
F		See Se	ction 11.4.7 of	ASCE 7								

Note: Use straight-line interpolation for intermediate values of  $\ensuremath{S_{\scriptscriptstyle S}}$ 

For Site Class = D and  $S_s$  = 0.806 g,  $F_a$  = 1.178

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F<sub>v</sub>

Site Class	Мар	ped Spectral R	ed Spectral Response Acceleration at 1-s Period										
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_i = 0.30$	$S_1 = 0.40$	$S_i \geq 0.50$								
А	0.8	0.8	0.8	0.8	0.8								
В	1.0	1.0	1.0	1.0	1.0								
С	1.7	1.6	1.5	1.4	1.3								
D	2.4	2.0	1.8	1.6	1.5								
Е	3.5	3.2	2.8	2.4	2.4								
F		See Se	See Section 11.4.7 of ASCE 7										

Note: Use straight-line interpolation for intermediate values of S<sub>1</sub>

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

**Equation (16-37):** 

 $S_{MS} = F_a S_S = 1.178 \times 0.806 = 0.949 g$ 

Equation (16-38):

 $S_{M1} = F_{\nu}S_{1} = 1.866 \times 0.267 = 0.498 g$ 

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

MAKINE OF C		RISK CATEGORY										
VALUE OF S <sub>DS</sub>	I or II	III	IV									
S <sub>ps</sub> < 0.167g	Α	А	Α									
0.167g ≤ S <sub>ps</sub> < 0.33g	В	В	С									
0.33g ≤ S <sub>DS</sub> < 0.50g	С	С	D									
0.50g ≤ S <sub>DS</sub>	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 C	RISK CATEGORY										
VALUE OF S <sub>D1</sub>	I or II	III	IV								
S <sub>D1</sub> < 0.067g	А	A	А								
0.067g ≤ S <sub>D1</sub> < 0.133g	В	В	С								
0.133g ≤ S <sub>D1</sub> < 0.20g	С	С	D								
0.20g ≤ S <sub>D1</sub>	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  $\equiv$  "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				The contract of	
		GOOGLE EARTH LATITUDE	N	=	41,360	0	
		LONGTITUDE	w	=	-111.740		
		USGS NSHMP - 2012 IBC		-21	0.000		
		BASED ON SITE CLASS B, Fa = 1.0 AND Fv = 1.0	Ss S1	=	0.806 0.267		
16CBC /	ASCE	E 7-10					
03.1.5		THQUAKE DESIGN DATA	w.				
	1	SEISMIC IMPORTANCE FACTOR AND OCCUPANCY OCCUPANCY	1	=	1.00 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	
	3	SITE CLASS	10111023250		D		IBC DEFAULT
		SITE COEFFICIENTS	1.00 Fv	-	1.18 1.87	_	TABLE 11.4-1 TABLE 11.4-2
	4	SPECTRAL RESPONSE COEFFICIENTS	SMS		0.949		
		of Editorial Grant Gold Tolding	SM1	= -	0.498		
			SDS SD1	= =	0.633 0.332	= 2/3 SMS = 2/3 SM1	ASCE 7-10 (11.4-3) ASCE 7-10 (11.4-4)
			301			2/3 5/4/1	
		SEISMIC DESIGN CATEGORY		-	D		IBC2012 TABLES 1613.5.6(1) & 1613.5.6(2)
AIN BUILD		DIRECTION - X BASIC SEISMIC-FORCE-RESISTING SYSTEM					
		A. BEARING WALL SYSTEMS					
		A-LIGHT-FRAMED WALLS SHEATHED WITH WOOD P	R		6.50		ASCE 7-10 TABLE 12.2-1
			Ωο	-	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	7	
	8	SEISMIC RESPONSE COEFFICIENT	Cs	=	0.097	= SDS / (R / I)	ASCE 7-10 (12.8-2)
			Cu	=	Cu Ta 1.40		ASCE 7-10 TABLE 12.8-1
			hn	=	10.00	ft	NOOL PTO PROLET LEGY
			Ct	=	0.02	(6)	ASCE 7-10 TABLE 12.8-2
			×	=	0.75		ASCE 7-10 TABLE 12.8-2
			Та	=	0.112	= Ct hn^x	ASCE 7-10 (12.8-7)
			T	=	0.157	S	ASCE 7-10 12.8.2
			TL Cs'	=	8.00 0.325	MAX	ASCE 7-10 FIG.22-16 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 US		0.097	10.1	Cs OVERIDE 0.000
	9	RESPONSE MODIFICATION FACTOR	R	= _	6.50		S TV SV V V V SV SV SV V SV V SV SV SV SV S
	10	ANALYSIS PROCEDURE USED	V	= E	EQUIVALEI 0.0973	NT LATERAL FOI W	RCE PROCEDURE  X-DIRECTION BASE SHEAR FORCE
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AIN BOILD		BASIC SEISMIC-FORCE-RESISTING SYSTEM					
		A. BEARING WALL SYSTEMS A-LIGHT-FRAMED WALLS SHEATHED WITH WOOD PA	ANELS				
		THE STATE OF THE STATE OF THE WOOD PA	R	-	6.50		ASCE 7-10 TABLE 12.2-1
			Ωο	= -	3.00		ASCE 7-10 TABLE 12.2-1
			Cd	= _	4.00	_	ASCE 7-10 TABLE 12.2-1
		DESIGN BASE SHEAR SEISMIC RESPONSE COEFFICIENT	V Cs	=	Cs W 0.097	= SDS / (R / I)	ASCE 7-10 (12.8-2)
	2		T	=	Cu Ta		
			Cu	=	1.40		ASCE 7-10 TABLE 12.8-1
			hn Ct	=	0.02	ft	ASCE 7-10 TABLE 12.8-2
			×	=	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)
					0.010	MIN'	ASCE 7-10 (12.8-6) Cs OVERIDE 0.000
	9	RESPONSE MODIFICATION FACTOR	Cs USE R		6.50	•	0.000 0.000
		RESPONSE MODIFICATION FACTOR ANALYSIS PROCEDURE USED		= =	6.50	- ĪT LATERAL FOR W	RCE PROCEDURE  Y-DIRECTION BASE SHEAR FORCE



AREA WEIGHTS - 2ND LEVEL:

ROOF

9.00 ft WALL HT.

WEIGHT TABULATIONS: 2X = 62496.90 lb 2Y = 58787.95 lb

WEIGHT	VEIGHT TYPES:				
A	METAL ROOF	15.76 psf	<u>N</u>	EXTERIOR	15.23 psf
Ω.	OPENED BALCONY	7.00 psf	>	INTERIOR	8.85 psf
ပ	2nd FLOOR	14.25 psf	3	ι	0.00 pst
۵	DRIVEWAY LOADS	85.38 psf	×		0.00 psf
ш	20% SNOW LOAD FOR SEISM	52.80 psf	<u>&gt;</u>	1	0:00 pst
ᄔ	GARAGE LOADS	63.50 psf	Z	t	0.00 psf

OTAL WEIGHT	17194.50 lb	45302.40 lb	0.00	q1 00'0	62496.90 lb	13485.55 lb	45302.40 lb	0.00 lb	0.00 lb dl 26.787.85	di 00.0	0.00 lb	0.00	0.00 lb	0.00 lb	0.00 lb	0.00 lb	0.00 lb	0.00 lb	0.00 lb	0.00 lb	0.00 lb	0.00 lb	qi 00:0	0.00 lb	0.00 lb	0.00 lb	0.00 lb	0.00 lb
WIDTH TC	24,00 ft =	26.00 ft =	00.00	00.00	Western Park	24.00 ft =	33.00	= 0.00 ft =	0.00	= 1 00°0	<b>■ #</b> 00'00 ;	= # 00.00	= # 00.00 ;		: 0.00 ft =	= 14 00'00 ti	0.00 €	= 0.00 ft =		0.00 ft ==	= 1000 0	= 11 00'00 11	= 14 00.00 ;		= 11 00.00	= 1000 O	x 0.00 ft =	0.00 ft =
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NUMBER	1.00	0.00	0.00	00.00		0.00	00'0	00.0	00.00	0.00	0.00	0.00	0.00		00.00	00'0	0.00	0.00		00.00	00.0	0.00	00.0		0.00	00'0	00.00	0.00
HEIGHT	5,00 ft x	0.00 ft x	0.00 ft x	0.00 ft x		0.00 ft x	0.00 ft x	0.00 ft x		0.00 ft ×	0.00 ft x				0.00 ft ×	0.00 ft x	0.00 ft x	0.00 ft x		0.00 ft x	0.00 ft x	0.00 ft x	0,00 ft x		0.00 ft ×	0.00 ft x	0.00 ft x	0.00 ft ×
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UNIT WEIGH				0.00		0.00			0.00	0.00	0.00		0.00		0.00	0.00	0.00	0.00		0.00			0.00		0.00			0.00
	+	+	+	+		+	+	+	+	+	+	+	+		+	+	+	+		+	+	+	+		+	+	+	+
NUMBER	2,00	00'0	0.00	0.00		2.00	00.00	00.00	0.00	00.00	00.00	00.00	00.00		00.00	00.00	0.00	0.00		0.00	0.00	00'0	00.00		00'0	00:00	00.00	0.00
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HEIGH	5,00 ft	00.00	0.00	0.00		5,00	0.00	0.00	00'0	00.00	0.00	00'0	00'0		00.00	0.00	0.00	00'0		0.00	0.00	0.00	00'0		00'0	0.00	0.00	0.00
	×	×	×	×			×	×	×	×	×	×	×		×	×	×	×		×	×	×	×		×	×	×	×
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UNIT WEIGHT	15.23 psf	0.00 psf	0.00 psf	0.00 psf		15.23 psf	0.00 psf	0.00 psf	0.00 psf	0.00 psf	0.00 psf		0.00 psf		0.00 psf	0.00 psf	0.00 psf	0.00 psf		0.00 psf	0.00 psf	0.00 psf	0.00 psf		0.00 psf	0.00 psf	0.00 psf	0.00 psf
N)	+	+	+	+		+	+	+	+	+	+	+	+		+	+	+	+		+	+	+	+		+	+	+	+
3TH	33.00 ft	33,00 ft	0.00	0.00		26.00 ft	26.00 ft	0.00	0,00 #	0.00 ft			0.00		0.00	0.00		0.00		0.00	0.00	0.00	000		00'00	0.00	0.00	0,00 ft
LENGTH		ee ×						×			×				×		×			×		×			×		×	
TYPE																							ū					
UNIT WEIGHT T	15.76 psf A		0.00 psf -	0,00 psf		15.76 psf A	psf	0.00 psf -	0.00 psf -	0.00 psf -	0.00 psf -		0.00 psf -		0.00 psf -	0.00 psf -		0.00 psf			0.00 psf -		0.00 psf -				0.00 psf -	0.00 psf -
Š		+	+	+			+	+	+		+	+	+			+	+	+			+	+	+			+	+	+
W# F#	2-1 1 2X					2-2 2 2 2Y				2					2					2					2			



**AREA WEIGHTS - 1ST LEVEL:** 

2ND FLOOR

9.00 ft WALL HT.

1X = 18564.00 lb 1Y = 18564.00 lb WEIGHT TABULATIONS:

WEIGHT TYPES:	<u> </u>				
A	METAL ROOF	15.76 psf	n n	EXTERIOR	15.23 psf
В	OPENED BALCONY	7.00 psf	>	INTERIOR	8.85 psf
ر د	2nd FLOOR	14.25 psf	W	,	0.00 psf
۵	DRIVEWAY LOADS	85.38 psf	×	1	0.00 psf
ш	20% SNOW LOAD FOR SEISM	52.80 psf	>	,	0.00 psf
ᄔ	GARAGE LOADS	63.50 psf	Z		0.00 psf

TOTAL WEIGHT	17640.00 lb	924 00 lb	01 00 U	0.00 dl 00.0	18564.00 lb	17640 00 lb	a 00 0	0.00 di 00.00	0.00 lb	17640.00 lb	924.00 lb	4 00 0	2 CO C	dl 00:0	924.00 lb	4 00 0	2 00.0 2 4	2 CO CO	al 00:0	dl 00.0	4	2 00 0 4 00 0	4 00 0	0.00 lb	0.00 lb
orange contains	24,00 ft ==	24 00 # ==	4	: #		24.00 ff =	: =	: ==	0.00 ft =		5,50 ft =		: #			# 000	= #	=	: #		0.00	: #	=		
WIDTH	× 24.	× 24				× 24					×	×				>					×	; c			
TRIB. WEIGHT	735.00 plf	38.50 plf	0.00 plf	0.00 plf	•	735 00 plf			0.00 plf		168.00 plf	0.00 plf			<u>-</u>	A 00.0			0.00 plf	-	0.00 pf			0.00 plf	
1	=	11	11	н		jI	11	ŧI	H		И	н	H	H		11	11	Н	IJ		n	И	11	H	
NUMBER	1.00	0.00	00.0	00'0		1 00	00.00	00.00	00.00		0.00	00.00	00.0	00.00		0	000	0.00	00.00		0.00	00.0	0.00	00.00	
HEIGHT	10,00 ft x	0.00 ft ×		0.00 ft ×		10.00 ft ×	0.00 ft x	0.00 ft x	0.00 ft ×		0.00 ft x	0.00 ft x	0.00 ft x			V # 00 0	× # 000	0.00 ft x			0.00 ft ×	0.00 ft ×	0.00 ft x	0.00 ft ×	
1	×	×	×	×		×	×	×	×		×	×	×	×		>	×	×	×		×	×	×	×	
TYPE	\ \	ŧ	,	,		>	1	,	,		,	1	,	ı		,		1	1		,	ı	,	1	
UNIT WEIGHT	8.85 psf	0.00 pst	0.00 psf	0.00 psf		8.85 psf	0.00 psf		0.00 psf		0.00 psf	0.00 psf				0.00 pst					0.00 psf			0.00 psf	
	+	+	+	+		+	+	+	+		+	+	+	+		+	+	+	+		+	+	+	+	
NUMBER	2.00	00.0	00.00	0.00		2.00	00.00	00.0	0.00		0.00	00.00	00.0	00.00		0.00	00.00	00.00	0.00		00.00	00.00	0.00	00'0	
	×	×	×	×		×	×	×	×		×	×	×	×		×	×	×	×		×	×	×	×	
HEIGHT	10.00 ft	0.00	0.00	0.00 #		10.00 #	0.00	0.00	0.00 ft		0.00	0.00	0.00	0.00		0.00	0.00	0.00	0.00		00.00	0.00	0.00	0.00 ft	
At Constant	×	×	×	×		×	×	×	×		×	×	×	×		×	×	×	×		×	×	×	×	
T TYPE	n	,	1	,		>	,	ŧ	ŧ			1	1			,	ī	1	,		•	3		ı	
UNIT WEIGHT	15.23 psf	0.00 psf	0.00 psf	0.00 psf		15.23 psf	0.00 psf	0.00 psf	0.00 psf		0.00 psf	0.00 psf	0.00 psf	0.00 psf		0.00 psf	0.00 psf	0.00 psf	0.00 psf		0.00 psf	0.00 psf	0.00 psf	0.00 psf	
	+	+	+	+		+	+	+	+		+	+	+	+		+	+	+	+		+	+	+	+	
LENGTH	24.00 ft	5.50 #	00'00	0.00		24.00 #	0.00	0.00	0.00 #		24.00 ft	00.00	0.00	0.00		00.00	0.00	0.00	0.00		0.00	0,00 #	0.00	00.00	
	×	×	×	×		×	×	×	×		×	×	×	×		×	×	×	×		×	×	×	×	
UNIT WEIGHT TYPE	14.25 psf C	7.00 psf B	0.00 psf -	0.00 psf -		14.25 psf C	0.00 psf -	0.00 psf -	0.00 psf -		7.00 psf B	0.00 psf -	0.00 psf -	0.00 psf -		0.00 psf -	0.00 psf -	0.00 psf -	0.00 psf -		0.00 psf -	0.00 psf -	0.00 psf -	0.00 psf -	
D	×	+	+	+		7	+	+	+		1,≺	+	+	+			+	+	+		ı	+	+	+	
W# F#	1-3					1-4					1-5 5														



## **EQUIVALENT LATERAL FORCE PROCEDURE:**

BASE SHEAR		V	=	Cs W	ASCE 7-10 (12.8-1)	
	ILDING DEAD WEIGHT - X DIRECTION ION SEISMIC RESPONSE COEFICIENT	Wx Csx	=	81060.90 lb 0.0973	SEE PREVIOUS PAGES Rx =	6.50
STRENGT	H LEVEL BASE SHEAR FORCE - X	Vx	=_	7891.14 lb		
TOTAL BU	ILDING DEAD WEIGHT - Y DIRECTION	Wy	=	77351.95 lb		
Y-DIRECT	ON SEISMIC RESPONSE COEFICIENT	Csy	=	0.0973	SEE PREVIOUS PAGES Ry =	6.50
STRENGT	H LEVEL BASE SHEAR FORCE - Y	Vy	=	7530.08 lb		

0.2648 s

ASCE 7-10

12.8.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES

PER EACH ORTHOGONAL DIRECTION x & y

X-DIRECTION BUILDING PERIOD

F = CvV  $Cv = wh^k$   $\Sigma wi hi^k$ 

ASCE 7-10 (12.8-11) ASCE 7-10 (12.8-12)

SEE PREVIOUS PAGES

X-DINE	OTION BOILDING! ENIOD	k	= 1.0		ASCE 7-	10 12.8.3
LEVEL	wx	hx	hx ^ k	wx hx ^ k	Cvx	STREN SHEAF FORCI LEVEL
_	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	

T

STRENGTH		
SHEAR		DISTRIBUTION
FORCE PER		FACTOR
LEVEL Fx	Wx Csx	Fx/(WxCsx)
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 k = 1.00 SEE PREVIOIUS PAGES ASCE 7-10 12.8.3

LEVEL	w	hy	hxy^k	wy hy ^ k	Cvv
-	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		DISTRIBUTION
FORCE PER		FACTOR
LEVEL Fy	Wy Csy	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		





## **SEISMIC FORCES:**

4TH LEVEL FORCES	-	DICT F	ACTOD.			COLONI OLICAD E	000E ( 0TD	ENGTO EVE		DICT	CONDUCTION ON CO.	ine I n ovenin
F#	TOTAL TRIB. WEIGHT	SU REAS TOURS SUPPLIES AND EAST FOR SUPPLIES	<u>casamannemisent</u>	Cs	CHARGE CONTRACTOR OF THE	ESIGN SHEAR F			-)	ופוע	RIBUTION OVER	IDE R OVERID
		x 0.00	X	0.0000	=	0.00 plf	R=	0.00				
		x 0.00	х	0.0000	=	0.00 plf	R =	0.00		-		
	0.00 plf		X	0.0000	=	0.00 plf	R =	0.00				
* "	0.00 plf		X	0.0000	=	0.00 plf	R =	0.00				
		x 0.00	х	0.0000	=	0.00 plf	R =	0.00				
		x 0.00	X	0.0000	=	0.00 plf	R =	0.00				
		x 0.00	x	0.0000	=	0.00 plf	R =	0.00			······	
		x 0.00	X	0.0000	=	0.00 plf	R =	0.00		-		
		x 0.00 x 0.00	X	0.0000	=	0.00 plf	R = R =	0.00 0.00				
	•		X	0.0000	=	0.00 plf	R=	0.00				
	0.00 plf	x 0.00 x 0.00	X	0.0000 0.0000	=	0.00 plf 0.00 plf	R=	0.00				
	0.00 plf		X	0.0000	=	0.00 plf	R=	0.00				
	0.00 plf		x x	0.0000	=	0.00 plf	R=	0.00				
	0.00 pii	X 0.00	^	0.0000	_	0.00 pii	Ν-	0.00		L.—	· · · · · ·	
3RD LEVEL FORCES	-											
F#	TOTAL TRIB. WEIGHT	DIST. FA	CTOR	Cs	DI	ESIGN SHEAR FO	ORCE (STR	NGTH LEVE	_ }	DIST	RIBUTION OVER	IDE R OVERID
- +	0.00 plf	x 0.00	х	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	0.0000	=	0.00 plf	R =	0.00				
		x 0.00	х	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	х	0.0000	=	0.00 plf	R =	0.00				
	0.00 plf		x	0.0000	=	0.00 plf	R =	0.00				
	0.00 plf		X	0.0000	=	0.00 plf	R =	0.00				
	0.00 plf		X	0.0000	=	0.00 plf	R =	0.00				
	0.00 plf		X	0.0000	=	0.00 plf	R =	0.00				
	0.00 plf		X	0.0000	=	0.00 plf	R =	0.00		L		
	0.00 plf		х	0.0000	=	0.00 plf	R =	0.00				
	0.00 plf		х	0.0000	=	0.00 plf	R =	0.00		L		
	0.00 plf	x 0.00	х	0.0000	=	0.00 plf	R =	0.00				
2ND LEVEL FORCES	·	x 0.00	х	0.0000	=	0.00 plf	R =	0.00		<u> </u>		
2ND LEVEL FORCES	ROOF								7	L	RIBUTION OVER	I IDE I R OVERIDI
F#	ROOF TOTAL TRIB. WEIGHT	DIST. FA	CTOR	Cs	DE	ESIGN SHEAR FO	DRCE (STRI	ENGTH LEVEL	.)	DIST	RIBUTION OVER	IDE R OVERIDI
F# 1 2X	ROOF TOTAL TRIB. WEIGHT 2403.73 plf	DIST. FA x 1.13	CTOR x	Cs 0.0973	DE =	ESIGN SHEAR FO	DRCE (STRI	ENGTH LEVEL 6.50	- )	DIST	RIBUTION OVER	IDE R OVERIDI
F#	ROOF TOTAL TRIB. WEIGHT 2403.73 plf 1781.45 plf	DIST. FA x 1.13 x 1.14	CTOR × ×	Cs 0.0973 0.0973	DE = =	ESIGN SHEAR FO 264.26 plf 197.07 plf	ORCE (STRI R = R =	ENGTH LEVEL 6.50 6.50		DIST	RIBUTION OVER	IDE R OVERID
F# 1 2X 2 2Y	ROOF TOTAL TRIB. WEIGHT 2403.73 pif 1781.45 pif 0.00 pif	DIST. FA × 1.13 × 1.14 × 0.00	CTOR × × ×	Cs 0.0973 0.0973 0.0000	DE = = =	ESIGN SHEAR FO 264.26 plf 197.07 plf 0.00 plf	DRCE ( STRI R = R = R =	ENGTH LEVEL 6.50 6.50 0.00	ann parameter and an annual service and an a	DIST	RIBUTION OVER	IDE R OVERID
F# 1 2X 2 2Y	ROOF TOTAL TRIB. WEIGHT 2403.73 plf 1781.45 plf 0.00 plf 0.00 plf	DIST. FA x 1.13 x 1.14 x 0.00 x 0.00	CTOR X X X X	Cs 0.0973 0.0973 0.0000 0.0000	DE = = =	ESIGN SHEAR FO 264.26 plf 197.07 plf 0.00 plf 0.00 plf	DRCE ( STRI R = R = R = R = R =	ENGTH LEVEL 6.50 6.50 0.00 0.00		DIST	RIBUTION OVER	IDE R OVERID
F# 1 2X 2 2Y	ROOF TOTAL TRIB. WEIGHT 2403.73 pif 1781.45 pif 0.00 pif 0.00 pif 0.00 pif	DIST. FA x 1.13 x 1.14 x 0.00 x 0.00 x 0.00	CTOR X X X X X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000	DE = = = = =	ESIGN SHEAR FO 264.26 plf 197.07 plf 0.00 plf 0.00 plf 0.00 plf	DRCE ( STRI R = R = R = R = R = R =	ENGTH LEVEL 6.50 6.50 0.00 0.00 0.00		DIST	RIBUTION OVER	IDE R OVERID
F# 1 2X 2 2Y	ROOF TOTAL TRIB. WEIGHT 2403.73 plf 1781.45 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf	DIST. FA x 1.13 x 1.14 x 0.00 x 0.00 x 0.00 x 0.00 x 0.00	X X X X X X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000	DE = = = = = = = = = = = = = = = = = = =	264.26 plf 264.26 plf 197.07 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf	DRCE ( STRI R = R = R = R = R = R =	ENGTH LEVEL 6.50 6.50 0.00 0.00 0.00 0.00		DIST	RIBUTION OVER	IDE R ÖVERID
F# 1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 plf  1781.45 plf  0.00 plf  0.00 plf  0.00 plf  0.00 plf  0.00 plf	DIST. FA x 1.13 x 1.14 x 0.00	X X X X X X X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000		264.26 plf 197.07 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf	DRCE ( STRI R = R = R = R = R = R = R =	6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	) ]	DIST	RIBUTION OVER	IDE R ÖVERID
F# 1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 plf  1781.45 plf  0.00 plf  0.00 plf  0.00 plf  0.00 plf  0.00 plf	DIST. FA x 1.13 x 1.14 x 0.00	CTOR  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DE = = = = = = = = = = = = = = = = = = =	ESIGN SHEAR FC 264.26 plf 197.07 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf	DRCE ( STRI R = R = R = R = R = R = R = R =	ENGTH LEVEL 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	)	DIST	RIBUTION OVER	IDE R OVERID
F# 1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 plf  1781.45 plf  0.00 plf  0.00 plf  0.00 plf  0.00 plf  0.00 plf	DIST. FA x 1.13 x 1.14 x 0.00	X X X X X X X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000	DE = = = = = = = = = = = = = = = = = = =	264.26 plf 197.07 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf	DRCE ( STRI R = R = R = R = R = R = R =	6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	)	DIST	RIBUTION OVER	IDE R OVERID
F# 1 2X 2 2Y	ROOF TOTAL TRIB. WEIGHT 2403.73 plf 1781.45 plf 0.00 plf	DIST. FA x 1.13 x 1.14 x 0.00	CTOR  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DE = = = = = = = = = = = = = = = = = = =	264.26 plf 197.07 plf 0.00 plf	DRCE ( STRI R = R = R = R = R = R = R = R = R =	ENGTH LEVEL 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00		DIST	RIBUTION OVER	IDE R ÖVERID
F# 1 2X 2 2Y	ROOF TOTAL TRIB. WEIGHT 2403.73 plf 1781.45 plf 0.00 plf	DIST. FA x 1.13 x 1.14 x 0.00	CTOR  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DE = = = = = = = = = = = = = = = = = = =	ESIGN SHEAR FC 264.26 plf 197.07 plf 0.00 plf	DRCE ( STRI R = R = R = R = R = R = R = R = R =	6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	.)	DIST	RIBUTION OVER	IDE R ÖVERID
F# 1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 pif  1781.45 pif  0.00 pif	DIST. FA x 1.13 x 1.14 x 0.00	CTOR  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DE = = = = = = = = = = = = = = = = = = =	264.26 plf 197.07 plf 0.00 plf	PRCE ( STRI R = R = R = R = R = R = R = R = R = R =	6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	.)	DIST	RIBUTION OVER	IDE R ÖVERIDI
F# 1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 plf  1781.45 plf  0.00 plf	DIST. FA x 1.13 x 1.14 x 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DE = = = = = = = = = = = = = = = = = = =	264.26 plf 197.07 plf 0.00 plf	DRCE ( STRI R = R = R = R = R = R = R = R = R = R =	ENGTH LEVEL 6.50 0.00 0.00 0.00 0.00 0.00 0.00 0.00	.)	DIST	RIBUTION OVER	IDE R OVERIDI
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 pif  1781.45 pif  0.00 pif	DIST. FA x 1.13 x 1.14 x 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DE = = = = = = = = = = = = = = = = = = =	ESIGN SHEAR FC 264.26 plf 197.07 plf 0.00 plf	DRCE (STRI R = R = R = R = R = R = R = R = R = R =	ENGTH LEVEL 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00	)	DIST	RIBUTION OVER	IDE R OVERIDI
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 plf  1781.45 plf  0.00 plf	DIST. FA  x 1.13  x 1.14  x 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	D8	ESIGN SHEAR FC 264.26 plf 197.07 plf 0.00 plf	DRCE (STRI R = R = R = R = R = R = R = R = R = R =	ENGTH LEVEL 6.50 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00				
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 plf  1781.45 plf  0.00 plf  200 plf  300 plf  400 plf  500 plf  600 plf	DIST. FA  x 1.13  x 1.14  x 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DE	264.26 plf 197.07 plf 0.00 plf	DRCE ( STRI	ENGTH LEVEL 6.50 0.00 0.00 0.00 0.00 0.00 0.00 0.00			RIBUTION OVER	
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 plf  1781.45 plf  0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 200 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf TOTAL TRIB. WEIGHT 773.50 plf	DIST. FA  x 1.13  x 1.14  x 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DE = = = = = = = = = = = = = = = = = = =	264.26 plf 264.26 plf 197.07 plf 0.00 plf	PRCE (STRE	6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00				
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 plf  1781.45 plf  0.00 plf  200 plf  0.00 plf	DIST. FA  × 1.13  × 1.14  × 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DE = = = = = = = = = = = = = = = = = = =	264.26 plf 197.07 plf 0.00 plf 42.52 plf 40.65 plf	PRCE (STRI	6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00				
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 pif 1781.45 pif 0.00 pif TOTAL TRIB. WEIGHT  773.50 pif 735.00 pif	DIST. FA  x 1.13  x 1.14  x 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DE DE	ESIGN SHEAR FC 264.26 plf 197.07 plf 0.00 plf	DRCE (STRI	ENGTH LEVEL 6.50 0.00 0.00 0.00 0.00 0.00 0.00 0.00				
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 plf  1781.45 plf  0.00 plf  200 plf  0.00 plf	DIST. FA  x 1.13  x 1.14  x 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DE = = = = = = = = = = = = = = = = = = =	ESIGN SHEAR FC 264.26 plf 197.07 plf 0.00 plf	DRCE (STRI	ENGTH LEVEL 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00				
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 plf  1781.45 plf  0.00 plf	DIST. FA  x 1.13  x 1.14  x 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DE	264.26 plf 197.07 plf 0.00 plf	DRCE (STRE	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0				
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 plf  1781.45 plf  0.00 plf  200 plf  0.00 plf  735.00 plf  168.00 plf  0.00 plf  0.00 plf  0.00 plf	DIST. FA  × 1.13  × 1.14  × 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000	DE DE	264.26 plf 197.07 plf 0.00 plf	PRCE (STRI	ENGTH LEVEL  6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.0				
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 pif  1781.45 pif  0.00 pif  200 pif  0.00 pif  168.00 pif  0.00 pif  0.00 pif  0.00 pif  0.00 pif  0.00 pif	DIST. FA  × 1.13  × 1.14  × 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000	DE	264.26 plf 197.07 plf 0.00 plf 42.52 plf 40.65 plf 9.29 plf 0.00 plf 0.00 plf 0.00 plf	DRCE (STRI	ENGTH LEVEL 6.50 0.00 0.00 0.00 0.00 0.00 0.00 0.00				
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 plf  1781.45 plf  0.00 plf  735.00 plf  168.00 plf  0.00 plf	DIST. FA  x 1.13  x 1.14  x 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000	DE = = = = = = = = = = = = = = = = = = =	ESIGN SHEAR FC 264.26 plf 197.07 plf 0.00 plf 42.52 plf 40.65 plf 9.29 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf	DRCE (STRE	ENGTH LEVEL 6.50 0.00 0.00 0.00 0.00 0.00 0.00 0.00				
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 plf  1781.45 plf  0.00 plf	DIST. FA  x 1.13  x 1.14  x 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000	DE	264.26 plf 197.07 plf 0.00 plf	DRCE (STRE	NGTH LEVEL 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00				
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 plf  1781.45 plf  0.00 plf	DIST. FA  × 1.13  × 1.14  × 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000	DE DE	264.26 plf 197.07 plf 0.00 plf	PRCE (STRI	ENGTH LEVEL 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00				
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 pif  1781.45 pif  0.00 pif  168.00 pif  0.00 pif	DIST. FA  × 1.13  × 1.14  × 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000	DE	264.26 plf 197.07 plf 0.00 plf 42.52 plf 40.65 plf 9.29 plf 0.00 plf	DRCE (STRE	ENGTH LEVEL 6.50 0.00 0.00 0.00 0.00 0.00 0.00 0.00				
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 plf  1781.45 plf  0.00 plf  735.00 plf  168.00 plf  0.00 plf	DIST. FA  x 1.13  x 1.14  x 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000	DE	ESIGN SHEAR FC 264.26 plf 197.07 plf 0.00 plf 42.52 plf 40.65 plf 9.29 plf 0.00 plf	DRCE (STRE	ENGTH LEVEL 6.50 0.00 0.00 0.00 0.00 0.00 0.00 0.00				
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  2403.73 pif  1781.45 pif  0.00 pif  168.00 pif  0.00 pif	DIST. FA  x 1.13  x 1.14  x 0.00  x 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000	DE	264.26 plf 197.07 plf 0.00 plf 42.52 plf 40.65 plf 9.29 plf 0.00 plf	DRCE (STRE	ENGTH LEVEL 6.50 0.00 0.00 0.00 0.00 0.00 0.00 0.00				



## SEISMIC FORCES TO SHEAR LINES:

2ND LEVEL

ROOF

and the same of	RLINE	FORCES						111100000		WEAT				D DEG: CI		D DEOIS	_	W 1 0 1 5	FFF	FOT		
INE		F#			FORCE			WIDTH		X BAR		L		R DESIGN	V 1	R REQ'D		% LOAD	EFF			0474 00 11
1		F1	-	(	264.26 ptf		) ×	24.00	×	1.00	1	2.00	×	6.50	/	6.50	X	1.00	1	1.00	=	3171.09 lb
	+		7	(			) ×	0.00	×	0.00	/	0.00	×	0.00	/	0.00	×	1.00	1	1.00	=	0.00 lb
	+		-	(			) x	0.00	×	0.00	1	0.00	×	0.00	1	0.00	×	1.00	- 1	1.00	=	0.00 1
	+		7	(			) ×	0.00	×	0.00	1	0.00	×	0.00	1	0.00	×	1.00	1	1.00	=	0.00 18
	+	4	-	(			) x	0.00	×	0.00	1	0.00	X	0.00	1	0.00	×	1.00	1	1.00	=	0.00 lb
	+		*	(			) x	0.00	×	0.00	1	0.00	×	0.00	1	0.00	×	1.00	1	1.00	=	0.00 15
	+				(		) x			0.00	1	0,00	×	0.00	1	0.00	×	1.00	1	1.00	=	0.00 lb
														6.50							_	3171.09 lb
HEA	R LINE F	FORCES			00 00000 A			100000000000000000000000000000000000000	_		_		_									
NE		F#			FORCE			WIDTH		X BAR		L		R DESIGN	4 1	R REQ'D		% LOAD	EFF			
2		F1	-	(	264.26 plf		) x	24.00	×	1.00	1	2.00	X	6.50	1	6.50	X	1.00	1	1.00		3171.09 lb
	#0	-	+	(			) x	0.00	×	0.00	1	0.00	×	0.00	1	0.00	×	1.00	1	1.00	=	0.00 lb
	+		2	(			) x	0.00	×	0.00	1	0.00	X	0.00	1	0.00	×	1.00	1	1.00	=	0.00 lb
	+	-	2	(			) x	0.00	x	0.00	1	0.00	×	0.00	1	0.00	×	1.00	1	1.00	=	0.00 lb
	+			1			) x	0.00	×	0.00	1	0.00	×	0.00	1	0.00	×	1.00	1	1.00	=	0.00 lb
	+	2	2	1			) x	0.00	×	0.00	1	0.00	X	0.00	1	0.00	×	1.00	1	1.00		0.00 lb
			7	1		5	) x	0.00	^	0.00	1	0.00	×	0.00	1	0.00	x	1.00	1	1.00		0.00 lb
	70				V		/^			0.00	,	0.00	^	6.50		0.00	Ŷ	1.00		1.00		3171.09 lb
IEA	R LINE F	ORCES					_		_		_		-								_	
VE.		F#			FORCE			WIDTH		X BAR		L		R DESIGN	4 F	R REQ'D	9	% LOAD	EFF	ECT		
3		F2	-	(	197.07 plf		) ×	24.00	×	1.00	1	2.00	×	6.50	1	6.50	x	1.00	1	1.00	=	2364.83 lb
	+	1000	2	(	10910100111011		) x	0.00	×	0.00	1	0.00	×	0.00	1	0.00	×	1.00	1	1.00	=	0.00 lb
	+			1			) x	0.00	×	0.00	1	0.00	×	0.00	1	0.00	x	1.00	1	1.00	=	0.00 lb
	+			ì			) x	0.00	×	0.00	1	0.00	×	0.00	1	0.00	×	1.00	1	1.00	=	0.00 lb
	+			,			) ×	0.00	×	0.00	1	0.00	×	0.00	1	0.00	x	1.00	1	1.00	10	0.00 lb
	_	- 5	- 7	,			) ×	0.00	×	0.00	,	0.00	×	0.00	1	0.00	×	1.00	,	1.00	=	0.00 lb
	7			,	,			0.00	^	0.00	- 3	0.00	×	0.00	1	0.00	x	1.00	1	1.00	_	0.00 lb
	+	*				-	) x			0.00	,	0.00	×	6.50	1	0.00	^	1.00	,	1.00	=	2364.83 lb
HEAL	R LINE F	ORCES							_		_		_	_	_		_			_		
NE		F#			FORCE		1000	WIDTH		X BAR		L		R DESIGN	N F	REQ'D	0	% LOAD	EFF	ECT		ANADAM DINAKA
4		F2	(1±)	(	197.07 plf		) x	24.00	×	1.00	1	2.00	×	6.50	1	6.50	×	1.00	1	1.00	=	2364.83 lb
	+	F2	-	(	197.07 plf		) x	5.50	x	26.75	1	24.00	×	6.50	1	6.50	×	1.00	1	1.00	=	1208.07 lb
	+	200	-	(	West State of the		) x	0.00	x	0.00	1	0.00	×	0.00	1	0.00	×	1.00	1	1.00	=	0.00 lb
	+		-	1			) ×	0.00	x	0.00	1	0.00	×	0.00	1	0.00	×	1.00	1	1.00		0.00 lb
	+		-	ì			) ×	0.00	x	0.00	1	0.00	×	0.00	1	0.00	×	1.00	1	1.00	=	0.00 lb
	1	8	12	1			) x	0.00	x	0.00	1	0.00	×	0.00	1	0.00	×	1.00	1	1.00	=	0.00 lb
				,	,		)×	0.00	^	0.00	,	0.00	×	0.00	1	0.00	×	1.00	1	1.00	=	0.00 lb
	*	-			(	15	10			0.00	6	0.00	^	6.50		0.00	^	1,00		1.00		3572.90 lb
IEAF	LINEF	ORCES							_				-		_		_	_	_			
1E		F#	-	TW.	FORCE		50.00	WIDTH	1955	X BAR		L	644	R DESIGN	I F	REQ'D		% LOAD	EFF		200	SCASONANNS.
-			-	(			) ×	0.00	×	0.00	1	0.00	×	0.00	1	0.00	×	1.00	1	1.00	=	0.00 lb
	+		-	(			) x	0.00	×	0.00	1	0.00	×	0.00	1	0.00	×	1.00	1	1.00	=	0.00 lb
	+	-		(			)×	0.00	×	0.00	1	0.00	×	0.00	1	0.00	×	1.00	1	1.00	=	0.00 lb
	+		-	ì			)×	0.00	×	0.00	1	0.00	×	0.00	1	0.00	×	1.00	1	1.00		0.00 lb
		- 5		)			)×	0.00	×	0.00	1	0.00	x	0.00	1	0.00	x	1.00	1	1.00	=	0.00 lb
		-		>			)×	0.00	×	0.00	,	0.00	×	0.00	,	0.00	×	1.00	1	1.00	=	0.00 lb
	45	*		(	5			0.00	×	0.00	',	0.00	×	0.00	1	0.00	×	1.00	1	1.00	_	0.00 lb
	-2												×									
	+				(	*	) ×			0.00		0.00	**	100.00	10.1	0.00	***	1750.00	60	1,00	_	0.00 lb



## SEISMIC FORCES TO SHEAR LINES:

1ST LEVEL

2ND FLOOR

	RLINE	FORCES													_	U I A I	and product	COL		
INE		F#	100		FORCE		WIDTH		X BAR		L		R DESIGN	R REQ'D		% LOAD		171111111111111111111111111111111111111		
5		F3	1000	(	42.52 plf	) x	24.00	×	1.00	1	2.00	×	6.50	6.50	X	1.00	1	1.00	=	510.22 lb
	+	-	LINE1	(		3171.09 lb ) x	1.00	×	1.00	1	1.00	×	6.50	6.50	×	1.00	1	1.00	=	3171.09 lb
	+		-	(		) x	0.00	×	0.00	1	0.00	×	0.00	0.00	×	1.00	1	1.00	=	0.00 18
	+	*	-	(		) ×	0.00	×	0.00	1	0.00	×	0.00	0.00	×	1.00	1	1.00	=	0.00 1
	+		-	(		) ×	0.00	×	0.00	1	0.00	×	0.00	0.00	X	1.00	1	1,00	=	0.00 18
	+	-		(		) ×	0.00	×	0.00	1	0.00	×	0.00	0.00	x	1.00	1	1.00		0.00 lb
	+	~			(	- )×			0.00	1	0.00	×	0.00 / 6.50	0.00	×	1.00	1	1.00	=_	0.00 lk 3681.31 lk
													0.00						=	0001101 12
-	R LINE	FORCES F#			FORCE		WIDTH		X BAR		L	_	R DESIGN	R REQ'D		% LOAD	ccc	ECT		
NE				1		17	24.00	v	1.00	,	2.00	x	6.50 /	6.50	x	1.00	1	1.00	=	510.22 lb
6		F3	LINES	,	42.52 plf	3171.09 lb ) x	1.00	×	1.00	,	1.00	×	6.50 /	6.50	X	1.00	,	1.00	=	3171.09 lb
	7	-	LINE2	,		200		×		,	0.00		0.00 /	0.00	×	1.00	,	1.00	_	0.00 lb
	+	-	7	(		) ×	0.00	×	0.00	',		X				1.00	1		-	0.00 lb
	+			,		) x	0.00	×	0.00	',	0.00	X	0.00	0.00	X		',	1.00		0.00 lb
	+	-		(		)×	0.00	×	0.00	'	0.00	X	0.00	0.00	X	1.00	,	1.00		
	+	(4)	*	(	4	) x	0.00	×	0.00	,	0.00	X	0.00 /	0.00	X	1.00	1	1.00	=	0.00 18
	+				(	- )×			0.00	1	0.00	X	0.00 /	0.00	х	1.00	1	1.00	=	0.00 lb
													6.50						-	3681.31 lb
	R LINE	FORCES																		
NE		F#			FORCE	Yw.	WIDTH 24.00		X BAR 1.00		2.00	x	R DESIGN 6.50 /	R REQ'D 6.50	×	% LOAD 1.00	EFF!	1.00	=	487.85 Ib
7	100	F4	VIII.	Ç	40.65 plf	) X		×		1			6.50 /	6.50		1.00	1	1.00	=	2364.83 lb
	+	*:	LINE3	,		2364.83 lb ) x	1.00	×	1.00	1.	1.00	X			×		',	1,00	-	0.00 lb
	+	*	-	(		) ×	0.00	X	0.00	1	0.00	×	0.00 /	0.00	X	1.00	',			
	+	-		(		) ×	0.00	X	0.00	1	0.00	×	0.00 /	0.00	×	1,00	',	1.00	=	0.00 1
	+	-	*	(		) ×	0.00	×	0.00	1	0,00	×	0.00 /	0.00	X	1.00	1	1.00	=	0.00 lb
	+	-	-	(	a di	) ×	0.00	×	0.00	1	0.00	×	0.00 /	0.00	×	1.00	1	1.00	=	0.00 lb
	+				(	- )×			0.00	1	0.00	X	0.00 / 6.50	0.00	X	1.00	1	1.00	-	0.00 lb 2852.67 lb
													170071		_					
HEAL	PLINE	EORCES																		
****	R LINE	FORCES			FORCE		WIDTH		X BAR		L	_	R DESIGN	R REQ'D	9	% LOAD	EFF	ECT		
NE	RLINE	F#	_	(	FORCE 40.65 plf	) x	WIDTH 24.00	×	X BAR	,		×	R DESIGN 6.50 /	R REQ'D 6.50	×	% LOAD 1.00	EFF	ECT 1.00	-	487.85 lb
-	R LINE	F# F4	:	(	40.65 plf	) x ) x	WIDTH 24.00 5.50	×	X BAR 1.00 26.75	1	L 2,00 24,00	×					EFFI		= =	
NE	R LINE	F#	H. W. Toward	(		) x	24.00 5.50	×	1.00 26.75	1	2,00 24.00	×	6.50 / 6.50 /	6.50 6.50	x	1.00	EFFI / /	1.00		56.96 lb
NE	+	F# F4		(	40.65 plf	) x 3572.90 lb ) x	24.00 5.50 1.00	×	1.00 26.75 1.00	1	2,00 24.00 1.00	×	6.50 / 6.50 /	6.50	x x x	1.00	1	1.00	=	56.96 lb 3572.90 lb
NE	+	F# F4	LINE4	( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	40.65 plf	3572.90 lb ) x ) x	24.00 5.50 1.00 0.00	× ×	1.00 26.75 1.00 0.00	1 1 1 1 1 1	2,00 24.00 1.00 0.00	×××	6.50 / 6.50 / 6.50 / 0.00 /	6.50 6.50 6.50 0.00	x x x	1.00 1.00 1.00 1.00	1	1.00 1.00 1.00 1.00	=	56,96 lb 3572,90 lb 0,00 lb
NE	+	F# F4	H. W. Toward	( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	40.65 plf	3572.90 lb ) x ) x ) x ) x	24.00 5.50 1.00 0.00 0.00	× × ×	1.00 26.75 1.00 0.00 0.00	1	2.00 24.00 1.00 0.00 0.00	× × ×	6.50 / 6.50 / 6.50 / 0.00 /	6.50 6.50 6.50 0.00 0.00	x x x x	1.00 1.00 1.00 1.00 1.00	1111	1.00 1.00 1.00 1.00 1.00	= = =	56.96 lb 3572.90 lb 0.00 lb 0.00 lb
NE	+	F# F4	LINE4	( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	40.65 plf	3572.90 lb ) x ) x ) x ) x ) x	24.00 5.50 1.00 0.00	× ×	1.00 26.75 1.00 0.00 0.00 0.00	1 1 1 1 1 1 1 1 1	2,00 24.00 1.00 0.00 0.00 0.00	× × × ×	6.50 // 6.50 // 6.50 // 0.00 // 0.00 //	6.50 6.50 6.50 0.00 0.00	x x x x	1.00 1.00 1.00 1.00 1.00	1	1.00 1.00 1.00 1.00 1.00 1.00	= =	56.96 lb 3572.90 lb 0.00 lb 0.00 lb 0.00 lb
NE	+	F# F4	LINE4	( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	40.65 plf	3572.90 lb ) x ) x ) x ) x	24.00 5.50 1.00 0.00 0.00	× × ×	1.00 26.75 1.00 0.00 0.00	! ! ! !	2.00 24.00 1.00 0.00 0.00	× × ×	6.50 / 6.50 / 6.50 / 0.00 /	6.50 6.50 6.50 0.00 0.00	x x x x	1.00 1.00 1.00 1.00 1.00	1111	1.00 1.00 1.00 1.00 1.00		487.85 lb 56.96 lb 3572.90 lb 0.00 lb 0.00 lb 0.00 lb 4117.71 lb
NE 8	+ + + + +	F# F4 F5 - - -	LINE4	( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	40.65 plf	3572.90 lb ) x ) x ) x ) x ) x	24.00 5.50 1.00 0.00 0.00	× × ×	1.00 26.75 1.00 0.00 0.00 0.00	1 1 1 1 1 1	2,00 24.00 1.00 0.00 0.00 0.00	× × × ×	6.50 / 6.50 / 6.50 / 0.00 / 0.00 / 0.00 /	6.50 6.50 6.50 0.00 0.00	x x x x	1.00 1.00 1.00 1.00 1.00	1111	1.00 1.00 1.00 1.00 1.00 1.00		56.96 lb 3572.90 lb 0.00 lb 0.00 lb 0.00 lb
NE 8	+ + + + +	F# F4	LINE4	( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	40.65 plf	3572.90 lb ) x ) x ) x ) x ) x	24.00 5.50 1.00 0.00 0.00 0.00	× × ×	1.00 26.75 1.00 0.00 0.00 0.00 0.00	1 1 1 1 1 1 1 1	2,00 24.00 1.00 0.00 0.00 0.00 0.00	× × × ×	6.50 / 6.50 / 6.50 / 0.00 / 0.00 / 0.00 / 6.50	6.50 6.50 6.50 0.00 0.00 0.00 0.00	× × × × ×	1.00 1.00 1.00 1.00 1.00 1.00 1.00	11111	1.00 1.00 1.00 1.00 1.00 1.00		56,96 lb 3572,90 lb 0,00 lb 0,00 lb 0,00 lb 4117,71 lb
NE 8	+ + + + +	F# F4 F5 - - - - -	LINE4	( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	40.65 plf 9.29 plf	3572.90 lb ) x ) x ) x ) x ) x	24.00 5.50 1.00 0.00 0.00 0.00	× × ×	1.00 26.75 1.00 0.00 0.00 0.00 0.00	1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	2,00 24.00 1.00 0.00 0.00 0.00 0.00	× × × ×	6.50 / 6.50 / 6.50 / 0.00 / 0.00 / 0.00 / 6.50	6.50 6.50 6.50 0.00 0.00 0.00 0.00	x x x x x	1.00 1.00 1.00 1.00 1.00 1.00 1.00	11111	1.00 1.00 1.00 1.00 1.00 1.00 1.00		56,96 lb 3572,90 lb 0.00 lb 0.00 lb 0.00 lb 4117.71 lb
NE 8	+ + + + +	F# F4 F5 - - - - -	LINE4	( ( (	40.65 plf 9.29 plf	3572.90 lb ) x 3572.90 lb ) x ) x ) x ) x	24.00 5.50 1.00 0.00 0.00 0.00	x x x x	1.00 26.75 1.00 0.00 0.00 0.00 0.00	1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	2,00 24.00 1.00 0.00 0.00 0.00 0.00	× × × ×	6.50 / 6.50 / 6.50 / 0.00 / 0.00 / 0.00 / 6.50	6.50 6.50 6.50 0.00 0.00 0.00 0.00	× × × × ×	1.00 1.00 1.00 1.00 1.00 1.00 1.00	11111	1.00 1.00 1.00 1.00 1.00 1.00 1.00		56.96 lb 3572.90 lb 0.00 lb 0.00 lb 0.00 lb 4117.71 lb 0.00 lb
NE 8	+ + + + +	F# F4 F5 - - - - -	LINE4	( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	40.65 plf 9.29 plf	) x 3572.90 lb ) x ) x ) x ) x - ) x	24.00 5.50 1.00 0.00 0.00 0.00	x x x x	1.00 26.75 1.00 0.00 0.00 0.00 0.00 0.00	1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	2,00 24.00 1.00 0.00 0.00 0.00 0.00	× × × × ×	6.50 / 6.50 / 6.50 / 0.00 / 0.00 / 0.00 / 6.50	6.50 6.50 6.50 0.00 0.00 0.00 0.00	× × × × × × ×	1.00 1.00 1.00 1.00 1.00 1.00 1.00	11111	1.00 1.00 1.00 1.00 1.00 1.00 1.00		56.96 lb 3572.90 lb 0.00 lb 0.00 lb 0.00 lb 4117.71 lb 0.00 lb
NE 8	+ + + + +	F# F4 F5 - - - - -	LINE4	((()	40.65 plf 9.29 plf	) x 3572.90 lb ) x ) x ) x ) x - ) x	24.00 5.50 1.00 0.00 0.00 0.00 0.00	x x x x	1.00 26.75 1.00 0.00 0.00 0.00 0.00 X BAR 0.00 0.00	1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	2,00 24.00 1.00 0.00 0.00 0.00 0.00	× × × × × ×	6.50 / 6.50 / 6.50 / 0.00 / 0.00 / 0.00 / 6.50	6.50 6.50 6.50 0.00 0.00 0.00 0.00 R REQ'D 0.00 0.00	× × × × × × × ×	1.00 1.00 1.00 1.00 1.00 1.00 1.00	11111	1.00 1.00 1.00 1.00 1.00 1.00 1.00		56.96 lb 3572.90 lb 0.00 lb 0.
NE 8	+ + + + +	F# F4 F5 - - - - -	LINE4	((((	40.65 plf 9.29 plf	) x 3572.90 lb ) x ) x ) x ) x - ) x	24.00 5.50 1.00 0.00 0.00 0.00 0.00	× × × × × ×	1.00 26.75 1.00 0.00 0.00 0.00 0.00 0.00	1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	2,00 24.00 1.00 0.00 0.00 0.00 0.00 0.00	× × × × × ×	6.50 / 6.50 / 6.50 / 0.00 / 0.00 / 0.00 / 6.50 / R DESIGN 0.00 / 0.00 /	6.50 6.50 6.50 0.00 0.00 0.00 0.00 0.00	× × × × × × × ×	1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00	EFFE	1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00		56.96 lt 3572.90 lt 0.00 lt 0.
NE 8	+ + + + +	F# F4 F5 - - - - -	LINE4	( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	40.65 plf 9.29 plf	) x 3572.90 lb ) x ) x ) x ) x - ) x	24.00 5.50 1.00 0.00 0.00 0.00 0.00 0.00 0	x x x x	1.00 26.75 1.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	2,00 24,00 1,00 0,00 0,00 0,00 0,00 0,00 0,00	× × × × × ×	6.50 / 6.50 / 6.50 / 0.00 / 0.00 / 0.00 / 6.50 / R DESIGN 0.00 / 0.00 / 0.00 / 0.00 /	6.50 6.50 6.50 0.00 0.00 0.00 0.00 0.00	× × × × × × × × × ×	1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00	EFFE	1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00		56.96 lt 3572.90 lt 0.00 lt 0.
NE 8	+ + + + +	F# F4 F5 - - - - -	LINE4		40.65 plf 9.29 plf	) x 3572.90 lb ) x ) x ) x ) x - ) x	24.00 5.50 1.00 0.00 0.00 0.00 0.00 0.00 0	x x x x	1.00 26.75 1.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	2,00 24.00 1.00 0.00 0.00 0.00 0.00 0.00 0.00	x x x x x x x x x x x x x x x x x x x	6.50 / 6.50 / 6.50 / 0.00 / 0.00 / 0.00 / 6.50 / R DESIGN 0.00 / 0.00 / 0.00 / 0.00 /	6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	× × × × × × × × × × × × × × × ×	1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00	EFFE	1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00		56.96 lb 3572.90 lb 0.00 lb 0.00 lb 0.00 lb 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:

STRENG	March State Company of the Company o	SHEAR FORCE SUN	IMARY: rho	1/ (	SEISMIC (STRENGTH)	-		***********	V SEISMIC (ASD)
Francisco Contraction of the Con	The second second	ORCE	navalek sulle musikka susekit in	orio Errore Milita	THE PROPERTY OF THE PROPERTY O	-	φ 0.70		HISTORIAN CONTRACTOR C
ı	-	0.00 lb x	1.30	=	·0.00 lb	Х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	Х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	Х	0.70	=	dl 00.0
l	-	0.00 lb x	1.30	=	0.00 lb	Х	0.70	=	0.00 lb
į	-	0.00 lb x	1.30	=	0.00 lb	Х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
1 .	-	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	dl 00.0	×	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
l	l -	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
www.	-	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	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	Х	0.70	=	0.00 lb
1	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
1	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	Х	0.70	=	0.00 lb
,	_	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	0.00 lb
1	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	Х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	Х	0.70	=	0.00 lb
1	l -	0.00 lb x	1.30	=	0.00 lb	Х	0.70	=	0.00 lb
1	<b>.</b>	0.00 lb x	1.30	=	0.00 lb	Х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	Х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	X	0.70	=	0.00 lb
	-	0.00 lb x 0.00 lb x	1.30 1.30	=	dl 00.0 dl 00.0	X	0.70 0.70	=	0.00 lb 0.00 lb
	1	0.00 lb x 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 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
1	-	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	di 00.0
	[	0.00 lb x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
l		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
ᄔ	1 _	0.00 lb x	1.30	=	0.00 lb	x	0.70	=	0.00 lb
ROOF	<b>l</b> _	0.00 lb x		=	0.00 lb	×	0.70	=	0.00 lb
Œ	_	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	0.00 lb
	_	0.00 lb x		=	0.00 lb	X	0.70	=	0.00 lb
	_	0.00 lb x		=	0.00 lb	x	0.70	=	0.00 lb
	_	0.00 lb x		=	0.00 lb	X	0.70	=	0.00 lb
	-	0.00 lb x		=	0.00 lb	x	0.70	=	0.00 lb
	-	0.00 lb x		=	0.00 lb	x	0.70	=	0.00 lb
1	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
I	-	0.00 lb x		=	0.00 lb	х	0.70	=	0.00 lb
·	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	×	0.70	=	3349.99 lb
	7	2852.67 lb x	1.30	=	3708.48 lb	х	0.70	=	2595.93 lb
<b>[</b>	8	4117.71 lb x	1.30	=	5353.03 lb	х	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	×	0.70	=	0.00 lb
	-	0.00 lb x		=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x		=	0.00 lb	х	0.70	=	0.00 lb
~	-	0.00 lb x		=	0.00 lb	х	0.70	=	0.00 lb
Ö	-	0.00 lb x		=	0.00 lb	х	0.70	=	0.00 lb
2ND FLOOR	-	0.00 lb x		=	0.00 lb	х	0.70	=	0.00 lb
DF	-	0.00 lb x		=	0.00 lb	х	0.70	=	0.00 lb
2 <u>N</u>	-	0.00 lb x		=	0.00 lb	х	0.70	=	0.00 lb
``	-	0.00 lb x		=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x		=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x		=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x		=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x		=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x		=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x		=	0.00 lb	х	0.70	= .	0.00 lb
	-	0.00 lb x		=	0.00 lb	х	0.70	=	0.00 lb
***************************************	-	0.00 lb x	1.30	=	0.00 lb	X	0.70	=	0.00 lb



## WIND LOAD DESIGN:

-001014	DIRE	CTIONAL PROCEDURE - MWFRS ENCLOSED BUILDING					ASCE 7-10 27.4.1
5-1A	BASI	C WIND SPEEDS FOR RISK CATEGORY II	BUILDING	38			
	1	BASIC WIND SPEED WIND DIRECTIONALLY FACTOR	V Kd	=	115.00 MP 0.85	н	ASCE 7-10 FIG 26.5-1 ASCE 7-10 TABLE 26.6-1
	2	IMPORTANCE FACTOR, CATEGORY II	1	-	1.00		ASCE 7-10 TABLE 6-1
II DIN	3 DIRE	CTION - X					
LUIIV	3	EXPOSURE CATEGORY		С	-		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 TERRAIN EXPOSURE CONSTRANT	α.	=	9.50 900.00 ft		ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1
		VELOCITY PRESSURE	zg Kz	=	0.90		ASCE 7-10 TABLE 27.3-1
	0		2357		WW.		SAMOURE AND
	4	TOPOGRAPHY FACTOR	Kzt	=	1.000	5	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
		CHET FACTOR	G	_	0.95		ASCE 7-10 26.9.1
	6	GUST FACTOR	G	7	0.85		ASCE 7-10 26.9.1
	7-	ENCLOSURE CLASSIFICATION		ENCLOSE	D		ASCE 7-10 26.10
	8	INTERNAL PRESSURE	GCpi	= -	0.18 +/-		ASCE 7-10 TABLE 26.11-1
	9	EXTERNAL PRESSURE COEFFICIENTS	esteration (	_			ASCE 7-10 FIG 27.4.1
	Э	EXTERNAL PRESSURE COEFFICIENTS	В	=	24.00		ASCE 7-10 FIG 27.4.1 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
		COUNTY PARTY SE	Ср 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
	10	DESIGN WIND LOAD	Cp 5	= qGCp - qi (	-0.900 Gcpi)	ROOF 2 0.00 deg	PITCH 0.00 :12 ASCE 7-10 TABLE 26.11-1
	10	WINDWARD q = qz & qi = qh	p	= 4000 4.1	17.649 -	-4.672 = 22.	321 psf LATERAL FORCE
		LEEWARD q = qh	p	=	-11.030 -		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 -	(M. 77.378)	
						*15.70 = 38	3.02 psf UPLIFT FORCE
			p ROOF		0.00 -		0.00 psf UPLIFT FORCE
LDING		CTION - Y					0.00 psf
ILDING				c			0.00 psf ASCE 7-10 26.7.3
ILDING		CTION - Y EXPOSURE CATEGORY	p ROOF		0.00 -		ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3
ILDING		CTION - Y	p ROOF	C CASE 1	0.00 - 20.00 ft		0.00 psf ASCE 7-10 26.7.3
ILDING		CTION - Y EXPOSURE CATEGORY HEIGHT ABOVE GROUND LEVEL	p ROOF	C CASE 1	0.00 -		2.00 psf  ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3 MEAN ROOF HEIGHT
ILDING		CTION - Y EXPOSURE CATEGORY HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT	p ROOF  z a zg	C CASE 1	0.00 - 20.00 ft 9.50		ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1
ILDING	3	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE	p ROOF  z α zg Kz	C CASE 1	20.00 ft 9.50 900.00 ft 0.90		ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1
ILDING	3	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT	p ROOF  z α zg Kz	C CASE 1	20.00 ft 9.50 900.00 ft		ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1
ILDING	3	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE	p ROOF  z α zg Kz	C CASE 1	20.00 ft 9.50 900.00 ft 0.90		ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1
ILDING	3 4 5	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR	p ROOF  z α zg Kz Kzt	C CASE 1	20.00 ft 9.50 900.00 ft 0.90 1.000		ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 (27.3-1)
ILDING	3 4 5	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE	p ROOF  z α zg Kz  Kzt  qz qz	C CASE 1	20.00 ft 9.50 900.00 ft 0.90 1.000 Kzt Kd V^2 I 25.954 psf		ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 26.8.2 ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT
ILDING	3 4 5	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR  VELOCITY PRESSURE  GUST FACTOR	z α zg Kz Kzt qz qz G	C CASE 1	20.00 ft 9.50 900.00 ft 0.90 1.000 Kzt Kd V^2 I 25.954 psf		ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT ASCE 7-10 26.9.1
ILDING	3 4 5 6 7 8	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE	p ROOF  z α zg κz κzt σz σ G GCpi	C CASE 1	20.00 - 20.00 ft 9.50 900.00 ft 0.90 1.000 Kzt Kd V^2 I 25.954 psf 0.85		ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT ASCE 7-10 26.9.1 ASCE 7-10 26.9.1 ASCE 7-10 26.10 ASCE 7-10 TABLE 26.11-1
ILDING	3 4 5 6 7	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR  ENCLOSURE CLASSIFICATION	p ROOF  z α zg Kz Kzt qz qz G GCpi	C CASE 1	20.00 - 20.00 ft 9.50 900.00 ft 0.90 1.000 Kzt Kd V^2 I 25.954 psf 0.85		ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 26.8.2 ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT ASCE 7-10 26.9.1 ASCE 7-10 26.9.1
LDING	3 4 5 6 7 8	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE	z α α zg Kz Kzt qz qz G GCpi L B	C CASE 1	20.00 - 20.00 ft 9.50 900.00 ft 0.90 1.000 Kzt Kd V^2 I 25.954 psf 0.85		ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT ASCE 7-10 26.9.1 ASCE 7-10 26.9.1 ASCE 7-10 26.9.1 ASCE 7-10 7ABLE 26.11-1 ASCE 7-10 TABLE 26.11-1 ASCE 7-10 TABLE 26.11-1
LDING	3 4 5 6 7 8	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE	p ROOF  z α zg κz κzt qz qz G GCpi L B L/B h/L	C CASE 1	0.00 -  20.00 ft 9.50 900.00 ft 0.90 1.000  Kzt Kd V^2 I 25.954 psf  0.85  0.18 +/- 24.00 24.00	0.00 =	ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT ASCE 7-10 26.9.1 ASCE 7-10 26.9.1 ASCE 7-10 TABLE 26.11-1 ASCE 7-10 TABLE 26.11-1 ASCE 7-10 TABLE 26.11-1 ASCE 7-10 FIG 27.4.1
LDING	3 4 5 6 7 8	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE	p ROOF  z α zg Kz Kzt qz qz G GCpi L B L/B h/L Cp 1	C CASE 1 = = .00256 Kz   = = = = = = = = = = = = = = = = = =	0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000  Kzt Kd V^2 I 25.954 psf  0.85  0.18 + / -  24.00 24.00 1.000 0.833 0.800	0.00 =	ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 26.8.2 ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT ASCE 7-10 26.9.1 ASCE 7-10 26.9.1 ASCE 7-10 TABLE 26.11-1 ASCE 7-10 FIG 27.4.1
LDING	3 4 5 6 7 8	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE	p ROOF  z α α zg Kz Kzt  qz qz G GCpi L B L/B h/L Cp 1 Cp 2	C CASE 1 = = .00256 Kz   = = = = = = = = = = = = = = = = = =	0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000  Kzt Kd V^2 I 25.954 psf  0.85  0.18 +/- 24.00 24.00 1.000 0.833 0.800 -0.500	WINDWARD WALL LEEWARD WALL	ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT ASCE 7-10 26.9.1 ASCE 7-10 TABLE 26.11-1 ASCE 7-10 FIG 27.4.1
ILDING	3 4 5 6 7 8	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR  VELOCITY PRESSURE  GUST FACTOR  ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS	p ROOF  z α zg kz  Kzt  qz qz  G  GCpi  L B L/B h/L Cp 1 Cp 2 Cp 3	C CASE 1 = = .00256 Kz I = = = .00256 Kz I = = = = = = = = = = = = = = = = = =	0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000  Kzt Kd V^2 I 25.954 psf  0.85  0.18 +/-  24.00 24.00 1.000 0.833 0.800 -0.500 -0.700	0.00 =	ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT  ASCE 7-10 26.9.1  ASCE 7-10 26.10  ASCE 7-10 TABLE 26.11-1  ASCE 7-10 FIG 27.4.1
LDING	3 4 5 6 7 8	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE	p ROOF  z α zg Kz Kzt qz qz G GCpi L B L/B h/L Cp 1 Cp 2 Cp 3 Cp 4	C CASE 1 = = :.00256 Kz I = = :.00256 Kz I = = :.00256 Kz I = :.00	0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000  Kzt Kd V^2 I 25.954 psf  0.85  0.18 +/-  24.00 24.00 24.00 0.833 0.800 -0.500 -0.700 -0.900	WINDWARD WALL LEEWARD WALL SIDE WALLS ROOF 1 0.00 deg	ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT ASCE 7-10 26.9.1 ASCE 7-10 26.9.1 ASCE 7-10 FIG 27.4.1
LDING	3 4 5 6 7 8 9	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS	z α α zg Kz Kzt qz qz G GCpi L B L/B h/L Cp 1 Cp 2 Cp 3 Cp 4 Cp 4	C CASE 1	0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000  Kzt Kd V^2 I 25.954 psf  0.85  0.18 +/-  24.00 1.000 0.833 0.800 -0.500 -0.700 -0.900	0.00 =	ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1  ASCE 7-10 7ABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT  ASCE 7-10 26.9.1  ASCE 7-10 TABLE 26.11-1  ASCE 7-10 FIG 27.4.1
LDING	3 4 5 6 7 8 9	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR  VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS  NORMAL TO RIDGE ( > 10 deg ) DESIGN WIND LOAD	y ROOF  z α α 2g Kz  Kzt  qz qz  G  GCpi  L  B  L/B h/L  Cp 1  Cp 2  Cp 3  Cp 4  p	C CASE 1 = = :.00256 Kz I = = :.00256 Kz I = = :.00256 Kz I = :.00	0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000  Kzt Kd V^2 I 25.954 psf  0.85  0.18 +/-  24.00 24.00 1.000 0.833 0.800 -0.500 -0.700 -0.900 Gcpl)	WINDWARD WALL LEEWARD WALL SIDE WALLS ROOF 1 0.00 deg ROOF 2 0.00 deg	ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT  ASCE 7-10 26.9.1  ASCE 7-10 26.9.1  ASCE 7-10 TABLE 26.11-1  ASCE 7-10 FIG 27.4.1 ASCE 7-10 TABLE 26.11-1
LDING	3 4 5 6 7 8 9	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR  VELOCITY PRESSURE  GUST FACTOR  ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS  NORMAL TO RIDGE ( > 10 deg )  DESIGN WIND LOAD WINDWARD q = qz & qi = qh	z α zg κz κzt qz qz G GCpi L B L/B h/L Cp 1 Cp 2 Cp 3 Cp 4 Cp 4 P P P	C CASE 1 = = .00256 Kz   = = = = = = = = = = = = = = = = = =	0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000  Kzt Kd V^2 I 25.954 psf  0.85  0.18 +/-  24.00 24.00 1.000 0.833 0.800 -0.500 -0.700 -0.900 GCpl) 17.649 -	WINDWARD WALL LEEWARD WALL SIDE WALLS ROOF 1 0.00 deg ROOF 2 0.00 deg	ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT  ASCE 7-10 26.9.1  ASCE 7-10 26.10  ASCE 7-10 TABLE 26.11-1  ASCE 7-10 FIG 27.4.1
LDING	3 4 5 6 7 8 9	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR  VELOCITY PRESSURE  GUST FACTOR  ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS  NORMAL TO RIDGE ( > 10 deg )  DESIGN WIND LOAD WINDWARD q = qz & qi = qh LEEWARD q = qh	z α zg κz κzt qz qz G GCpi L B L/B h/L Cp 1 Cp 2 Cp 3 Cp 4 Cp 4 P P P	C CASE 1 = = .00256 Kz I = = .00256 Kz I = = = .00256 Kz I = = = .00256 Kz I = = .00256 Kz I = .0025	0.00 -  20.00 ft 9.50 900.00 ft 0.90 1.000  Kzt Kd V^2 I 25.954 psf  0.85  0.18 +/- 24.00 24.00 1.000 0.833 0.800 -0.500 -0.700 -0.900 Gcpi) 17.64911.030 -	WINDWARD WALL LEEWARD WALL SIDE WALLS ROOF 1 0.00 deg ROOF 2 0.00 deg -4.672 = 22. 4.672 = -15.	ASCE 7-10 26.7.3  ASCE 7-10 TABLE 6-3  MEAN ROOF HEIGHT  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 (27.3-1)  PRESSURE AT MEAN ROOF HEIGHT  ASCE 7-10 26.9.1  ASCE 7-10 26.9.1  ASCE 7-10 TABLE 26.11-1  ASCE 7-10 FIG 27.4.1
LDING	3 4 5 6 7 8 9	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR  VELOCITY PRESSURE  GUST FACTOR  ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS  NORMAL TO RIDGE ( > 10 deg )  DESIGN WIND LOAD WINDWARD q = qz & qi = qh LEEWARD q = qh SIDE WALLS qi = qh	z α zg κz κzt qz qz G GCpi L B L/B h/L Cp 1 Cp 2 Cp 4 Cp 4 P P P P	C CASE 1 = = : .00256 Kz I = = : .00256 Kz I = = : .00256 Kz I	0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000  Kzt Kd V^2 I 25.954 psf  0.85  0.18 +/-  24.00 24.00 24.00 1.000 0.833 0.800 -0.500 -0.700 -0.900 -0.900 GSph) 17.64911.03015.443 -	WINDWARD WALL LEEWARD WALL SIDE WALLS ROOF 1 0.00 deg ROOF 2 0.00 deg 4.672 = 22.3 4.672 = -15.4 4.672 = -20.3	ASCE 7-10 26.7.3  ASCE 7-10 TABLE 6-3  MEAN ROOF HEIGHT  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 (27.3-1)  PRESSURE AT MEAN ROOF HEIGHT  ASCE 7-10 26.9.1  ASCE 7-10 26.9.1  ASCE 7-10 TABLE 26.11-1  ASCE 7-10 FIG 27.4.1  ASCE 7-10 FIG 37.4.1  ASCE 7
LDING	3 4 5 6 7 8 9	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR  VELOCITY PRESSURE  GUST FACTOR  ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS  NORMAL TO RIDGE ( > 10 deg )  DESIGN WIND LOAD WINDWARD q = qx & qi = qh LEEWARD q = qh SIDE WALLS qi = qh ROOF 1 qi = qh	Σ α α 2 g Kz Kzt qz qz G GCpi L B L/B h/L Cp 1 Cp 2 Cp 3 Cp 4 P P P P P P P P P P	C CASE 1	0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000  Kzt Kd V^2 I 25.954 psf  0.85  0.18 +/-  24.00 1.000 0.833 0.800 -0.500 -0.700 -0.900 Gcpi) 17.64911.03015.44319.855 +	WINDWARD WALL LEEWARD WALL SIDE WALLS ROOF 1 0.00 deg ROOF 2 0.00 deg -4.672 = 22. 4.672 = -15. 4.672 = -204.672 = -20.	ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT  ASCE 7-10 26.9.1  ASCE 7-10 26.9.1  ASCE 7-10 FIG 27.4.1 ASCE 7-10 FIG
LDING	3 4 5 6 7 8 9	EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR  VELOCITY PRESSURE  GUST FACTOR  ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS  NORMAL TO RIDGE ( > 10 deg )  DESIGN WIND LOAD WINDWARD q = qz & qi = qh LEEWARD q = qh SIDE WALLS qi = qh	Σ α α 2 g Kz Kzt qz qz G GCpi L B L/B h/L Cp 1 Cp 2 Cp 3 Cp 4 P P P P P P P	= .00256 Kz l	0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000  Kzt Kd V^2 I 25.954 psf  0.85  0.18 +/-  24.00 24.00 24.00 1.000 0.833 0.800 -0.500 -0.700 -0.900 -0.900 GSph) 17.64911.03015.443 -	WINDWARD WALL LEEWARD WALL SIDE WALLS ROOF 1 0.00 deg ROOF 2 0.00 deg -4.672 = 22. 4.672 = -15. 4.672 = -204.672 = -244.672 = -24.	ASCE 7-10 26.7.3  ASCE 7-10 TABLE 6-3  MEAN ROOF HEIGHT  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 (27.3-1)  PRESSURE AT MEAN ROOF HEIGHT  ASCE 7-10 26.9.1  ASCE 7-10 26.9.1  ASCE 7-10 TABLE 26.11-1  ASCE 7-10 FIG 27.4.1  ASCE 7-10 FIG 37.4.1  ASCE 7



## HOFFMAVER INC. STRUCTURAL ENGINEERING

WIND FORCES:

TRIB HT VWIND (STRENGT		0.00 × 0.00 ± ( ± 0.00 × 0.00	× ×	(t) = (1) 000 × 000	0.00 × 0.00 (ft ) = 0.00	0000 × 0000 × 0000	0.00 × 0.00 ft ) #	0.00 × 0.00 ft ) =	= ( # 000 × 000	0.00 × 0.00	0.00 × 0.00	0000 # (# 0000 × 000	000 = ( # ) 000 × 000	0.00 × 00.00	$0.00 \times 0.00$ ft ) = 0.00	$0.00 \times 0.00$ ft ) =	0.00 × 0.00 ft ) = 0.00	0.00 × 0.00 ft ) = 0.00	= (1) 0000 × 0000	0.00 x 0.00 ft ) =	$0.00 \times 0.00$ ft ) = 0.00	0.00 × 0.00 ft ) ==	0.00 × 0.00 H	× 00.0	0.00 x 0.00 ff ) = 0.00	0.00 × 0.00 u ) = 0.00	× × 0000	0.00 × 0.00 ft ) = 0.00	0.00 × 00.00	0.00 × 0.00 ft ) = 0.00	0.00 × 0.00 ft ) = 1	0.00 × 0.00 ft) = 0.00	0.00 x 0.00 ft) = 0.00	0.00 × 0.00 ft ) = 0.00	00.00 × 00.00	$0.00 \times 0.00 \text{ ft}$ ) = 0.00	$0.00 \times 0.00$ ft ) = 0.00	0.00 × 0.00 ft ) = 380.23	1 ( H 00.0 × 00.0	0.00 = (1) = 0.00 0.00 0.00	$0.00 \times 7.00 \text{ (ft)} = 0.00$	$0.00 \times 0.00$ ft ) = 0.00	0.00 × 0.00 ft) = 0.00	0.00 × 0.00 H ) = 0.00	11 0000 × 0000 × 0000	0.00 × 0.00 ft ) =	
RIB. HT. LEE FORCE ROOF	ft - 0.00 psf x	0.00	ft - 0.00 psf x	ft - 0.00 psf x	ft - 0.00	ft - 0.00 psf x	ft - 0.00 psf x	ft - 0.00 psf x	ft - 0.00 psf x	ft - 0.00 psf x	ft - 0.00 psf x	ft - 0.00 psf x	ft - 0.00 psf x	ft - 0.00	ft - 0.00 psf x	ft - 0.00 psf x	ft ~ 0.00	π - 0.00 pst x	# - 0.00 psi x	ft - 0.00 psf x	ft - 0.00 psf x	ft - 0.00 psf x	0.00	# - 0.00 - #	0.00 ft - 0.00 psf x SiN	X 180 0.00 - 11	ft24.53 ft24.53	ft - 0.00 psf x	ft - 0.00 psf x	# - 0.00 #	11 - 0.00 psi x	ft - 0.00 psf x	ft - 0.00 psf x	, ; E ¢	# 0.00 psf ×	ft - 0.00 psf x	ft - 0.00 psf x	ft24.53 ft24.53	ft24.53 psf x	ft - 0.00 psf x	ft - 0.00 psf x	ft - 0.00 psf x	ft - 0.00 psf x	- 11	0.00 psi x	ft - 0.00 psf x	
D FORCE ROOF	× 00.0	OCO NIS × Jsd	0.00 NIS × 3rd 0.00	psf x SIN 0.00	psf x SiN 0.00	psf x SIN 0.00	0.00	psf x SIN 0.00	psf x SiN	psf × SiN	psf × SIN	psf × SIN	psf x SIN	SIN	psf × SIN	pst × SIN	Z 2	× ×	psf × SiN	psf x SiN	psf × SIN	psf × SiN	pst × sin	psf × SiN	psf ×	DSI X SIN 0.00	-24.53 psf x SIN 0.00 x -24.53 psf x SIN 0.00 x	psf x SiN	psf × SIN	pst × sin	psf × SiN	psf x SiN		psr × sin	psf × SIN	psf x SIN 0.00	psf × SIN	-24.53 pst x SIN 0.00 x -24.53 psf x SIN 0.00 x	psf × SIN	psf x SiN	psf x SiN	psf x SIN		x 0.00 Nis x set 0.00	ost × SIN	psf × SiN	
WIND FORCE WALL TRIB. HEIGHT WINI	0.00 x Jsd 00.0		0.00 psf x 0.00 + (			bsf x	0.00 psf x 0.00 + (	psf x	0.00 psf × 0.00 + (	bsf x	x jsd	bsf ×	x jsd	psf	0.00 psf x 0.00 + (	bsf ×	bot ×	× ×	psf ×	psf x	0.00 psf x 0.00 + (	bst ×	××	y y	0.00 psf × 0.00 +(	751 ×	38.02 psf x 5.00 + (	psf x	psf ×	pst ×	psf ×	psf x	psf x	y ×	psf ×	0.00 psf x 0.00 + (	bst ×	38.02 pst x 10.00 +(	bsf ×	bst ×	psf x	psf x	0.00 psf × 0.00 + (	××	pst bst	psf ×	7 70 0 may 2 7 3 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2
P# WIND		1			,	•	ı	ı	,	1	,		,	,	,	,			1	1	1	1	: 1		, ,		2 2	ţ	1	1 1	,			1 1	1	1	The second secon	ω 4 ≺ <del>≻</del>	5 1		1	1				ı	,



## WIND FORCES TO SHEAR LINES:

2ND LEVEL

ROOF

SHEAR LINE	FORCES											NEC AND DESCRIPTIONS	and the factor of the factor o	remain in particular and the		
LINE	F#	and the second	it belgeren er er	FORCE	WIDTH	X BA	R	L		FACTORS			% LOAD E	FFECT	National Resident	
1	F1	_	(	190.11 plf	) x 24.00	x 1.0	) /	2.00	х	1,00 /	1.00	Х	1.00	/ 1.00	=	2281.36 lb
. +		_	ì	, , , , , , , , , , , , , , , , , , ,	) x 0.00	x 0.0		0.00	x	1.00 /	1.00	х	1.00	/ 1.00	=	0.00 lb
+	_	_	ì		) x 0.00	x 0.0		0.00	X	1.00 /	1.00	X		/ 1.00	=	0.00 lb
<u>.</u>	_	_	,		) x 0.00	x 0.0		0.00	×	1.00 /	1.00	x		/ 1.00		0.00 lb
		_	,		) x 0.00	x 0.0		0.00	×	1.00 /	1.00	×	1.00	/ 1.00		0.00 lb
	-	-	,		) x 0.00	x 0.00		0.00	×	1.00 /	1.00	x		/ 1.00		0.00 lb
Ţ.	-	-	1	,	) x 0.00	0.0		0.00	x	1,00 /	1.00	x		/ 1.00		0.00 lb
+	-			(	) ×	0.0	, ,	0.00	^	1,00 1	1.00	^	1.00	1.00		2281.36 lb
															===	2261.30 ID
SHEAR LINE	FORCES					**************************************		*************			O SECRETARIO DE CARROLINA DE CAR	veres:		**************************************		<del></del>
LINE	F#	Charles Control		FORCE	WIDTH	X BA	R	L	VECTOR NUMBER	FACTORS		(	% LOAD E	FFECT	THE CONTRACTOR	Am lama para transmissiones may consequence processor
2	F1	_	(	190.11 plf	) x 24.00	x 1.00	) /	2.00	х	1.00 /	1.00	х	1.00	/ 1.00	=	2281.36 lb
- +		_	ì		) x 0.00	x 0.00		0.00	X	1.00 /	1.00	×	1.00	/ 1.00	=	0.00 lb
		_	ì		) x 0.00	x 0.00		0.00	X	1.00 /	1.00	X	1.00	/ 1.00	=	0.00 lb
4		_	ì		) x 0.00	x 0.00		0.00	X	1.00 /	1.00	x		/ 1.00		0.00 lb
T	-	-	(		) x 0.00	x 0.00		0.00	×	1.00 /	1.00	x		/ 1.00		0.00 lb
7	-	-	(		,			0.00		1.00 /	1.00	X		/ 1.00 / 1.00	=	0.00 lb
+	-	-	(	,	) x 0.00				Х							
+	-			(	) x	0.00	, /	0.00	Х	1.80 /	1.00	X	1.00	/ 1.00	=-	0.00 lb
																2281.36 lb
SHEAR LINE	FORCES				ILLUNICONE PIECE DI LONGO CONTROLO PROPRIO POR LA CONTROLO POR LA CONTROLO POR LA CONTROLO POR LA CONTROLO POR			***************************************							***********	
LINE	F#	***************************************		FORCE	WIDTH	X BA	Ŕ	L	**********	FACTORS			% LOAD E	FFECT		
3	F2	_	1	190.11 plf	) x 24.00	x 1.00		2.00	х	1.00 /	1.00	х	1.00	/ 1.00	=	2281.36 lb
+	12		,	100.11 pii	) x 0.00	x 0.00		0.00	×	1.00 /	1.00	×		/ 1.00	=	0.00 lb
	-	-	(		) x 0.00	x 0.00		0.00	x	1.00 /	1.00	x		/ 1.00	=	0.00 lb
+	-	-	(		,			0.00		1.00 /	1.00	x		/ 1.00	<u>-</u>	0.00 lb
+	-	-	(		7				×							
+	-	-	(		) x 0.00	x 0.00		0.00	х	1.00 /	1.00	Х		/ 1.00	=	0,00 lb
+	-	-	(		) x 0.00	x 0.00		0.00	X	1.00 /	1.00	х		/ 1.00	=	0.00 lb
+	-			(	) x	0.00	) /	0.00	Х	1,00 /	1.00	Х	1.00	/ 1.00	=	dl 00.0
															_	2281.36 lb
SHEAR LINE	FORCES					- Williams of the Committee Committe		**********************	***************************************	<del>oraș (upuși anternatural de la c</del>	document to a special property of	orania mobbiet				
LINE	F#		фунородомансикалис	FORCE	WIDTH	X BA	R	L.		FACTORS	********	Ç	% LOAD E	FFECT		
4	F2	_	(	190.11 plf	) x 24.00	x 1.00	) /	2.00	х	1.00 /	1.00	х	1.00	/ 1.00	=	2281.36 lb
. +	F2	_	ì	190.11 plf	) x 5.50	x 26.7		24.00	×	1.00 /	1.00	х	1.00	/ 1.00	=	1165.44 lb
i	-	_	,	100.11 pii	) x 0.00	x 0.00		0.00	×	1.00 /	1.00	X		/ 1.00	=	0.00 lb
	-	_	(		) x 0.00	x 0.00		0.00	×	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	=	0.00 lb
+	-	-	(		,									/ 1.00	=	0.00 lb
+	-	-	(	,	) x 0.00	x 0.00		0.00	х		1.00	X				
+	-			(	) x	0.00	) /	0.00	х	1.00 /	1.00	Х	1.00	/ 1.00	=_	0.00 lb
															===	3446.80 lb
SHEAR LINE	FORCES			MANAGER STREET, STREET		***************************************			***************************************			Cathar-server		***		
LINE	F#	Market Comment		FORCE	WIDTH	Х ВА	R	L		FACTORS		9	6 LOAD E	FFECT	National Control	
-	- "	-	(		) x 0.00	x 0.00	1	0.00	х	1.00 /	1.00	х	1.00	/ 1.00	=	0.00 lb
+	-	_	ì		) x 0.00	x 0.00		0.00	х	1.00 <i>I</i>	1.00	х	1.00	/ 1.00	=	0.00 lb
·	_	_	7		) x 0.00	x 0.00		0.00	x	1,00 /	1.00	x		/ 1.00	Ξ	0.00 lb
+	-	-	,		•			0.00	X	1.00 /	1.00	x		/ 1.00	=	0.00 lb
	-	-	(		,							×		/ 1.00	=	0.00 lb
+	-	-	(		) x 0.00	x 0.00		0.00	X		1.00					
+	-	-	(		) x 0.00	x 0.00		0.00	×	1.00 /	1,00	Х	1.00	/ 1.00	=	0.00 lb
+	-			(	) x	0.00	/	0.00	х	1.00 <b>/</b>	1.00	Х	1.00	/ 1.00	=	0.00 lb
															_	0.00 lb
															_	



## WIND FORCES TO SHEAR LINES:

1ST LEVEL

2ND FLOOR

SHEAR LINE	FORCES		on a constant	- Charles Andre III - Addition III (1900)							*****	years and the second		-	***************************************			**************************************
LINE	F#	Secretary and the second	COLUMN TO SERVICE (COLUMN TO SER	FORCE		WIDTH	okuezennia win	X BAR	****	L		FACTORS			% LOAD	EFFECT	LIVE CONTRACTOR	
5	F3	_	(	380.23 plf	) x	24.00	x	1.00	1	2.00	х	1.00 /	1.00	х	1.00	/ 1.0	) =	4562.73 lb
+	-	F1	ì		2281.36 lb ) x	1.00	Х	1.00	1	1.00	x	1.00 /	1.00	х	1.00	/ 1.0	) =	2281.36 lb
+	-	-	ì		) x	0.00	x	0.00	1	0.00	x	1.00 /	1.00	х	1.00	/ 1.0	) =	0.00 lb
+	-	-	(		) x	0.00	×	0.00	1	0.00	х	1.00 /	1.00	х	1.00	/ 1.0	) =	0.00 lb
+	-	-	(		) x	0.00	x	0.00	1	0.00	х	1.00 /	1.00	Х	1.00	/ 1.0	= C	0.00 lb
+	-	-	(		) x	0.00	×	0.00	1	0.00	х	1.00 /	1.00	Х	1.00	/ 1.0		0.00 lb
+	-			(	) x			0.00	1	0.00	X	1.00 /	1.00	х	1.00	/ 1.0	) = _	0.00 lb
																	_	6844.09 lb
SHEAR LINE	EODOES						<del>(************************************</del>	***************************************			anatoar ara-		*********				-	
LINE	F#		***************************************	FORCE	Marie Control of Contr	WIDTH		X BAR		L		FACTORS		-	% LOAD	EFFECT	alester (Meditaries	ті тайтый ана інсерсовання на простава под простава под простава под простава под простава под простава под пр
6	F3	_	(	380.23 plf	) x	24.00	х	1.00	1	2.00	x	1.00 /	1.00	х	1.00	/ 1.0	) =	4562.73 lb
+	_	F2	į	•	2281.36 lb ) x	1.00	x	1.00	1	1.00	x	1.00 /	1.00	х	1.00	/ 1.0	) =	2281.36 lb
+	-	-	(		) x	0.00	x	0.00	1	0.00	X	1.00 /	1.00	Х	1.00	/ 1.0	) =	0.00 lb
+	_	-	(		) x	0.00	x	0.00	1	0.00	х	1.00 /	1.00	х	1.00	/ 1.0	) =	0.00 lb
+	-	-	(		) x	0.00	x	0.00	1	0.00	×	1.00 /	1.00	Х	1.00	/ 1.0	) =	0.00 lb
+	-	-	(		) x	0.00	x	0.00	1	0.00	x	1.00 /	1.00	Х	1.00	/ 1.0	) =	0.00 lb
+	-			(	) x			0.00	1	0.00	X	1.00 /	1.00	Х	1.00	/ 1.0	) =_	0.00 lb
																	_	6844.09 lb
SHEAR LINE	FORCES												SIMPLE CONTRACTOR	Contraction of the Contraction o			HIII/A/2299	and American contract the many for the Property and Property of the Child College
LINE	F#	2000 to 1000 to 2000 to 1000 to	in a management	FORCE		WIDTH	//////////////////////////////////////	X BAR	-	L.	***	FACTORS		(	6 LOAD I	EFFECT	CHACACULAR AND	1514 1414 1414 1414 1414 1414 1414 1414
7	F4	-	(	380.23 plf	) x	24.00	х	1.00	1	2.00	х	1.00 /	1.00	х	1.00	/ 1.00	) =	4562.73 lb
+	-	F3	į	•	2281.36 lb ) x	1.00	x	1.00	1	1.00	х	1.00 /	1.00	х	1.00	/ 1.00	) =	2281.36 lb
+	-	-	(		) x	0.00	×	0.00	1	0.00	x	1.00 /	1.00	х	1.00	/ 1.00	) =	0.00 lb
+	-	-	(		) x	0.00	×	0.00	1	0.00	x	1.00 /	1.00	Х	1.00	/ 1.00		0.00 lb
+	-	-	(		) x	0.00	×	0.00	1	0.00	х	1.00 /	1.00	Х	1.00	/ 1.00		0.00 lb
+	-	-	(		) x	0.00	×	0.00	1	0.00	×	1.00 /	1.00	Х	1.00	/ 1.00		0.00 lb
+	-			(	) x			0.00	/	0.00	х	1.00 /	1.00	Х	1.00	/ 1.00	) =_	0.00 lb
																	_	6844.09 lb
SHEAR LINE	FORCES	www.www.com	CONTRACTOR OF THE CO		THE CONTRACTOR AND ADDRESS OF THE CONTRACTOR AND ADDRESS OF THE CONTRACTOR ADDRESS OF THE CONTRA						**********		0-00-0-0-0-0-0-0-0-0-0-0-0-0-0-0-0-0-0	******				
LINE	F#		· · · · · · · · · · · · · · · · · · ·	FORCE		WIDTH		X BAR	***********************************	L		FACTORS		C,	6 LOAD I	EFFECT	***************************************	
8	F4	-	(	380.23 plf	) x	24.00	х	1.00	1	2.00	х	1.00 /	1.00	Х	1.00	/ 1.00		4562.73 lb
+	F5	-	(	380.23 plf	) x	5.50	Х	26.75	1	24.00	Х	1.00 /	1.00	х	1.00	/ 1.00		2330.87 lb
+	-	F4	(		3446.80 lb ) x	1.00	Х	1.00	1	1.00	X	1.00 /	1.00	х	1.00	/ 1.00		3446.80 lb
+	-	-	(		) x	0.00	Х	0.00	1	0.00	х	1.00 /	1.00	х	1.00	/ 1.00		0.00 lb
+	-	-	(		) x	0.00	X	0.00	1	0.00	X	1.00 /	1.00	х	1.00	/ 1.00		0.00 lb
+	-	-	(		) x	0.00	X	0.00	1	0.00	Х	1.00 /	1.00	Х	1.00	/ 1.00		0.00 lb
+	-			(	) x			0.00	/	0.00	×	1.00 /	1.00	х	1.00	/ 1.00	) =_	0.00 lb
																	_	10340.41 lb
SHEAR LINE	FORCES	<del>Discussion</del>		Pro the second transfer the second transfer to the second transfer transfer to the second transfer transfe			<del></del>		************		50 eq ======			n friedrich der				
LINE	F#	· · · · · · · · · · · · · · · · · · ·	**********	FORCE		WIDTH		X BAR		L	<del></del>	FACTORS	***************************************		6 LOAD I		Pardinkski Paketon	maga ayan da ayan ayan ayan ayan ayan ayan
-		-	(		) x	0.00	Х	0,00	/	0.00	х	1.00 /	1.00	Х	1.00	/ 1.00		0.00 lb
+	•	-	(		) ×	0.00	Х	0.00	/	0.00	х	1.00 /	1,00	х	1.00	/ 1.00		0.00 lb
+	-	-	(		) x	0.00	Х	0.00	1	0.00	X	1.00 /	1.00	Х	1.00	/ 1.00		0.00 lb
+	~	-	(		) x	0.00	Х	0.00	1	0.00	×	1.00 /	1.00	Х	1.00	/ 1.00		0.00 lb
+	-	-	(		) x	0.00	X	0,00	,	0.00	X	1,00 /	1.00	х	1.00	/ 1.00		0.00 lb
+	-	-	(		) x	0.00	x	0.00	1,	0.00	X	1.00 /	1.00	X	1.00	/ 1.00		0.00 lb
+	-			(	) x			0.00	,	0.00	Х	1.00 /	1.00	х	1.00	/ 1.00		0.00 lb 0.00 lb
																	-	U. UU. II

## WIND FORCE SUMMARY:

ALLOWABLE STRESS DESIGN SHEAR FORCE SUMMARY:

ALLOWAE	Transmiss Change of Control	The same of the sa	R FORCE SUMMAR	Y:		NOTEN STATE OF	V WIND (ASD)
<b>F</b>	LINE		D (STRENGTH)	<b>├</b>	φ Λ 60	-	
	-		0.00 lb	X	0.60	=	0.00 lb 0.00 lb
	_		0.00 lb	X	0.60 0.60	=	0.00 lb
	1	=	0.00 lb 0.00 lb	X	0.60	=	0.00 lb
	-	_	0.00 lb	8	0.60	=	0.00 lb
1			0.00 lb	X	0.60	=	0.00 lb
l		=	0.00 lb	×	0.60	=	0.00 lb
		=	0.00 lb	î	0.60	=	0.00 lb
1	Ĭ _	=	0.00 lb	x	0.60	=	0.00 lb
'	١.	=	0.00 lb	×	0.60	=	0.00 lb
	I _	=	0.00 lb	x	0.60	=	0.00 lb
	l -	=	0.00 lb	Х	0.60	=	0.00 lb
	-	=	0.00 lb	х	0.60	=	0.00 lb
	-	=	dl 00.0	х	0.60	=	0.00 lb
	-	=	0.00 lb	х	0.60	=	0.00 lb
	-	=	0.00 lb	х	0.60	=	0.00 lb
	-	=	0.00 lb	х	0.60	=	0.00 lb
L	<u> </u>	=	0.00 lb	Х	0.60	=	0.00 lb
	Ī -	==	0.00 lb	Х	0.60	=	0.00 lb
	-	=	0.00 lb	х	0.60	=	0.00 lb
	-	=	0.00 lb	х	0.60	=	0.00 lb
	-	=	0.00 lb	х	0.60	=	0.00 lb
	] -	=	0.00 lb	Х	0.60	=	0.00 lb
	-	=	0.00 lb	Х	0.60	=	dl 00.0
l	-	=	0.00 lb	х	0,60	=	0.00 lb
	-	=	0.00 lb	Х	0.60	=	0.00 lb
	-	<b>=</b>	0.00 lb	Х	0.60	=	0.00 lb
	-	=	0.00 lb	X	0.60	=	0.00 lb
	-	<b>=</b>	0.00 lb	Х	0.60	=	0.00 lb
	-	=	0.00 lb	Х	0.60	=	0.00 lb
	-	=	0.00 lb	Х	0.60	=	0.00 lb
	-	=	dl 00.0 dl 00.0	X	0.60 0.60	=	0.00 lb 0.00 lb
	-	=	0.00 lb	×	0.60	=	0.00 lb
	Ī.	=	0.00 lb	×	0.60	=	0.00 lb
	I -	=	0.00 lb	x	0.60	=	0.00 lb
	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	х	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	х	0.60	=	0.00 lb
ROOF	-	=	0.00 lb	х	0.60	=	0.00 lb
2	-	=	0.00 lb	х	0.60	=	0.00 lb
_	-	=	0.00 lb	х	0.60	=	0.00 lb
	-	=	0.00 lb	х	0.60	=	0.00 lb
	-	=	0.00 lb	х	0.60	=	0.00 lb
	-	=	0.00 lb	х	0.60	=	0.00 lb
	-	=	0.00 lb	Х	0.60	=	0.00 lb
	-	<b>=</b>	0.00 lb	Х	0.60	=	0.00 lb
	l -	=	0.00 lb	Х	0.60	=	0.00 lb
***************************************	-	=	0.00 lb	X	0.60		0.00 lb
	5	=	6844.09 lb	X	0.60	=	4106.46 lb 4106.46 lb
	6 7	=	6844.09 lb 6844.09 lb	X	0.60 0.60	=	4106.46 lb
	8	=	10340.41 lb	X	0.60	=	6204.24 lb
	_	=	0.00 lb	×	0.60	=	0.00 lb
	_	=	0.00 lb	×	0.60	=	0.00 lb
	_	=	0.00 lb	x	0.60	=	0.00 lb
	l -	=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	Х	0.60	=	0.00 lb
2ND FLOOR	-	=	0.00 lb	Х	0.60	=	0.00 lb
Š	-	=	0.00 lb	×	0.60	=	0.00 lb
) F	-	=	0.00 lb	х	0.60	=	0.00 lb
Ž.	-	=	0.00 lb	х	0.60	=	0.00 lb
N	-	=	0.00 lb	х	0.60	=	0.00 lb
	-	=	0.00 lb	х	0.60	=	0.00 lb
	-	=	0.00 lb	х	0.60	=	0.00 lb
		=	0.00 lb	×	0.60	=	0.00 lb
	ı			х	0.60	=	0.00 lb
	-	=	0.00 lb	9			
	-	=	0.00 lb	х	0.60	=	0.00 lb
	- - -	=	0.00 lb 0.00 lb	x x	0.60 0.60	=	0.00 lb 0.00 lb
	- - -	=	0.00 lb	х	0.60		0.00 lb

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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)<sup>1,2</sup>

	Framing	M	inimum Facing Connection		Shear Strengt
Spline Type <sup>3</sup>	Minimum SG <sup>4</sup>	Chord <sup>2</sup>	Plate <sup>2</sup>	Spline <sup>3</sup>	h <sup>5</sup> (plf)
	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
	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
Block, Surface, or Lumber Spline (Type S, Type L)	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) 0.413" x 2-3/8" nails 3" on center Staggered (3/8" edge distance and 3/4" edge distance)	720
	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

<sup>1</sup> 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)

<sup>2</sup> Chords, hold-downs and connections to other structural elements must be designed by a registered design professional in accordance with accepted engineering practice.

<sup>3</sup> Spline type at interior panel-to-panel joints only, solid chord members are required at each end of each shear wall segment.

<sup>4</sup> 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.

<sup>5</sup> 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.

OSE FOR THE SIGN NEW PO SIP G = 16 SIP 3 = 16 SIP 2 = 16

This listing report is intended to indicate that NTA, Inc. has evaluated the product described and found it to be eligible for labeling. Product not labeled as specified herein is not covered by this report. NTA, Inc. makes no warranty, either expressed or implied, regarding the product covered by this report.





1,2,4,2,16   5,00 ft	LINE VS	VSTRENGTH	A	8	ပ	٥	Е	ıL	9	I	LENGTH	v ASD = 0.7 V / L	TYPE	CAPACITY	I	M	M/H	2 W/H	2 W / H AD III STED CAPACI	VEION
1,224,24   2,50 ft + 7,00 ft + ft	- Character	4122.42 lb	+ H 00'9	5.00 ft +	4.00 ft +	##	ft +	ff+		42		192.38 plf	Sipe	360.00	10.00	4 00	. 13		200 000	
3774.28 lb         7.00 ft	2	4122.42 lb	5.50 ft +	7.00 ft +	##	##	##	##		#	12.50 ft		Sibe	360.00	10.00	25.50			200.00	
464.77 lb         3.50 ft	7157	3074.28 lb	7.00 ft +	##	+#	##	+ 4=	##		4	7 00 #		SIDE	360.00	10.00	200	1.02		360.00	
478570 b 1350 ft		4644.77 lb	3.50 ft +		+	+	+#	##	+	#	7.00 #		CID	920.00	10.00	2,50	30.0	0.70	300.00	
4785.70 lb         13.50 ft         ## ft		4785.70 lb	13.50 ft+		##	##	##	##	+#	#	= 13.50 ft	123	Sipe	360.00	000	13.50	0.00	0.70	380.00	
370848 lb 24,00 ft		4785.70 lb	13.50 ft+	##	+ #	##	+#	##	##	: #	= 13.50 ft	248 15 nif	Sibe	360.00	0000	12.50	0.07		360.00	
5353.03 lb		3708.48 lb	24.00 ft +	ff+	+#	##	<b>#</b>	+#	##	4	= 24 00 <del>ft</del>	108 16 off	Sibe	360.00	000	24.00	0.00		200.00	
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# WIND SHEAR FORCES TO SHEAR LINES AND ALLOWABLE STRESS SHEAR WALL FORCES:

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AD HISTED CABACITY	260 00 216	390.00 pil	360.00 pit	360.00 plf	920 00 nlf	360 00 plf	36000 35000	360.00	300.00 pil	920.00 pil										
H / /// < 3.5.	***************************************	7.00	1.82	1.43	2.86	0.87	78.0	000	0.36	5.0										
//		00.4	00.0	7.00	3.50	13.50	2 2 2	27.00	25.50	9										
Ţ	ı								8 8											
CAPACITY	360.00	360.00	390.00	360.00	920.00	360.00	360.00	360.00	920.02											
TYPE	SIDA	SIDE	o Lio	SIP6	SIP2	SIP6	Sipe	edis:	SIBS	ı		,	ı		,	,	,		,	
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ENGTH	15.00 ft	12 50 ft	12:00	1.00.1	7.00 ft	13.50 ft	13.50 ft	24.00 ft	7 00 ft	<b>4</b> =	: 423	: <b>d</b> =	: <b>4</b> =	<b>#</b>	¥	#	<b>#</b>	#	4=	#
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В	5.00 ft +	7.00 ft +	4		3.50 # +	+ #	+ #	##	3.50 ft +	<b>+</b> ₩	+#	+#	##	+#	+ #	+ #	<b>+</b> <b>#</b>	##	+#	<b>+</b> ₩
A	6.00 ft +	5.50 ft +	7 00 #	200.7	3,50 11 +	13.50 ft +	13.50 ft +	24.00 ft +	3.50 # +	##	<del>+</del> #	##	##	+ #	#	##	##	<b>+</b> ₩	+	##
SD	1368.82 lb	1368.82 lb	1368 82 lb	20,000	ZU08.U8 ID	4106.46 lb	4106.46 lb	4106.46 lb	6204.24 lb	Q	ପ	Ω	Q	Q	a	Ф	Q	Q	മ	മ



# STRAP HOLD DOWN CAPACITY SCHEDULE:

UPLIFT CAPACITY IS BASED ON THE MINIMUM RATING x 0.75 SIMPSON STRAPS

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TOLD DOVEN	うとこくと りょう	FOSI	וווא ווופ	LA KEPOKI	ICC REPORT	LA REPORT ICC REPORT LARR 25910 25% REDUCTION MAX DEFLECTION (NAIL SIZE	MAX DEFLECTION		TOTAL NAILS
2000									
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1	:								
MS137	2710.00 lbs	* <del>*</del>	2710.00 lbs	RR25713	ESR-2105	2032,50 lbs	0.030	164	22
MSTAB	4205 00 the	*	420E OO 1bc	00000	2010	7 27 27	000		
	1200.0031	ź	4200.00.103	51 15744	CO17-NO3	S01 C / SC1 C	0.030		34
MST60	4605.00 lbs	<del>*</del>	4605.00 lbs	RR25713	ESR-2105	3453.75 lbs	0.030	16d	34
MST72	8505 00 the	~	SEOF OO INC	0005740	מינים מינים	1070 75 16-	0		. (
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CMST12-42	10710.00 lbs	4 <del>,</del>	9215.00 lbs	RR25713	ESR-2105	6911.25 lbs	0 030	160	84
CMST144.33	7755 00 lbs	*	6490 00 lbs	DD05712	2000	2007	000	3 7	- (
	000	<b>5</b>	20.001	01/07/11/1	2017	SOI 00: /00t	0.00	no-	00
CS16-11	2080.00 lbs	*4 *	1705.00 lbs	RR25713	ESR-2105	1278.75 lbs	0.030	10d	22
					4.				
*******			*						
Contraction		-			*				

POST ABOVE	HOLD DOWN STRAP	# NAILS ABV.	16" MAX STAP CLEAR SPAN	# NAILS BEL.	POST BELOW

MST STRAP CALCULATION	ULATION			
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 BAS	E ON 2012	NAIL VALUES BASE ON 2012 NDS 11P - 16d w/ G=0.5	3.	

4425 CANOGA AVENUE WOODLAND HILLS CA 91364 PHONE (818) 347-7008 FAX (818) 883-8869 6/27/2017:2017 SEISMIC AND WIND DESIGN 2016 CBC.xlsm:STRAPS



## HOLD DOWN CAPACITY SCHEDULE:

HOLDOWN CAPACITIES BASED ON CODE TABLES AND RESEARCH REPORTS

## SIMPSON HOLD DOWN CAPACITIES

SIMPSON	POST	ANCHOR	CATALOG CAPACITY	da	ICC	ICC CAPACITY	ICC STRENGTH	da	LARR	75% CAP.
HOLD DOWN	SIZE	DIAMETER	lbs	in	REPORT	lbs	lbs	in	REPORT	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

1712220111011101	••••••									
SIMPSON	ANCHOR	ANCHOR	CAPACITY	de	F	ICC	ICC CAPACITY	ICC STRENGTH	LARR	LARR CAP
ANCHOR	BOLT	DIAMETER	lbs	in	in	REPORT	lbs	lbs	REPORT	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.		Manager and the second	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.		1	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.	l .		-	-

## HOLD DOWN SUMMARY

SIMPSON	POST	ANCHOR	ANCHOR	75% ASSEMBLY CAPACITY
HOLD DOWN	SIZE	LA CITY	ALTERNATE	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	ds
lbs	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



SEISMIC OVERTURNING DESIGN:
ASD LEVEL FORCES
USE EQUATION ASCE 7:41 12.4.2.3 #8
(0.6 - 0.14 Sds) D + 0.7 p Qe
Sds = 0.633
0.6 - 0.14 Sds = 0.633
USE THE LESSER OF TCC-ES AND LARR VALUES FOR HOLD DOWNS... CAPACITY IS BASED ON 0.75 x HOLD DOWN ALLOWABLE STRESS DESIGN CAPACITY

LINE#	LENGTH	T HEIGHT	ADJUSTED ASD	OVERTURNING	0.7 p Qe	WEIGHT	ADD LOAD AL	LOAD ADD LOAD (0.6-0.14Sds) D RST (	14Sds) D RST	OVERTURNING +/- LEVER ARM ADDED UPLIFT WALL	· LEVER ARM	ADDED UPLIF		NET UPLIFT	HOLDOWN	CAPACITY
##	#	₽	plf	ft-lb	q	plf	þlf	Q	ft-lb	ft-lb	Ħ	Q	ABV.	Q	TYPE	q
1 A	6.00	10.00	192.38	11542.8	1923.8	150.0	370.0		4786.8	6756.0	-0.50		1	1228 4	MST37	2032 5
1 8	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	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
8	7.00	10.00	307.43	21519.9	3074.3	150.0	50.0		2505.9	19014.0	-0.50		,	2925.2	MSTAR	3153.8
4 A	3.50	10.00	464.48	16258.7	4644.8	150.0	100.0		783.1	15473.6	-0.50		,	5157.9	CMST12-45	69113
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	60113
5 A	13.50	9.00	55.77	6775.8	501.9	100.0	100.0		9320.5	-2544 7	-0.50	1228 36	14	1032 6	HDOR CHES	65580
9 9	13.50	9.00	17.29	2101.0	155.6	100.0	100.0		9320.5	-7219.5	-0.50	2152.66	DA.	1597.3	HDO8-SDS3	6558.0
7 A	24.00	9.00	-199.26	-43041.0	-1793.4	100.0	100.0	S18	29457.4	-72498.4	-0.50	2925 23	3A	-159.8	HDO8-SDS3	6558.0
89 W	3.50	9.00	70.83	2231.0	637.4	100.0	100.0		626.5	1604.5	-0.50	5157.87	44	5692 7	HHD014-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	48	5692.7	HHDQ14-SDS2.5	10282.5



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

CAPACITY	ಥ	2032 50	2032 50	2032.50	3153.75	3153.75	3153.75	6911.25	6911.25	6558.00	6558.00	6558.00	10282.50	10282.50								
HOLDOWN	TYPE	MST37	MST37	MST37	MST48	MST48	MST48	CMST12-45	CMST12-45	HDQ8-SDS3	HDQ8-SDS3	HDQ8-SDS3	HHDQ14-SDS2.5	HHDQ14-SDS2.5			•		•	•	,	
NET UPLIFT	٩	-25.59	147 27	700 05	750.81	726.98	1653,57	3140.55	3140.55	1148.90	1729.13	41.74	9099.80	9099.80								
WALL	ABV.	-		,	,	1	1	,		4	2A	3A	4 4	4B	1	•	1	,	ı	1	ι	
WEIGHT ADD LOAD ADD LOAD 0.6 D RST OVERTURNING+/- LEVER ARMADDED UPLIFT	Q	Annual des des la company de l									750.81	1653.57	3140.55	3140.55								
- LEVER ARM	₽	-0.50	-0.50	-0.50	-0,50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50								
ERTURNING +	ft-lp	-140.72	662.73	2450.18	3754.05	4725.39	10748.19	9421.66	9421.66	14935.68	12718.19	39839.73	17877.73	17877.73								
.6 D RST OVE	ft-lb	5616.00						918.75				•		_								
ADD LOAD (	q																					
ADD LOAD	plf	370.0	370.0	100.0	100.0	50.0	50.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0								
WEIGHT	plf	150.00	150.00	150.00	150.00	150.00	150.00	150.00	150,00	100.00	100.00	100.00	100.00	100.00								
} :	q	912.55	912.55	912.55	1095.06	1095.06	1955.46	2954.40	2954.40	1916.35	1752.09	-219.99	5317.92	5317.92								
ASD FORCE OVERTURNING	q:-1	5475.28	4562.73	3650,18	6022.80	7665.39	13688.19	10340.41	10340.41	25870.68	23653.19	-5279.73	18612.73	18612.73								
ASD FORCE	pit	91.25	91.25	91.25	109.51	109.51	195.55	295.44	295.44	212.93	194.68	-24.44	590.88	590.88								
- E	######################################	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	9.00	9.00	9.00	9.00	00.6								
_	Approximation of the second	∢	Δ	ပ	⋖	Ω	⋖	∢	Ω	⋖	∢	⋖	¥	Ω								
INE # CENGIN	μ	6.00	5.00	4.00	5.50	7.00	7.00	3.50	3.50	13.50	13.50	24.00	3.50	3.50								
L I	#	<del>-</del>	τ-	-	7	7	ო	4	4	2	ဖ	7	ω	∞								





PER ASCE 7-10 12.3.4.1  $\,\rho$  = 1.0 FOR DRIFT CALCULATIONS - ADJUST v ASD BY 1 /  $\rho$ 

o constant			National Park												
AIDADE		.⊆	00 %	9 6	8 8	3 8	3 6	8 6	3 6	8 6	2.00	2,5	2 1 0	27.0	2.70
C	) } %	Ś. <u>e</u>	134	5 6	0. 4	7.7	5 5	177	7.1	2 17	7 20		9 9	2 6	2.40
THE CONTRACTOR OF THE PERSONS	2	3	- V	) C	0 0	) (	0.5	) C	0 0		7.0	) C	0 0	2 0	4.0
SECONDO DE LA COMPOSITION DEL COMPOSITION DE LA	d X	2. ⊆	336	37.7	177	303	351	436	243	543	263	360			0.599
	ds x h/b	-	4												0.363
Average Contract of the Contra	^	5													
	vh/1000Gz	.5	0.159	0 150	0.150	191	9 6	0.255	0.208	0.208	0.185	0.185	0.08	0.24	0.216
teromostoccostoccost	8vh^3 /Fab	Ę	0.013	0.015	0.019	0.016	0.013	0.017	0.052	0.052	0000	0000	0.001	0.013	0.013
	3 SHRINK	.⊑	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050
ONGATION	IS CRUSH	<u>.</u> ⊆	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020
'ERTICAL E	ds SLIP	2.	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010
2	e O		13.0	13.0	13.0	13.0	13.0	13.0	24.0	24.0	13.0	13.0	13.0	24.0	24.0
USE	ds D/C	. <u>.</u>	0.014	0.016	0.022	0.016	0.015	0.021	0.017	0.017	0.016	0.024		0.061	0.061
	sp	.⊑	0.030	0.030	0.030	0.030	0.030	0.030	0.030	0.030	0.130	0.130	0.130	0.144	0.144
HOLDOWN	D/C	RATIO	0.4649	0.5294	0.7215	0.5251	0.5124	0.7135	0.5741	0.5741	0.1211	0.1874	-0.0187	0.4259	0.4259
	TaHD	Ω	2033	2033	2033	3154	3154	3154	6911	6911	6558	6558	6558	10283	10283
ASE	T HD / p	Q	945	1076	1466	1656	1616	2250	3968	3968	794	1229	-123	4379	4379
	A POST	in^2	12.25	12.25	12.25	12.25	12.25	12.25	12.25	12.25	19.25	19.25	19.25	39.88	39.88
	HOLDOWN	TYPE	MST37	MST37	MST37	MST48	MST48	MST48	CMST12-45	CMST12-45	HDQ8-SDS3	HDQ8-SDS3	HDQ8-SDS3	HHDQ14-SDS2.5	HHDQ14-SDS2.5
STRENGTH	1.4 v / p	plf	207.2	207.2	207.2	248.6	248.6	331.1	500.2	500.2	267.2	267.2	116.5	576.5	576.5
	v ASD	plf	192.4	192.4	192.4	230.9	230.9	307.4	464.5	464.5	248.1	248.1	108.2	535.3	535.3
	ΡĽ	TYPE	SIP6	SIP6	SIP6	SIP6	SIP6	SIP6	SIP2	SIP2	SIP6	SIP6	SIP6	SIP2	SIP2
		#													
	LENGTH	¥	6.00	5.00	4.00	5.50	7.00	7.00	3.50	3.50	13.50	13.50	24.00	3.50	3.50
	LINE	#	1	<b>-</b>	-	7	7	က	4	4	22	ဖ	7	80	œ



## **SHEAR WALL SUMMARY:**

COMPARE SEISMIC AND WIND REQUIREMENTS FOR FINAL RESULTS

INE	LENGTH	SEISMIC FORCE	WIND FORCE	GOVERNING FORCE	WALL TYPE	HOLD DOWN	WALL CAPAC	YTIC
#	ft	plf	plf	plf	PER SCHED.	SIMPSON	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	V
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	V
8	3.5	535.3	886.3	886.3	SIP2	HHDQ14-SDS2.5	920.0	V
8	3.5	535.3	886.3	886.3	SIP2	HHDQ14-SDS2.5	920.0	V
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## 

0.6 - 0.14 Sds Ωo

= 0.633 = 0.511 = 3.00

OVERSTRENGTH	(0.6 - 0.14 Sds) D + Ωo Qe	sql	39728	4194 1	4781.5	5473.4	53517	7254.7	12244.1	12244.1	4458.6	5129.4	1774.6	13751.4	13751.4
	· WALL	ABV.	Accompanies of the Contraction o	1	,	,			1		14	ZA	œ S	₹	4B
	ADDED UPLIFT	<u>Q</u>	STANDS OF STREET, STANDS OF STANDS OF STREET, STANDS OF STREET, STANDS OF ST								3972.79	5473,43	7254.69	12244.13	12244.13
	+/- LEVER ARM	¥	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50
	NET O.T.	ft-lb	21850,4	18873,5	16735.3	27367.1	34786.2	47155.5	36732.4	36732.4	6316.0	-4472.1	-128782.7	4522.0	4522.0
	(0.6-0.14Sds) D RST	ft-10	4786.8	3324.2	1022.8	1933.8	2505.9	2505.9	783.1	783.1	9320.5	9320.5	29457.4	626.5	626.5
	AD ADD LOAD	മ												_	
National Section (Section 1977)	ADD LO	þlí	370.0	370.0	100.0	100.0	50.0	50.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
	WEIGHT	plf	150.0	150.0	150.0	150.0	150.0	150.0	150.0	150.0	100.0	100.0	100.0	100.0	100.0
	ದಿಂ Qe	ql	10245.1	10245.1	10245.1	12294.1	12294.1	16371.9	24735.5	24735.5	2672.9	828.8	-9550.5	3394.6	3394.6
	ASD to OVERTURNING	ft-lb	26637.2	22197.7	17758.1	29300.9	37292.1	49661.4	37515.5	37515.5	15636.5	4848.4	-99325.3	5148.5	5148.5
	ADJUSTED V ASD	plf	192.38	192.38	192.38	230.86	230.86	307.43	464.48	464.48	55.77	17.29	-199.26	70.83	70.83
	HEIGH	ft	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	9.00	9.00	9.00	9.00	9.00
	LENGIH	ft	6.00	5.00	4.00	5.50	7.00	7.00	3.50	3.50	13.50	13.50	24.00	3.50	3.50
11 Total Contract Con	#	***************************************	4	Ω	ပ	∢	മ	∢	∢	Ω	⋖	∢	∢	∢	മ
VINDENDESCO	Z Z	#	~	_	~	2	7	ო	4	4	2	ဖ	7	∞	∞

1 1 1 1 1

## 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 3433 SHEET 3 OF

DINHRAGMS:	18 Mr.
Fx = 264 } 264 PLF	12ª Mr.
FUROSE 1978 1978	24' 6'
STREWETH SEISMIC DE	
Duns = 0.7 (264)24/2(20) ASP = 93PLF	Dus Rook
ROSE	1 Ma
STRONGTH WIND T	
Fx = 43}300 PLF	ra' M
F = 41 7 300 PLF ()	24'
) = 0.6(3-80)24/ MIX 2(24) SEP = 114-PLF 2ND FLOR	Vis Vo Fron
ROOF: THE 1/2 COX /03	B W/ Blech = - CAP = 1 to 12

4425 CANOGA AVENUE WOODLAND HILLS ČA 91364 PHONE (818) 347-7008 FAX (818) 883-8869 Info@RichmondHoffmayer.com

## RICHMOND HOFFMAYER INC. STRUCTURAL ENGINEERING



PROJECT VILLAGE NEST JOB 2017-0610

CLIENT 
ADDRESS EDEN, UT

DESIGN HUUM

ENGINEER J.H. DATE 6/14/2017

RICHAND PE 44628 HOPFMAYER SE 5495 SHEET OF

PLY Y/SNOW LOAD:  SPAN = 16" SL  WS = LL4 PSF  PEN APPA  REDID SPAN RATING  A9/20	FALE JULY S GRAIN ACLOSS SOPPERS
Co=1.16 -> 264/1.16	MAX (BENDING GOVERNS) = 229 PSF & 293 PSF OX
ROSE REQUIES & 5/	100 e 6,12

## RICHMOND HOFFMAYER INC. STRUCTURAL ENGINEERING



PROJECT VILLAGE NEST JOB 2017-0610

CLIENT ADDRESS EDEN, UT

DESIGN HUUM

ENGINEER J.H. BACK-POINT PE 44628 HOPPMAYER SE 5435 SHEET SG OF

CHORDS & DRAGS:
MASS CASAD => (0.6(300) 29/6)/2= 6-84 11
USE NOLE SIG TO PLOTE  WI LSTAIR STOPP - UPP = 795 16
OPTION:
CHORDS & DRESS:  MANN CAND => (0.6(380) 29/e)/a = 6-84     USE    SINGLE SIC TO BLOODE = 795     USE    NA    OPTION:  VI SINGLE 2x TO PLATE  VI 1,3E RIM BODRO - SEE PG # 157
PEQUILES
SEE PLOTE STUKE, PG # 22-8





 PROJECT
 FELDSPAR LOWER
 JOB
 2017-0610

 CLIENT
 VILLAGE NEST

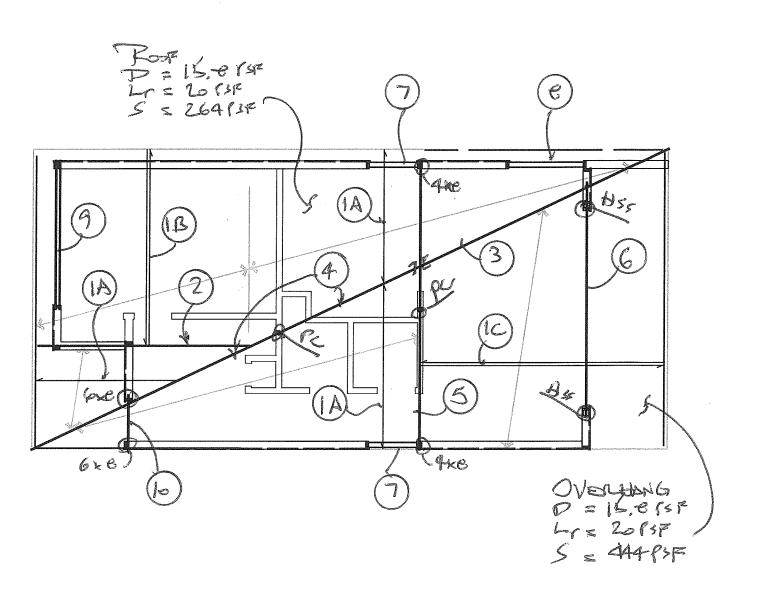
 ADDRESS
 EDEN, UTAH

 DESIGN
 HUUM

 ENGINEER
 J.H.

 RICHMOND PE 49628
 HOFFMAYER SE 3935

 SHEET
 37 OF



ROOF FRAMING PLAN
FELDSPAR - UPPER
NTS

O: BEEN &



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

Project ID: 2017-0610

138

**Wood Beam** 

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Lic. # : KW-06002886

Description: B1A - RAFTERS

## **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

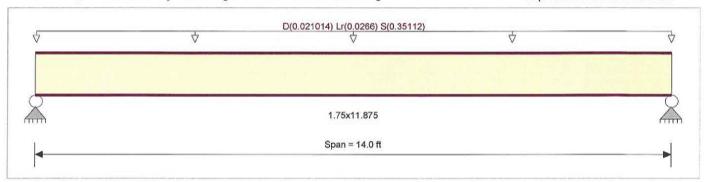
Load Combination Set: ASCE 7-10

## **Material Properties**

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

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

Density 42.0 pcf Repetitive Member Stress Increase



## **Applied Loads**

Max Downward Total Deflection

Max Upward Total Deflection

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		1.75×11.875	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 Deflec	tion	0.658 in Ratio	= 255>=240.		
Max Upward Transient Deflection	n	0.000 in Ratio	o = 0 < 240.0		

240>=240.

0<240.0

Vertical Reactions		Values in KIPS	
Load Combination	Support 1	Support 2	
Overall MAXimum	2.605	2.605	

0.697 in Ratio =

0.000 in Ratio =

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



Project Title: Engineer: Project Descr: VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

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**Wood Beam** 

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**B1B-RAFTERS** 

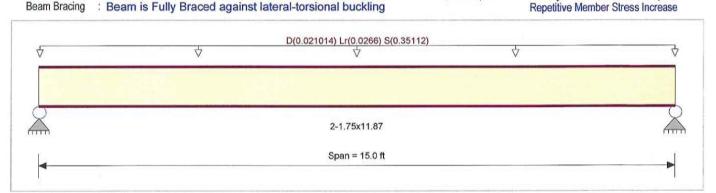
## **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set : ASCE 7-10

## **Material Properties**

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



## **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	=	14.015ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection					
Max Downward Transient Deflect	ion	0.434 in Ratio	= 415>=240.		
Max Upward Transient Deflection	greenen E	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	V		



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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**Wood Beam** 

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Lic. #: KW-06002886 Description: B1C - F

**B1C - RAFTERS w OVERHANG** 

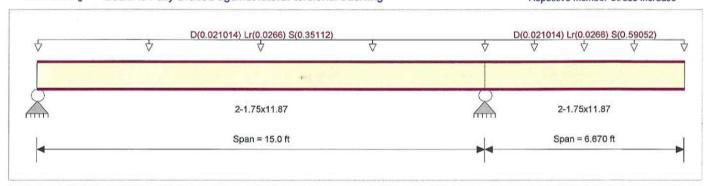
#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

A STATE OF THE PROPERTY OF THE				
Analysis Method : Allowable Stress Design	Fb+	2,600.0 psi	E : Modulus of Elasti	city
Load Combination :ASCE 7-10	Fb-	2,600.0 psi	Ebend- xx	1,900.0ksi
	Fc - Prll	2,510.0 psi	Eminbend - xx	965.71ksi
Wood Species : Trus Joist	Fc - Perp	750.0 psi		
Wood Grade : MicroLam LVL 1.9 E	Fv	285.0 psi		
Trood Grado . Milorocam EVE 1.0 E	Ft	1,555.0 psi	Density	42.0pcf
Beam Bracing : Beam is Fully Braced against lateral-to	rsional buckling	A N		or Stress Increase



#### **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 Section used for this span	=	0.577. 1 2-1.75x11.87	Maximum Shear Stress Ratio Section used for this span	=	0.384 : 1 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 Location of maximum on span Span # where maximum occurs	=	+D+S 15.000ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	= =	+D+S 15.000 ft Span # 1
Maximum Deflection Max Downward Transient Deflect Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection		0.474 in Rat -0.046 in Rat 0.475 in Rat -0.044 in Rat	tio = 3444>=240. tio = 336>=240.		

Vertical Reactions		Sup	pport 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			



VILLAGE NEST

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

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**Wood Beam** 

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ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29 Literisee: RICHMOND HOFFMAYER

Lic. # : KW-06002886 Description:

**B1C - RAFTERS w OVERHANG** 

Support notation: Far left is #1 **Vertical Reactions** Values in KIPS Load Combination Support 3

Support 1 Support 2 +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) S Only 0.160 0.416 1.758 7.448



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

Design OK

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**Wood Beam** 

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Lic. #: KW-06002886

Licensee: RICHMOND HOFFMAYER

Description: **B2-ROOF BEAM** 

#### **CODE REFERENCES**

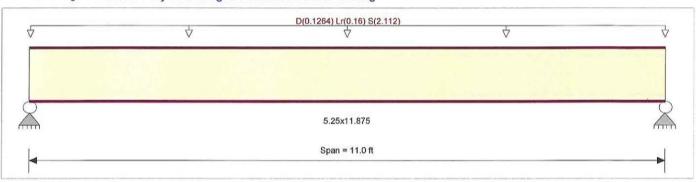
Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

Analysis Method: Allowable Stress Design E: Modulus of Elasticity Fb+ 2,900.0 psi 2,000.0ksi Load Combination :ASCE 7-10 Fb -2,900.0 psi Ebend- xx Fc - Prll 2,900.0 psi Eminbend - xx 1,016.54ksi Fc - Perp 625.0 psi Wood Species : Trus Joist 290.0 psi : Parallam PSL 2.0E F۷ Wood Grade Ft 2,025.0 psi 45.050pcf Density

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)

1	D	Е	S	IG	N	S	U	MI	VI A	٩F	?)	1
		_	•			•	•				•	

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

Max Downward Transient Deflection 0.478 in Ratio = 276>=240. 0.000 in Ratio = Max Upward Transient Deflection 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	



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID:

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2017-0610

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Steel Beam

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Lic. #: KW-06002886 Description: B3 - R

**B3 - RIDGE AT OVERHANG** 

#### **CODE REFERENCES**

Calculations per AISC 360-10, IBC 2015, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

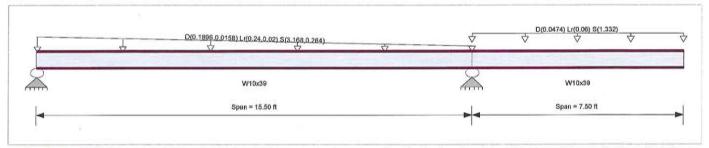
Analysis Method: Allowable Strength Design

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

Bending Axis: Major Axis Bending

Fy : Steel Yield : E: Modulus : 50.0 ksi

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

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 = Section used for this span Ma : Applied Mn / Omega : Allowable	0.351: 1 Ma	aximum Shear Stress Ratio =	0.253 : 1
	W10x39	Section used for this span	W10x39
	41.029 k-ft	Va : Applied	15.799 k
	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 Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	0.215 in Ratio = -0.041 in Ratio = 0.235 in Ratio = -0.049 in Ratio =	866>=360 4,422>=360 791>=180 3657>=180	

#### Maximum Forces & Stresses for Load Combinations

Load Combin	ation	ALIFE THE PARTY OF	Max Stress	Ratios		5	Summary of M	Ioment Valu	ies			Summa	ary of Sh	ear Values
	SCHOOL STATE OF THE STATE OF TH	Span#	М	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 =		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 Co	mb Run (*L)													
Dsgn. L =		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 Co	mb 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 Co	mb 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, L	L 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, l	L 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 = +D+0.750Lr. L	7.50 ft L Comb Run (LL)	2	0.021	0.010		-2.43	2.43	195.00	116.77	1.00	1.00	0.65	93.74	62.50



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

Steel Beam

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Lic. # : KW-06002886

Description: B3 - RIDGE AT OVERHANG

Load Combin	ation		Max Stress	Ratios			Summary of M	loment Valu	ies			Summa	ary of Sh	ear Values
Segmer	nt Length	Span #	M	٧	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 = +D+0.750S	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
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 = +0.60D		2	0.261	0.130		-30.53	30.53	195.00	116.77	1.00	1.00	8.14	93.74	62.50
	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. "-" Defi	Location in Span	Load Combination	Max. "+" Defi	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				
0.01	44.000	04.055				

21.955

14.633



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

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**Steel Beam** 

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Lic. #: KW-06002886 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

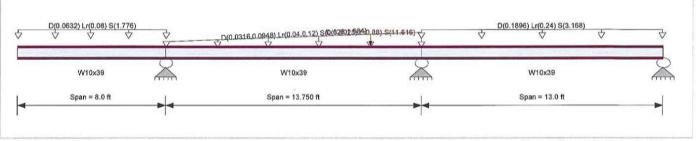
Beam is Fully Braced against lateral-torsional buckling Beam Bracing:

Major Axis Bending Bending Axis:

Fy: Steel Yield: E: Modulus :

50.0 ksi

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 = 0.4440 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	60.102k-ft	Va : Applied	25.661 k
Mn / Omega : Allowable	116.766k-ft	Vn/Omega : Allowable	62.496 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	8.000ft	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 Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	0.667 in Ratio -0.105 in Ratio 0.699 in Ratio -0.108 in Ratio	= 1,571 >=240. = 275 >=180	

#### Maximum Forces & Stresses for Load Combinations

Load Combination	1115 - 41	Max Stress	Ratios	V-171VS	5	Summary of M	loment Valu	ies			Summa	ary of Sh	ear Values
Segment Length	Span #	М	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only	12/	Seattle of	IA HAUNCI		AL ST.		000 000	1700A ETEA	INCOM.	196255	201945	20 = 44	100000
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**)						V 7 1 3 7 (2)	107070707						
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



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

## Steel Beam

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CONTRACTOR CONTRACTOR OF THE PROPERTY OF THE P	GE OVE	RENTRY							LIG	ensee	. KICHIVIC	וטה פואנ	daminated.
THE RESERVE AND THE PARTY OF TH			ss Ratios			Summary of M	Ioment Valu	201			Sumr	nary of Sh	ear Values
Load Combination Segment Length	Span #	M	V	· Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Ch	Rm	Va Max		Vnx/Omeg
Dsgn. L = 13.00 ft	3	0.030	0.027	3.47	-2.93	3.47	195.00	116.77		1.00	1.71	93.74	62.5
D+Lr, LL Comb Run (L*L)	9	0.000	0.027	0.47	-2.55	3.47	133.00	110.77	1.00	1.00	1.7.1	00.74	OZ.
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	62.5
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	62.5
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	62.5
D+Lr, LL Comb Run (LL*)	8		27.22		2723			7.72.22	0.00		122		
Dsgn. L = 8.00 ft	1	0.050	0.024	4.00	-5.83	5.83	195.00	116.77		1.00	1.50	93.74	62.5
Dsgn. L = 13.75 ft	2	0.050	0.043 0.030	1.92 2.72	-5.83	5.83	195.00 195.00	116.77 116.77		1.00	2.69 1.86	93.74 93.74	62.5 62.5
Dsgn. L = 13.00 ft D+Lr, LL Comb Run (LLL)	3	0.041	0.030	2.12	-4.83	4.83	195.00	110.77	1.00	1.00	1.00	93.74	02.
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	62.5
Dsgn. L = 13.75 ft	2	0.062	0.058	0.32	-7.30	7.30	195.00	116.77		1.00	3.61	93.74	62.5
Dsgn. L = 13.00 ft	3	0.062	0.058	6.59	-7.30	7.30	195.00	116.77		1.00	3.61	93.74	62.5
D+S													
Dsgn. L = 8.00 ft	1	0.515	0.240		-60.10	60.10	195.00	116.77		1.00	15.03	93.74	62.5
Dsgn. L = 13.75 ft	2	0.515	0.411	-0.00	-60.10	60.10	195.00	116.77		1.00	25.66	93.74	62.5
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	62.5
D+0.750Lr, LL Comb Run (**L	-)	0.000	0.040		0.07	2.07	105.00	440.77	4.00	4.00	0.00	02.74	60.5
Dsgn. L = 8.00 ft	2	0.028 0.047	0.013 0.049	-0.00	-3.27	3.27 5.44	195.00 195.00	116.77 116.77		1.00	0.82 3.07	93.74 93.74	62.5 62.5
Dsgn. L = 13.75 ft Dsgn. L = 13.00 ft	3	0.052	0.049	6.13	-5.44 -5.44	6.13	195.00	116.77		1.00	3.07	93.74	62.5
D+0.750Lr, LL Comb Run (*L*		0.032	0.049	0.13	-5.44	0.13	133.00	110.77	1.00	1.00	3.07	35.14	02.0
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	62.5
Dsgn. L = 13.75 ft	2	0.043	0.041	1.87	-5.02	5.02	195.00	116.77		1.00	2.55	93.74	62.5
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	62.5
D+0.750Lr, LL Comb Run (*LL	_)												
Dsgn. L = 8.00 ft	1	0.028	0.016	12/12/10	-3.27	3.27	195.00	116.77		1.00	1.01	93.74	62.5
Dsgn. L = 13.75 ft	2	0.059	0.051	0.81	-6.86	6.86	195.00	116.77		1.00	3.18	93.74	62.5
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	62.5
D+0.750Lr, LL Comb Run (L** Dsgn. L = 8.00 ft	, ,	0.044	0.021		-5.19	5.19	195.00	116.77	1.00	1.00	1.30	93.74	62.5
Dsgn. L = 13.75 ft	2	0.044	0.021	-0.00	-5.19	5.19	195.00	116.77	13.50 T.17.	1.00	1.72	93.74	62.5
Dsgn. L = 13.00 ft	3	0.029	0.028	3.40	-3.10	3.40	195.00	116.77		1.00	1.72	93.74	62.5
D+0.750Lr, LL Comb Run (L*L		200000000000000000000000000000000000000	474707	71.00	(2000)	2012	10.00000	1012/2012	75.757	537.0	100000		MAGNA
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	62.5
Dsgn. L = 13.75 ft	2	0.044	0.049	-0.00	-5.19	5.19	195.00	116.77		1.00	3.04	93.74	62.5
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	62.5
D+0.750Lr, LL Comb Run (LL'	"		0.004		- 40	- 10	405.00	440 ==		4.00	4.00	00.74	00.5
Dsgn. L = 8.00 ft	1	0.044	0.021	4.40	-5.19	5.19	195.00	116.77		1.00	1.32	93.74 93.74	62.5 62.5
Dsgn. L = 13.75 ft Dsgn. L = 13.00 ft	2	0.044	0.038 0.029	1.49 2.83	-5.19 -4.52	5.19 4.52	195.00 195.00	116.77 116.77		1.00	2.38 1.83	93.74	62.5
D+0.750Lr, LL Comb Run (LLI		0.039	0.029	2.03	-4,32	4.52	195.00	110.77	1.00	1.00	1.03	33.14	02.0
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	62.5
Dsgn. L = 13.75 ft	2	0.055	0.050	0.29	-6.37	6.37	195.00	116.77		1.00	3.15	93.74	62.5
Dsgn. L = 13.00 ft	3	0.055	0.050	5.74	-6.37	6.37	195.00			1.00	3.15	93.74	62.5
D+0.750S													
Dsgn. L = 8.00 ft	1	0.393	0.184		-45.89	45.89	195.00	116.77		1.00	11.47	93.74	62.5
Dsgn. L = 13.75 ft	2	0.393	0.315	-0.00	-45.89	45.89	195.00	116.77		1.00	19.69	93.74	62.5
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	62.5
0.60D	200	0.047	0.000		4.00	4.00	105.00	440.77	1.00	1.00	0.40	02.74	60 5
Dsgn. L = 8.00 ft	1	0.017	0.008	0.14	-1.96	1.96	195.00	116.77		1.00	0.49	93.74 93.74	62.5 62.5
Dsgn. L = 13.75 ft Dsgn. L = 13.00 ft	2	0.018 0.018	0.017 0.017	0.14 1.92	-2.15 -2.15	2.15 2.15	195.00 195.00	116.77 116.77		1.00	1.06 1.06	93.74	62.5
			0.017	1.92	-2.10	2.10	100.00	110.77	1.00	1.00	1.00	33.74	02.0
Overall Maximum	Defle		22 222			-							
Load Combination		Span	Max. "-" Defl		in Span	Load Com	bination			Max	. "+" Defl	Location	
+D+S		1	0.6989		0.000						0.0000		0.000
21.55		2	0.0000		0.000	+D+S					-0.1082		4.950
+D+S		3	0.2230		7.020						0.0000	,	4.950
<b>Vertical Reactions</b>					Support	notation : Far	left is #1			Values in	n KIPS		
Load Combination		Support 1	Support 2	Support	3 Sup	port 4							

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum	(4/24)	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



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 4 JUL 2017, 3:28PM

Steel Beam

Lic. #: KW-06002886

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Description: B4 - RIDGE OVER ENTRY

Vertical Reactions			S	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**)	100000000000000000000000000000000000000	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		



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 4 JUL 2017, 3:30PM

Steel Beam

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Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886 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

Beam Bracing:

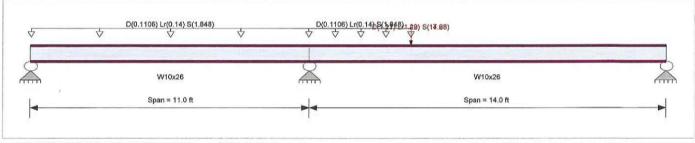
Beam is Fully Braced against lateral-torsional buckling

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)

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 Section used for this span W10x26 W10x26 Ma: Applied 67.037 k-ft Va: Applied 36.178 k Mn / Omega: Allowable Vn/Omega: Allowable 78.094 k-ft 53.560 k Load Combination +D+S Load Combination +D+S 11.000ft 11.000 ft Location of maximum on span Location of maximum on span 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. 0.364 in Ratio = Max Downward Total Deflection 461 >= 180 Max Upward Total Deflection -0.073 in Ratio = 1815 >= 180

#### Maximum Forces & Stresses for Load Combinations

Load Combin	nation		Max Stress	Ratios		S	lummary of M	loment Valu	ies			Summa	ary of Sh	ear Values
Segmer	nt Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only		5.5.5	LONG TO MARKE	W 510 W 151	DATE OF THE	80.117.008.01	O MANAGE A	THE COLUMN TWO IS NOT						2/14/2/2013
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 =		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
		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
	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



VILLAGE NEST JJH MULTI UNIT PROJECT

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Lic. #: KW-06002886
Description: B5 - RIDGE SUPPORT

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
756.43	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			

week with the



VILLAGE NEST

JJH MULTI UNIT PROJECT Project ID: 2017-0610

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Steel Beam

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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**

Bending Axis:

Analysis Method: Allowable Strength Design

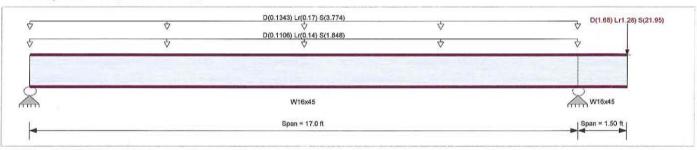
Beam Bracing: Beam is Fully Braced against lateral-torsional buckling

Major Axis Bending

Fy: Steel Yield: E: Modulus :

50.0 ksi

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 = Section used for this span Ma : Applied Mn / Omega : Allowable	0.955: 1 Ma <b>W16x45</b> 196.188 k-ft 205.339 k-ft	aximum Shear Stress Ratio = Section used for this span Va : Applied Vn/Omega : Allowable	0.471 : 1 <b>W16x45</b> 52.339 k 111.090 k
Load Combination Location of maximum on span Span # where maximum occurs	+D+S 8.160ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	+D+S 17.000 ft Span # 1
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	0.564 in Ratio = -0.144 in Ratio = 0.593 in Ratio = -0.151 in Ratio =	344 >= 180	

#### Maximum Forces & Stresses for Load Combinations

Load Combin	nation		Max Stress	Ratios			Summary of M	Ioment Valu	ies			Summ	ary of Sh	ear Values
Segmen	nt Length	Span #	М	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only	ALL DESCRIPTIONS		CANADA	E / C///Au/SPRE	0.000.000	0.000,00	1000 80000	SPECIFICATION OF COLUMN 1	OPERATOR INCOME.			NYS. 480.878		100000000000000000000000000000000000000
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	0.0000000000000000000000000000000000000	127	(0.000/0.00)	263.730		22/21/2004	(87) (3224.5)	X51/5703500	977.D47075594	4,555,550	350/335/2		VEGETAL.	
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		1.05	166.64	111.09



Project Title: VILLAGE NEST JJH MULTI UNIT PROJECT

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Steel Beam

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B6 - w/ SHEAR WALLS EACH END - STEEL Description:

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Spar
+D+S	1	0.5926	8.432	369 696	0.0000	0.000
X(5:0)50	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	1000			
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				



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

**Wood Beam** 

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Lic. #: KW-06002886 **B7 - BEARING HEADERS** Description:

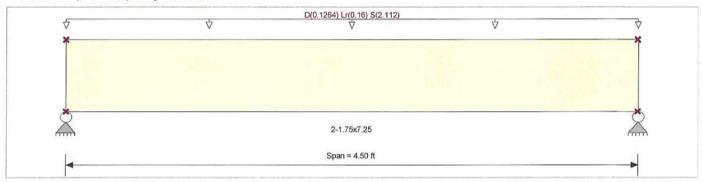
#### **CODE REFERENCES**

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 Elasti	city
Load Combination :ASCE 7-10	Fb - Fc - Prll	2,600.0 psi 2,510.0 psi	Ebend- xx Eminbend - xx	1,900.0ksi 965.71ksi
Wood Species : Trus Joist Wood Grade : MicroLam LVL 1.9 E	Fc - Perp Fv Ft	750.0 psi 285.0 psi 1.555.0 psi		Control of Control
Beam Bracing : Completely Unbraced	rt	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 = 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 Deflecti	ion	0.093 in Ratio	o = 581 >= 240.		
Max Upward Transient Deflection		0.000 in Ratio	0 < 240.0		
Max Downward Total Deflection		0.098 in Ratio	o = 548>=240.		
Max Upward Total Deflection		0.000 in Ratio	0 < 240.0		

	Support notation : Far left is #1	Values in KIPS
Support 1	Support 2	
5.036	5.036	
0.171	0.171	
0.284	0.284	
0.644	0.644	
5.036	5.036	
0.554	0.554	
3.848	3.848	
0.171	0.171	
0.360	0.360	
4.752	4.752	
	5.036 0.171 0.284 0.644 5.036 0.554 3.848 0.171 0.360	5.036 5.036 0.171 0.171 0.284 0.284 0.644 0.644 5.036 5.036 0.554 0.554 3.848 3.848 0.171 0.171 0.360 0.360



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID:

2017-0610

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**Wood Beam** 

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Description: B9 - GARAGE HEADER

Internal Control of the

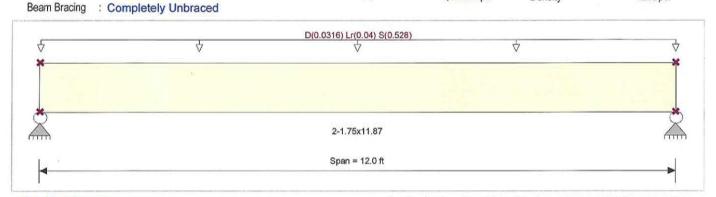
#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

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



#### **Applied Loads**

Max Upward Total Deflection

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		0.500.4		-	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		2-1.75×11.87
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 Deflect	ion	0.267 in Ratio	o = 539>=240.		
Max Upward Transient Deflection		0.000 in Ratio	o = 0<240.0		
Max Downward Total Deflection		0.283 in Ratio	508>=240		

0<240.0

0.000 in Ratio =

	Support notation : Far left is #1	Values in KIPS
Support 1	Support 2	
3.358	3.358	
0.114	0.114	
0.190	0.190	
0.430	0.430	
3.358	3.358	
0.370	0.370	
2.566	2.566	
0.114	0.114	
0.240	0.240	
3.168	3.168	
	3.358 0.114 0.190 0.430 3.358 0.370 2.566 0.114 0.240	Support 1 Support 2  3.358



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

154

Printed: 4 JUL 2017, 3:44PM

**Wood Beam** 

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Lic. #: KW-06002886

Licensee: RICHMOND HOFFMAYER

Description: B8 - BEARING HEADERS

#### **CODE REFERENCES**

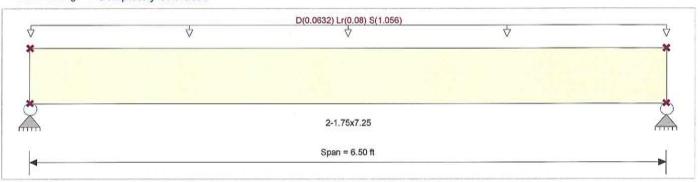
Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

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

Beam Bracing : Completely Unbraced



#### **Applied Loads**

Max Upward Total Deflection

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 Section used for this span	=	0.785 1 2-1.75x7.25	Maximum Shear Stress Ratio Section used for this span	=	0.536 : 1 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 Location of maximum on span	=	+D+S 3.250ft	Load Combination Location of maximum on span	=	+D+S 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			
Max Upward Transient Deflection	n	0.000 in Ratio			
Max Downward Total Deflection		0.214 in Ratio	= 364>=240.		

0<240.0

0.000 in Ratio =

	Support notation : Far left is #1	Values in KIPS
Support 1	Support 2	
3.637	3.637	
0.123	0.123	
0.205	0.205	
0.465	0.465	
3.637	3.637	
0.400	0.400	
2.779	2.779	
0.123	0.123	
0.260	0.260	
3.432	3.432	
	3.637 0.123 0.205 0.465 3.637 0.400 2.779 0.123 0.260	3.637 3.637 0.123 0.123 0.205 0.205 0.465 0.465 3.637 3.637 0.400 0.400 2.779 2.779 0.123 0.123 0.260 0.260



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID:

Printed: 4 JUL 2017, 3:51PM

#### **Wood Beam**

File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Bulld:10.17.6.29, Ver:10.17.6.29

Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886 **B10 - ENTRY HEADER** Description:

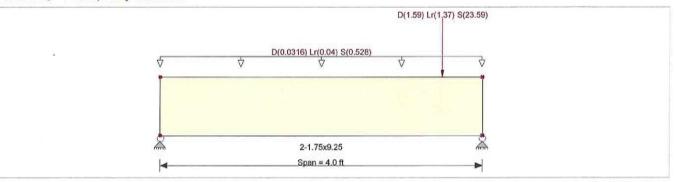
#### **CODE REFERENCES**

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 Elasti	icity
Load Combination :ASCE 7-10	Fb - Fc - Prll	2,600.0 psi 2,510.0 psi	Ebend- xx Eminbend - xx	1,900.0ksi 965.71ksi
Wood Species : Trus Joist Wood Grade : MicroLam LVL 1.9 E	Fc - Perp Fv Ft	750.0 psi 285.0 psi 1,555.0 psi	Density	42.0pcf
Beam Bracing : Completely Unbraced	11	1,555.0 psi	Density	42.000



#### **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 Section used for this span	=	0.934 1 2-1.75x9.25	Maximum Shear Stress Ratio Section used for this span	=	0.543 : 1 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 Deflect Max Upward Transient Deflection Max Downward Total Deflection	11-7-57	0.054 in Rati 0.000 in Rati 0.057 in Rati	o = 0 <240.0		
Max Upward Total Deflection		0.000 in Ratio			

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<sup>1,3</sup>

Panel			Panel Span (ft)								
Core Thickness (in)	Deflection Limit <sup>2</sup>	44	8	10	12	14	16	18	20	22	24
	L/360	103	45	33	24	18	11				
3.5	L/240	225	68	47	34	26	17				
	L/180	297*	91	61	45	34	23				
	L/360	307*	129	57	42	34	25	20	15		
5.5	L/240	307*	182*	87	61	49	37	30	22		
	L/180	307*	182*	112*	80	65	49	39	29		
	L/360	253	171	82	66	54	41	32	23		
7.25	L/240	288*	188*	128	100	81	61	48	35		
	L/180	288*	188*	133*	117*	105	80	63	45		
	L/360	286	188*	117	101	80	58	47	36	32	27
9.25	L/240	326*	188*	147*	134*	120	90	71	52	47	41
	L/180	326*	188*	147*	134*	121	108*	93	68	61	53
	L/360	327*	188*	167*	141	116	91	75	58	47	36
11.25	L/240	327*	188*	167*	153*	132	110*	97	83*	69	53
	L/180	327*	188*	167*	153*	132	110*	97	83*	83	70

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.

\*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		Panel Span (ft)						
Core Thickness (in)	8	10	12	16	20	24		
3.5	3500	2553	2453	2117				
5.5	4250	4943	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		

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

Table 5: Maximum Allowable Uniform Axial Loads (plf) - Type L Panels 1,2,3,4

Panel	Panel Span (ft)						
Core Thickness (in)	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	

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.

MAX LOAD BIB W=(16,8+20+264)1/2 = 2249P2F

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.

<sup>4</sup> 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.

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.

Both facings must bear on the supporting foundation or structure.

<sup>&</sup>lt;sup>4</sup> Tabulated values for 8 ft walls apply to panels constructed with the OSB strength axis oriented either parallel or perpendicular to supports.

<sup>\*</sup> Limited by 1/8 in. deflection (compression)

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.

Roth feelings must be a set to set to

Both facings must bear on the supporting foundation or structure.

<sup>&</sup>lt;sup>4</sup> Tabulated values for 8 ft walls apply to panels constructed with the OSB strength axis oriented either parallel or perpendicular to supports.

<sup>\*</sup> Limited by 1/8 in. deflection (compression)

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

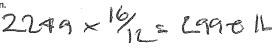
	5: Maximam Andwable Axial Compression Fount Loads (103) - Type O Fai						
1.5" Minimum	3" Minimum						
Bearing Width	Bearing Width						
2040	2450						
4030	4678						
-	Bearing Width						

Top plate secured to facings as required in Section 6.3

<sup>2</sup> Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.

<sup>3</sup> Concentrated loads shall be applied concentrically to the top of the panel.

<sup>4</sup> Tabulated values are based on the strong-axis of the facing material oriented parallel to





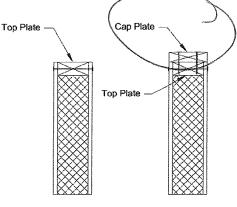


Figure 3: Top Plate Configurations

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

Header	Deflection				
Depth <sup>3</sup> (in)	Limit <sup>4</sup>	4	6	8	10
	L/480	740	384	228	142
12	L/360	740	384	229	142
	L/240	740	384	229	142
	L/480	798	574	385	311
18	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

Continuous over opening

Tabulated values are based on the strong-axis of the facing material oriented perpendicular to the direction of header span.

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<sup>1,2</sup>

Header Depth <sup>3</sup> (in)	Deflection		Header Span (ft)			
	Limit⁴	4	6	8	10	
	L/480	345	243	156	99	
12	L/360	450	295	190	125	
	L/240	630	382	236	153	
	L/480	705	388	254	235	
18	L/360	750	482	302	281	
	L/240	750	482	302	281	
	L/480	698	556	368	350	
24	L/360	896	556	368	350	
	L/240	896	556	368	350	

SiPs wall panel spline minimum 6° from edge of opening

<sup>3</sup> Minimum depth of facing above opening.

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.

Minimum depth of facing above opening.
 Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code.

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.

<sup>&</sup>lt;sup>2</sup> Tabulated values are based on the strong-axis of the facing material oriented perpendicular to the direction of header span.

<sup>&</sup>lt;sup>4</sup> Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code.



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 7 JUL 2017, 10:33AM

**Wood Column** 

File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886

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 End Fixities		e Stress Des ottom Pinned		W	ood Section Name ood Grading/Manuf.		d Lumber	
Overall Column F	leight non-siender cald	odations )	10.50 ft		ood Member Type	Sawn		
Wood Species Wood Grade	Spruce - Pir No.1			100	cact Width cact Depth	3.50 in 7.250 in	Allow Stress Modification Factor Cf or Cv for Bending Cf or Cv for Compression	1.30 1.050
Fb+	850.0 psi		125.0 psi		Area Ix	25.375 in^2 111.148 in^4	Of O. for Taxalan	1.20
Fb - Fc - Prll Fc - Perp	850.0 psi 700.0 psi 425.0 psi	Density	550.0 psi 26.210 pcf		ly	25.904 in^4		1.0
E : Modulus of Ela	asticity	x-x Bending	y-y Bending	Axial			Kf : Built-up columns	1.0 1.0 NDS 15.3.2
	Basic Minimum	1,300.0 470.0	1,300.0 470.0	1,300.0 ks		eflection (bucklin	Use Cr : Repetitive ?	No

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

0.3827:1

BENDING LOADS . . .

WIND: Lat. Uniform Load creating Mx-x, W = 0.2050 k/ft

#### **DESIGN SUMMARY**

	Bending	&	Shear	Check	Resul	ts
--	---------	---	-------	-------	-------	----

PASS Max. Axial+Bending Stress Ratio =

Load Combination	+D+0.60W
Governing NDS Forumla Comp + N	1xx, NDS Eq. 3.9-3
Location of max.above base	5.285 ft
At maximum location values are	
Applied Axial	0.6241k
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 38.172 psi Applied Design Shear Allowable Shear 200.0 psi Maximum SERVICE Lateral Load Reactions . .

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

Bottom along Y-Y Bottom along X-X 1.076 k 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

0.0 in Along X-X

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	CD	CP	Stress Ratio	Status	Location	Stress Ratio	Status	Location	
D Only	0.900	0.877	0.04238	PASS	0.0ft	0.0	PASS	10.50 ft	
+D+Lr	1.250	0.815	0.05858	PASS	O.Oft	0.0	PASS	10.50 ft	
+D+S	1.150	0.834	0.3556	PASS	O.Oft	0.0	PASS	10.50 ft	
+D+0.750Lr	1.250	0.815	0.05215	PASS	O.Oft	0.0	PASS	10.50 ft	
+D+0.750S	1.150	0.834	0.2755	PASS	0.0ft	0.0	PASS	10.50 ft	
+D+0.60W	1.600	0.750	0.3827	PASS	5.285ft	0.1909	PASS	0.0 ft	
+D+0.750Lr+0.450W	1.600	0.750	0.2914	PASS	5.215ft	0.1431	PASS	0.0 ft	
+D+0.750S+0.450W	1.600	0.750	0.3748	PASS	5.285ft	0.1431	PASS	0.0 ft	
+0.60D+0.60W	1.600	0.750	0.3795	PASS	5.285ft	0.1909	PASS	0.0 ft	



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID:

Printed: 7 JUL 2017, 10:33AM

**Wood Column** Lic. # : KW-06002886

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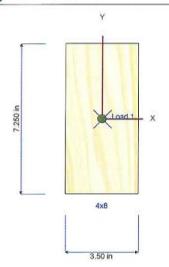
Description:

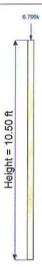
ROOF SUPPORT POSTS - EXTERIOR WALLS - B5 ENDS

#### **Load Combination Results**

			1	Maximum A	Axial + E	Bending S	tress Ratios		Maxim	ium Sh	ear Rati	OS
Load Combination	CD	CP	_	Stress Ra	atio	Status	Location	Stre	ss Ratio	Sta	atus I	Location
+0.60D	1.600	0.750		0.016	74 P	ASS	0.0 ft		0.0	PA	ASS	10.50 ft
Maximum Reactions								Note: C	nly non-	zero re	eactions	s are liste
	X-X Axis F	Reaction	k	Y-Y Axis F	Reaction	Axial	Reaction	My - End M	oments	k-ft	Mx - Er	nd Moments
Load Combination	@ Base	@ Top		@ Base	@ Top	@	) Base	@ Base	@ Top		@ 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	0.64	16	0.624					
+D+0.750Lr+0.450W				0.484	0.48	34	0.991					
+D+0.750S+0.450W				0.484	0.48	84	4.925					
+0.60D+0.60W				0.646	0.64	6	0.374					
+0.60D							0.374					
Lr Only							0.489					
S Only							5.734					
W Only				1.076	1.07	6						
Maximum Deflections for Load Co	mbinations											
Load Combination	Max. X-X Defle	ection I	Distanc	е	Max. Y	-Y Deflect	ion Distanc	се				
D Only	0.0000	in	0.000	0 ft		0.000 in	0.00	00 ft				

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				







VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 7 JUL 2017, 10:28AM

**Wood Column** 

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Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886

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 End Fixities Overall Column F	Top & Bo	e Stress Desi ottom Pinned		We	ood Section Name ood Grading/Manuf. ood Member Type		d Lumber	
Wood Species Wood Grade Fb + Fb - Fc - Prll Fc - Perp	Spruce - Pir No.1 850.0 psi 850.0 psi 700.0 psi 425.0 psi	ne - Fir Fv Ft Density	125.0 psi 550.0 psi 26.210 pcf	Ex	act Width act Depth Area Ix Iy	5.50 in 7.50 in 41.250 in <sup>2</sup> 193.359 in <sup>4</sup> 103.984 in <sup>4</sup>	Cf or Cv for Tension	1.0 1.0 1.0 1.0 1.0
E : Modulus of El	asticity Basic Minimum	x-x Bending 1,300.0 470.0	y-y Bending 1,300.0 470.0	Axial 1,300.0 ksi		effection (bucklin	Kf : Built-up columns Use Cr : Repetitive ?	1.0 NDS 15.3.2 No

Brace condition for deflection (buckling) along columns:

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

**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**

3endin	g & Shear Check Results	
PASS	Max. Axial+Bending Stress Ratio = Load Combination	0.8885 : 1 +D+S
	Governing NDS Forumla	Comp Only, fc/Fc'
	Location of max.above base	0.0 ft
	At maximum location values are	
	Applied Axial	25.259k
	Applied Mx	O.O 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

0.0 ft 23.482 psi 200.0 psi

Maximum SERVICE Lateral Load Reactions . . Top along Y-Y Bottom along Y-Y 1.076 k 1.076 k Bottom along X-X Top along X-X 0.0 k 0.0 k

Maximum SERVICE Load Lateral Deflections . . .

0.2254 in at 5.285 ft above base Along Y-Y for load combination: W Only

Along X-X 0.0 in 0.0 ft above base

for load combination: n/a

Other Factors used to calculate allowable stresses . . .

Compression **Tension** 

## Allowable Shear **Load Combination Results**

Applied Design Shear

	807	CP	Maximum Axial	+ Bending	Stress Ratios	Maximum Shear Ratios			
Load Combination	CD		Stress Ratio	Status	Location	Stress Ratio	Status	Location	
D Only	0.900	0.894	0.07186	PASS	0.0ft	0.0	PASS	10.50 f	
+D+Lr	1.250	0.840	0.1002	PASS	O.Oft	0.0	PASS	10.50 f	
+D+S	1.150	0.856	0.8885	PASS	O.Oft	0.0	PASS	10.50 f	
+D+0.750Lr	1.250	0.840	0.08890	PASS	O.Oft	0.0	PASS	10.50 f	
+D+0.750S	1.150	0.856	0.6811	PASS	O.Oft	0.0	PASS	10.50 f	
+D+0.60W	1.600	0.783	0.3002	PASS	5.285ft	0.1174	PASS	0.0 f	
+D+0.750Lr+0.450W	1.600	0.783	0.2329	PASS	5.285ft	0.08806	PASS	0.0 f	
+D+0.750S+0.450W	1.600	0.783	0.6023	PASS	5.215ft	0.08806	PASS	0.0 f	
+0.60D+0.60W	1.600	0.783	0.2956	PASS	5.285ft	0.1174	PASS	0.0 f	



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

161

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**Wood Column** 

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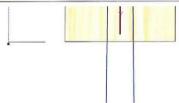
Lic. #: KW-06002886 Description:

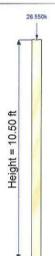
ROOF SUPPORT POSTS - EXTERIOR WALLS - B10 ENDS

						D	
0.2	C	io.	ทษแ	กลก	on	Resu	TS

	015	540.00	M	aximum Axi	al + Bend	ing Stress Rati	ios		Maxim	um Sh	ear Ratio	os
Load Combination	CD	CP	almostro	Stress Ratio	-			Stres	s Ratio	Sta	atus L	ocation
+0.60D	1.600	0.783		0.02768	PAS	S 0.	Oft		0.0	PA	ASS	10.50 ft
Maximum Reactions							N	lote: Or	nly non-	zero re	eactions	are liste
	X-X Axis F	Reaction	k	Y-Y Axis Rea	action	Axial Reaction	My -	End Mo	ments	k-ft		nd Moments
Load Combination	@ Base	@ Top	(	@ Base @	Тор	@ Base	@ E	Base	@ Top		@ Base	@ Top
D Only		-				1.669						
+D+Lr						3.039						
+D+S						25.259						
+D+0.750Lr						2.696					7	
+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 L	oad Combinations				V 1111000000000000000000000000000000000							
Load Combination	Max. X-X Defl	ection D	istance	N	Max. Y-Y D	eflection Di	stance					
D Only	0.0000	in	0.000	ft	0.0	00 in	0.000 ft					
+D+I c	0.000	in	0.000	ft	0.0	00 in	0.000 ft					

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					







VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID:

2017-0610

162

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#### Steel Column

Lic. # : KW-06002886

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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 Fy: Steel Yield A-36, Carbon Steel, Fy = 36 ksi

E : Elastic Bending Modulus

36.0 ksi 29,000.0 ksi Overall Column Height

11.0 ft Top & Bottom Pinned

Top & Bottom Fixity Top & Bottom
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**

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 = Load Combination	0.8326 +D+S	:1
	Location of max.above base At maximum location values are	0.0	ft
	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

PASS Maximum Shear Stress Ratio =
Load Combination
Location of max.above base
At maximum location values are . . .

Mn-y / Omega: Allowable

Va : Applied Vn / Omega : Allowable

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

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

Maximum Load Deflections . . .

Along Y-Y 0.0 in at for load combination :

0.0ft above base

ioi ioaa cc

Along X-X 0.0

0.0 in at 0.0ft above base

Note: Only non-zero reactions are listed.

for load combination:

#### **Load Combination Results**

	Maximum Axial +	Bending S	tress Ratios	Maximu	atios	
Load Combination	Stress Ratio	Status	Location	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

7.275 k-ft

0.0:1

0.0 ft

0.0 k

#### **Maximum Reactions**

Y-Y Axis Reaction k-ft Mx - End Moments My - End Moments **Axial Reaction** X-X Axis Reaction **Load Combination** @ Base @ Base @ Top @ Base @ Top @ Base @ Top @ 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



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

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Steel Column

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13.600 in^4

Lic. #: KW-06002886 Description:

RIDGE SUPPORT - B4 AND B5

The second		1				
E 200	trem			-4	-	-
- 7		S-y PC	$\mathbf{e}$	СТ		1 -

	,	Axial Reaction	X-X Axis	Reaction	k	Y-Y Axis	Reaction	Mx - End M	oments	k-ft	My - End	Moments
Item	Extreme Value	@ Base	@ Base	@ Top		@ Base	@ Тор	@ Base	@ Top		@ Base	@ Top
Axial @ Base	Maximum	53.307	10.00				///					
	Minimum	2.356										
Reaction, X-X Axis Base	Maximum	3.926										
H	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										
mailment ox reparties	Minimum	3.926										

### Maximum Deflections for Load Combinations

Load Combination	Max. X-X Defle	ction	Distance		Max. Y-Y Def	ection	Distanc	oe .
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

Charle	Castian	Properties :	Pine4 Std
STEPL :	Section	Properties	F1064 510

Depth	=	4.500	in	XX	=	6.82 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	l yy	=	6.820 in^4
Weight	=	10.858	plf	S yy	=	3.030 in^3
				R yy	=	1.510 in

Ycg

0.000 in



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

169

Printed: 7 JUL 2017, 10:36AM File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29 Steel Column Lic. # : KW-06002886 Licensee : RICHMOND HOFFMAYER Description: RIDGE SUPPORT - B4 AND B5



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

115

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Steel Column

File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886 Description: B6 SL

**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 : Analysis Method : HSS6x6x3/8 Allowable Strength

Steel Stress Grade

Fy: Steel Yield E: Elastic Bending Modulus 46.0 ksi 29,000.0 ksi Overall Column Height Top & Bottom Fixity

ight 10.0 ft Top & Bottom Pinned

Note: Only non-zero reactions are listed

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

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**

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 N	Max. Axial+Bending Stress Ratio =	0.9468 :	1	Maximum Load	d Reactions			
	Load Combination	+D+S		Top alor	g X-X		0.4751 k	
	Location of max.above base	9.933 ft	t	Bottom a	along X-X		0.4751 k	
	At maximum location values are			Top alor	a Y-Y		0.0 k	
	Pa : Axial	76.295 k		Bottom a	along Y-Y		0.0 k	
	Pn / Omega : Allowable	91.797 k						
	Ma-x : Applied	0.0 k	-ft	Maximum Load	d Deflections			
	Mn-x / Omega : Allowable	36.267 k	1000	Along Y-Y	0.0 in	at	0.0ft	above base
	Ma-y: Applied	-4.719 k	-ft	for load co	mbination .			
	Mn-y / Omega : Allowable	36,267 k	-ft	Along X-X	-0.04638 in	at	5.839ft	above base
				for load o	ombination:+D+S	3		
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 Vn / Omega : Allowable	0.4751 k 57.137 k						

#### **Load Combination Results**

	Maximum Axial +	-Bending S	tress Ratios	Maximu	atios		
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	0.094	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**

Maximum Reactions						Note. C	any non-	26101	eactions a	ne iisteu
III and Carlot I amount I amou	'-Y Axis Reaction Mx - End Moments k-ft My									
Load Combination	@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	)	@ Base	@ Top
D Only	4.635	0.027	0.027		- deleteration					
+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							



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

166

Steel Column

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ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29
Liteenstee: RIGHIMOND HOFFIMAYER

Lic. #: KW-06002886 Description: B6 SUPP

B6 SUPPORT - MAX AT B3 SUPPORT

ACT TO SERVICE A SERVICE ASSESSMENT OF THE S		Axial Reaction	X-X Axis	Reaction	k	Y-Y Axis	Reaction	Mx - Er	nd Mome	ents	k-ft	My - End	Moments
Item	Extreme Value	@ Base	@ Base	@ Тор		@ Base	@ Top	@ Bas	se i	@ Тор		@ Base	@ Top
Axial @ Base	Maximum	76.295	0.47	5 0.47	5								
	Minimum	2.781	0.01	6 0.01	6								
Reaction, X-X Axis Base	Maximum	76.295	0.47	5 0.47	5								
н	Minimum	2.781	0.01	6 0.01	6								
Reaction, Y-Y Axis Base	Maximum	4.635	0.02	7 0.02	7								
ir.	Minimum	4.635	0.02	7 0.02	7								
Reaction, X-X Axis Top	Maximum	76.295	0.47	5 0.47	5								
	Minimum	2.781	0.01	6 0.01	6								
Reaction, Y-Y Axis Top	Maximum	71.660	0.44	8 0.44	8								
	Minimum	4.635	0.02	7 0.02	7								
Moment, X-X Axis Base	Maximum	4.635		0.02	7								
	Minimum	4.635		0.02	7								
Moment, Y-Y Axis Base	Maximum	4.635	0.02	7 0.02	7								
	Minimum	4.635	0.02	7 0.02	7								
Moment, X-X Axis Top	Maximum	4.635	0.02	7 0.02	7								
	Minimum	4.635	0.02	7 0.02	7								
Moment, Y-Y Axis Top	Maximum	4.635	0.02	7 0.02	7								
·	Minimum	4.635	0.02	7 0.02	7								
Maximum Deflection	ns for Load Comb	oinations											
Load Combination		Max. X-X Defle	ection Dis	stance		Max. Y-Y D	eflection	Distanc	e				
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+0.7E01 =		0.0046	in C	920 #		0.000	In	0.000	4				

D Only	-0.0027	in	5.839	44	0.000	in	0.000	ft			_
D Only	-0.0027	in		н		in		IL.			
+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										
	0.000				00.00 1.44				C4 CD0 !- A	4	_

roperties :		JOSOXO.	X3/0					
=	6.000	in	1 xx	=	39.50 in^4	J	=	64.600 in^4
=	0.349	in	S xx	=	13.20 in^3			
=	6.000	in	R xx	=	2.280 in			
=	0.375	in	Zx	=	15.800 in^3			
=	7.580	in^2	l yy	=	39.500 in^4	C	=	22.100 in^3
=	27.480	plf	S yy	=	13.200 in^3			
			R yy	=	2.280 in			
	= = = = = = = = = = = = = = = = = = = =	= 6.000 = 0.349 = 6.000 = 0.375 = 7.580	= 6.000 in = 0.349 in = 6.000 in = 0.375 in = 7.580 in^2	= 6.000 in 1 xx = 0.349 in S xx = 6.000 in R xx = 0.375 in Zx = 7.580 in^2 1 yy = 27.480 plf S yy	= 6.000 in	= 6.000 in	= 6.000 in	= 6.000 in

0.000 in Ycg



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

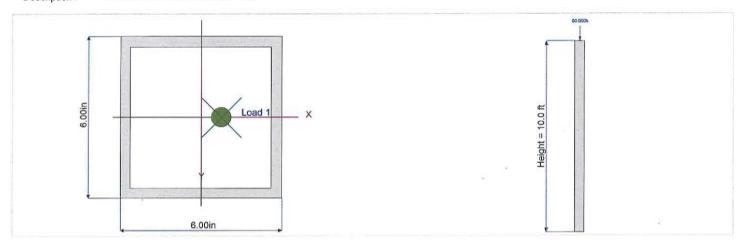
**Steel Column** 

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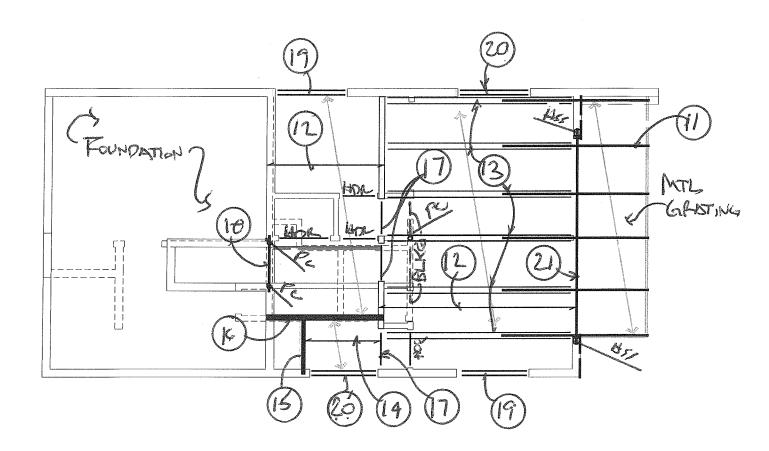
B6 SUPPORT - MAX AT B3 SUPPORT



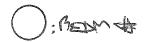




**PROJECT** FELDSPAR LOWER JOB 2017-0610 CLIENT VILLAGE NEST ADDRESS EDEN, UTAH **DESIGN** HUUM J.H. DATE 6/16/2017 **ENGINEER** RICHMOND PE 49628 HOFFMAYER SE 3935 SHEET // OF



ZNP FLOOR FRAMING PLAN FELDSPAR - UPPER N.T.S.





VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

17.

Printed: 7 JUL 2017, 1:59PM

**Wood Beam** 

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Lic. # : KW-06002886 Description : B14 -

B14 - SHORT FLOOR JOISTS

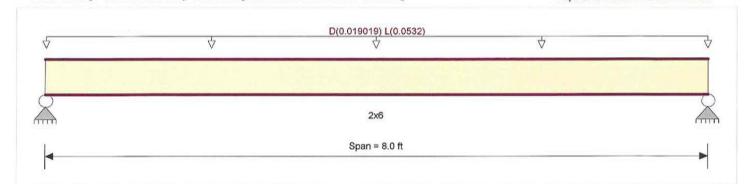
#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

Analysis Method: Allowable Stress Design E: Modulus of Elasticity 875.0 psi Fb+ 875.0 psi 1,400.0ksi Load Combination ASCE 7-10 Fb-Ebend-xx 1,150.0 psi Fc - Prll Eminbend - xx 510.0ksi 425.0 psi 135.0 psi Fc - Perp Wood Species : Spruce - Pine - Fir F۷ : No. 1/No. 2 Wood Grade 450.0 psi 26.210pcf Density 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)

= = =	0.701: 1 <b>2x6</b> 916.76psi 1,308.13psi	Maximum Shear Stress Ratio Section used for this span fv : Actual	=	0.346 : 1 2x6
		fv : Actual	=	The same transfer of the same
=	1 308 13nei			46.77 psi
	1,500.15081	Fv : Allowable	=	135.00 psi
=	+D+L 4.000ft	Load Combination Location of maximum on span	=	+D+L 7.562ft
=	Span # 1	Span # where maximum occurs	=	Span # 1
	0.000 in Rati 0.230 in Rati	o = 0<360.0 o = 417>=240.		
		= Span # 1  0.169 in Rati 0.000 in Rati 0.230 in Rati	Span # 1 Span # where maximum occurs  0.169 in Ratio = 566 >= 360. 0.000 in Ratio = 0 < 360.0	Span # 1 Span # where maximum occurs =  0.169 in Ratio = 566>=360. 0.000 in Ratio = 0<360.0 0.230 in Ratio = 417>=240.

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		



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 7 JUL 2017. 2:02PM

**Wood Beam** 

Lic. #: KW-06002886

File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29 Licensee: RICHMOND HOFFMAYER

Description:

**B15 - FLOOR SUPPORT** 

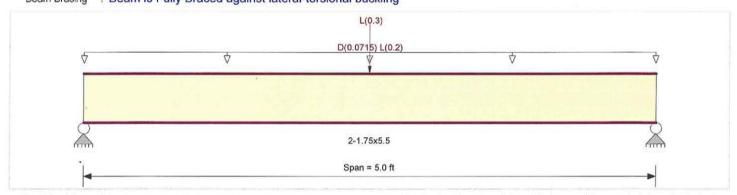
#### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

Analysis Metho	: Allowable Stress Design	Fb+	2,600.0 psi	E : Modulus of Elasti	city
	Load Combination :ASCE 7-10		2,600.0 psi	Ebend- xx	1,900.0ksi
		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		
Wood Grade	, miorozam zvz no z	Ft	1,555.0 psi	Density	42.0pcf
Beam Bracing	: Beam is Fully Braced against lateral-	torsional buckling	E PARTIE AND THE APPLICATION	: TO THE OFFICE OF THE OFFI	Charles and Market



#### **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 Section used for this span	=	0.325 1 2-1.75x5.5	Maximum Shear Stress Ratio Section used for this span	=	0.196 : 1 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 Location of maximum on span Span # where maximum occurs	= =	+D+L 2.500ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	= =	+D+L 0.000 ft Span # 1
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	n	0.045 in Ratio 0.000 in Ratio 0.057 in Ratio 0.000 in Ratio	o = 0 < 360 o = 1048 > = 240.		

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	

## RICHMOND HOFFMAYER



PROJECT	VILLAGE NEST	JOB	2017-0610
CLIENT			
ADDRESS	EDEN, UT		
DESIGN	HUUM		
ENGINEER	J.H.	DATE	6/14/2017
	RICHTOND PE 44628 NOFFHAYER SE 5455	SHEET	iz @ OF

# LETAL GRATING: D= 12 PSF L= 608312 3 = 2648SF - (OPENERS GRATING - POINT DBL SHOW) LOAD COMBINATIONS: ロナム = フレイデ P+3 = 276837 - GOVERNS P+076L+0765 = 2668 M = W9 (2- SPAN OF) SPAN 2944 W-12 6624 N-1 1177612-11 MAJ. ALIMINUM STEEL E = 10000 ksi 29000 kci F = 12000 PSI MIN ( STAN IN MULES) FSK>M & EIK > 5 US/4000 × 240 C BALL/FL L 27 16 15 F 12000 (S)27.4 > 2944 -> 5 = 0.009 12 } AL 12000 (S)27.4 > 6624 -> 5 = 0.020 12 } 12000 (S)27.4 > 11776 -> 5 = 0.035 12 }

## RICHMOND HOFFMAYER INC. STRUCTURAL ENGINEERING



PROJECT	VILLAGE NEST	JOB	2017-0610
CLIENT	-		
ADDRESS	EDEN, UT		
DESIGN	HUUM		
ENGINEER	J.H.	DATE	6/14/2017

Info@RichmondHoffmayer.com

 $12/18(0.000) = 0.00613^{3}$   $12/18(0.000) = 0.013310^{3}$   $12/18(0.036) = 0.023310^{3}$   $12/18(0.036) = 0.023310^{3}$   $12/18(0.036) = 0.023310^{3}$   $12/18(0.036) = 0.023310^{3}$   $12/18(0.036) = 0.023310^{3}$   $12/18(0.036) = 0.023310^{3}$   $12/18(0.036) = 0.023310^{3}$   $12/18(0.036) = 0.023310^{3}$   $12/18(0.036) = 0.023310^{3}$   $12/18(0.020) = 0.023310^{3}$  1

10/(0,0009) = 0,00013 1NA 10/29 (0,0009) = 0,000035 NA 10/29 (0,0009) = 0,0001 1NA

1" DEER X 3/L" AL = 0.03/3/N°

WOSE UP TO 4-6" SPANK U/ RARGE & 7/L"

I" DEER X 3/L" STED = 3 = 0.03/3/N°

I = 0.0156/N°

I = 0.03/3/N°

I =



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 7 JUL 2017, 11:47AM

Steel Beam

File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29 Licensee: RICHMOND HOFFMAYER

Description:

Lic. #: KW-06002886 **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

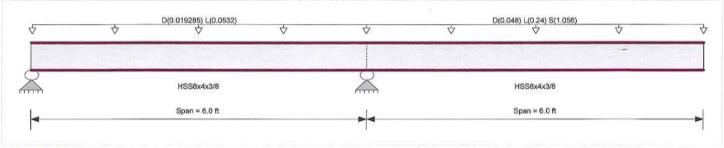
Beam is Fully Braced against lateral-torsional buckling Beam Bracing:

Major Axis Bending Bending Axis:

Fy: Steel Yield: E: Modulus :

50.0 ksi

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 Location of maximum on span	+D+S 6.000ft	Load Combination Location of maximum on span	+D+S 6.000	ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1	
Maximum Deflection				

Maximum Deflection

Max Downward Transient Deflection 0.089 in Ratio = 1,616>=360. Max Upward Transient Deflection -0.009 in Ratio = 7,748 >= 360.0.397 in Ratio = Max Downward Total Deflection 363 >= 240. Max Upward Total Deflection -0.043 in Ratio = 1675 >= 240.

Load Combin	ation		Max Stress	Ratios	Summary of Moment Values						Summary of Shear Values			
Segmen	t Length	Span #	М	٧	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	Maximu	m Deflect	tions											

Overall maximum	Delicetions					
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



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

Steel Beam

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Lic. #: KW-06002886

Description: B11 - DECK SUPPORT Description:

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	A the district of the form of the Control of the Co	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				



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 7 JUL 2017, 11:49AM

**Wood Beam** 

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Lic. #: KW-06002886 Description:

**B12-FLOOR JOISTS** 

#### **CODE REFERENCES**

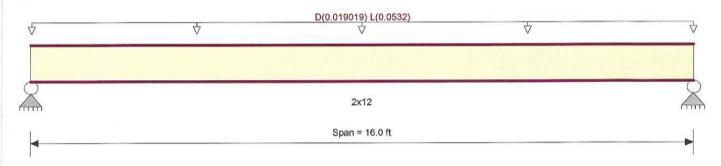
Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

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

: Beam is Fully Braced against lateral-torsional buckling Beam Bracing Repetitive Member Stress Increase D(0.019019) L(0.0532)



#### **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 Section used for this span	=	0.871: 1 M 2x12	aximum Shear Stress Ratio Section used for this span	=	0.336 : 1 2x12
fb : Actual	=	876.47psi	fv : Actual	=	45.36 psi
FB : Allowable	=	1,006.25psi	Fv : Allowable	=	135.00 psi
Load Combination Location of maximum on span Span # where maximum occurs	= =	+D+L 8.000ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	= =	+D+L 15.066 ft Span # 1
Maximum Deflection Max Downward Transient Deflec Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection		0.317 in Ratio = 0.000 in Ratio = 0.430 in Ratio = 0.000 in Ratio =	0<360.0 446>=240.		

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	



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 7 JUL 2017, 11:50AM

Design OK

**Wood Beam** 

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Lic. #: KW-06002886

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**

E: Modulus of Elasticity Analysis Method: Allowable Stress Design 875.0 psi Fb + 1,400.0ksi Load Combination ASCE 7-10 875.0 psi Fb -Ebend-xx Fc - Prll 1,150.0 psi Eminbend - xx 510.0ksi 425.0 psi Fc - Perp Wood Species Spruce - Pine - Fir 135.0 psi F۷ : No. 1/No. 2 Wood Grade 450.0 psi 26.210pcf Density



### **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)

DESI	GN	SU	IMM!	ARY	
	-			0.	1141

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	=	934.27psi	fv : Actual	=	57.02 psi
FB : Allowable	=	1,006.25psi	Fv : Allowable	=	135.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	7.766ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span #1	Span # where maximum occurs	=	Span #1
Maximum Deflection					

Maximum Deflection

0.317 in Ratio = Max Downward Transient Deflection 606>=360. Max Upward Transient Deflection 0.000 in Ratio = 0<360.0 Max Downward Total Deflection 0.463 in Ratio = 414>=240. 0.000 in Ratio = Max Upward Total Deflection 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		



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 7 JUL 2017, 11:53AM

**Wood Beam** 

File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29 Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886 Description:

B13 - FLOOR JOISTS w/ DECK SUPPORT

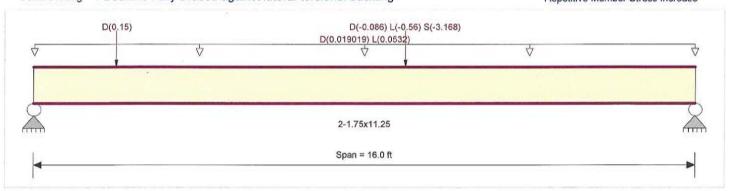
### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

### **Material Properties**

Analysis Metho	: Allowable Stress Design	¥	Fb+	2,600.0 psi	E : Modulus of Elasti	city
	on ASCE 7-10		Fb-	2,600.0 psi	Ebend- xx	1,900.0ksi
			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		
Wood Grade	, miorozam Eve nio E		Ft	1,555.0 psi	Density	42.0pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsiona		kling	Carlo March March 1985	Repetitive Membe	r 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)

Max Upward Transient Deflection

Max Downward Total Deflection

Max Upward Total Deflection

L Only

S Only

Point Load: D = -0.0860, L = -0.560, S = -3.168 k @ 9.0 ft, (B9)

D	ES	IGN	S	JMI	VIA.	RY

DESIGN SUMMARY					Design OK
Maximum Bending Stress Ratio Section used for this span fb : Actual	= 10000000000	0.554: 1 2-1.75x11.25 1,903.30psi	Maximum Shear Stress Ratio Section used for this span fv : Actual	=	0.207 : 1 2-1.75x11.25 67.79 psi
FB : Allowable	=	3,438.50psi	Fv : Allowable	=	327.75 psi
Load Combination Location of maximum on span Span # where maximum occurs	=	+D+S 8.993ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	= =	+D+S 9.051 ft Span # 1
Maximum Deflection Max Downward Transient Deflect	ion	0.003 in Ra	tio = 68982 >=360		

49011>=360.

3690>=240. 494>=240.

-0.004 in Ratio =

0.052 in Ratio =

-0.388 in Ratio =

0.181

-1.386

Vertical Reactions		Support nota	ation : 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 Only +D+L +D+S	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			

0.111

-1.782



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 7 JUL 2017, 2:05PM

**Wood Beam** 

Lic. #: KW-06002886

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Description:

**B16 - FLOOR SUPPORT** 

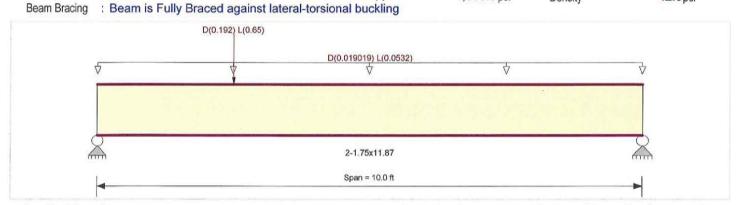
### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

### **Material Properties**

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



### **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)

D	ESI	GN	SUN	IMA	RY
				1	

DESIGN SUMMARY					Design OK
Maximum Bending Stress Ratio Section used for this span	=	0.133 1 2-1.75x11.87	Maximum Shear Stress Ratio Section used for this span	=:	0.123 : 1 2-1.75x11.87
fb : Actual	=	345.66psi	fv : Actual	=	35.01 psi
FB : Allowable	=	2,600.00psi	Fv : Allowable	=	285.00 psi
Load Combination Location of maximum on span	=	+D+L 2.518ft	Load Combination Location of maximum on span	=	+D+L 0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection Max Downward Transient Deflect	ion	0.031 in Ratio	= 3922>=360		

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	



VILLAGE NEST JJH MULTI UNIT PROJECT

Printed: 7 JUL 2017. 2:09PM

Wood Beam

File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886 Description:

B17 - FLOOR SUPPORT - FLUSH w/ FLOOR JOISTS

### **CODE REFERENCES**

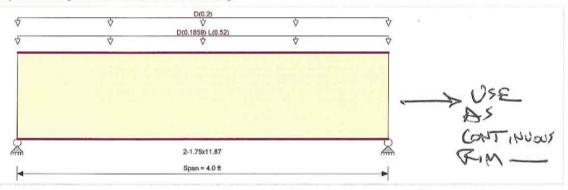
Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

### **Material Properties**

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

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

Max Downward Total Deflection

Max Upward Total Deflection

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.)

-			~ .		M M F	
"	_	16 - NI	-	ININ	7/1/1	J V

DESIGN SUMMARY					Design OK
Maximum Bending Stress Ratio Section used for this span	=	0.103 1 2-1.75x11.87	Maximum Shear Stress Ratio Section used for this span	=	0.119 : 1 2-1.75x11.87
fb : Actual	=	267.84psi	fv : Actual	=	33.86 psi
FB : Allowable	=	2,600.00psi	Fv : Allowable	=	285.00 psi
Load Combination Location of maximum on span Span # where maximum occurs	=	+D+L 2.000ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	=	+D+L 3.022 ft Span # 1
Maximum Deflection Max Downward Transient Deflect Max Unward Transient Deflection		0.003 in Ratio			

8374>=240.

0<240.0

Vertical Reactions	Support notation : Far left is #1	Values in KIPS
--------------------	-----------------------------------	----------------

0.006 in Ratio =

0.000 in Ratio =

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	
원 (1995년 1995년)			



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 7 JUL 2017, 2:41PM

Steel Beam

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Lic. #: KW-06002886

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

Beam Bracing:

Completely Unbraced

Fy: Steel Yield:

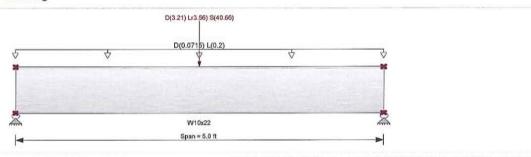
50.0 ksi

E: Modulus :

29,000.0 ksi

Bending Axis:

Major Axis Bending



Maximum Shear Stress Ratio =

Section used for this span

Va: Applied -

### **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

Maximum Bending Stress Ratio = 0.850:1 Section used for this span W10x22 Ma: Applied 55.130 k-ft Mn / Omega: Allowable 64.870 k-ft +D+S Load Combination Location of maximum on span

Vn/Omega: Allowable Load Combination 2.500ft Location of maximum on span Span #1 Span # where maximum occurs

0.453:1W10x22 22.169 k 48.960 k +D+S 0.000 ft Span #1

Design OK

Maximum Deflection

Span # where maximum occurs

Max Downward Transient Deflection 0.005 in Ratio = 12,761>=360 Max Upward Transient Deflection 0.000 in Ratio = 0 < 360 0.009 in Ratio = Max Downward Total Deflection 6433 >= 240. Max Upward Total Deflection 0.000 in Ratio = 0 < 240.0

### Maximum Forces & Stresses for Load Combinations

Load Combina	ation Max Stress Ratios					Summary of Moment Values							Summary of Shear Values		
Segment Length	Span#	М	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega		
D Only			3,777,00												
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+0.7	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 Positions			Support	notation : Far left is #1	Values in KIPS	

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			
D Only	1.000	1.000			



VILLAGE NEST MULTI UNIT PROJECT

Project ID: 2017-0610

101

Steel Beam

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Lic. #: KW-06002886
Description: B18 - RIDGE SUPPORT

Values in KIDS

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		
1/70				



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

187

Printed: 7 JUL 2017, 3:07PM

Steel Column Lic. #: KW-06002886 File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29 Ligensee: 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 E: Elastic Bending Modulus 36.0 ksi

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

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 =

Load Combination
Location of max.above base
At maximum location values are . . .

Va : Applied Vn / Omega : Allowable 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

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

Maximum Load Deflections . . .

Along Y-Y 0.0 in at for load combination :

0.0ft above base

101 1000

Along X-X

0.0 in at 0.0ft above base

for load combination:

**Load Combination Results** 

Load Combination Results

Maximum Shear Ratios Maximum Axial + Bending Stress Ratios Load Combination Stress Ratio Status Location Stress Ratio Status Location D Only 0.034 0.000 PASS 0.00 ft 0.00 ft PASS +D+L 0.042 0.00 ft 0.000 PASS 0.00 ft PASS +D+Lr 0.065 0.000 PASS 0.00 ft 0.00 ft PASS +D+S 0.393 0.00 ft 0.000 PASS 0.00 ft PASS +D+0.750Lr+0.750L 0.064 0.00 ft 0.000 PASS 0.00 ft PASS +D+0.750L+0.750S 0.310 PASS 0.00 ft 0.000 PASS 0.00 ft +0.60D 0.020 0.00 ft 0.000 PASS 0.00 ft PASS

0.0:1

0.0 ft

0.0 k 0.0 k

**Maximum Reactions** 

Note: Only non-zero reactions are listed.

maximum resourcine							1.507.5701.67				
	Axial Reaction	X-X Axis	Reaction	k	Y-Y Axis	Reaction	Mx - End M	loments k-ft	My - End	Moments	
Load Combination	@ Base	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top	@ 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										



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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Steel Column								ENERCALC, IN			C Projects\201	
Lic. #: KW-06002886				-T-10 Tuesday		Total State of the last of the	Ser and the series		ensee:R			
Description: B18 S	UPPORT											
Maximum Reactions	3							Note:	Only non-	zero	reactions a	are listed.
Load Combination		Axial Reaction @ Base	X-X Axis I @ Base	Reaction @ Top	k	Y-Y Axis @ Base	Reaction @ Top	Mx - End @ Base	Moments @ Top	k-ft	My - End @ Base	Moments @ Top
Lr Only		1.780		0 ,								
L Only S Only		0.500 20.330										
<b>Extreme Reactions</b>												
		Axial Reaction	X-X Axis	Reaction	k	Y-Y Axis	Reaction	Mx - End I	Moments	k-ft	My - End	Moments
Item	Extreme Valu		@ Base	@ Тор		@ Base	@ Тор	@ Base	@ Тор		@ Base	@ Тор
Axial @ Base	Maximum Minimum	22.234 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 Minimum	1.904 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 Deflection	s for Load Co											
Load Combination		Max. X-X Deflect	ion Dist	ance		Max. Y-Y D	eflection	Distance				
D Only		0.0000 i	n 0.0	000 ft		0.000	) in	0.000 f	t			
+D+L		0.0000 i	n 0.0	000 ft		0.000	) in	0.000 f	t			
+D+Lr		0.0000 i	n 0.0	000 ft		0.000	) in	0.000 f	t			
+D+S			n 0.0	000 ft		0.000	) in	0.000 f	t			
+D+0.750Lr+0.750L			n 0.0	000 ft		0.000			t			
+D+0.750L+0.750S			n 0.0	000 ft		0.000	) in	0.000 f	ť			
+0.60D				000 ft		0.000			t			
Lr Only				000 ft		0.000			t			
L Only				000 ft		0.000			t			
S Only		0.0000 i	n 0.0	000 ft		0.000	) in	0.000 f	t			
Steel Section Proper		Pipe4STD			- Carrett Control							
Depth	4.500					in^4		J	=		13.600 in^4	
711		Sx				in^3						
	4.500				1.510							
	0.237		=		4.050							
	2.960				6.820							
Weight :	10.800				3.030							
		Ry	y =		1.510	in						
Ycg =	0.000	in										



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver.10.17.6.29
Licensee: RICHIMOND HOFFMAYER Steel Column Lic. # : KW-06002886 Description: **B18 SUPPORT** 24,449k



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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**Wood Beam** 

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Lic. #: KW-06002886 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 Elasti	city
Load Combination ASCE 7-10	Fb -	2,600.0 psi	Ebend- xx	1,900.0ksi
	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		
Wood Glade . Island Earl Eve 1.0 E	Ft	1,555.0 psi	Density	42.0pcf
Beam Bracing : Completely Unbraced		Control Market Control		-10000100



## **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 SHIMMARY

DESIGN SUMMARY					Design OK
Maximum Bending Stress Ratio Section used for this span	=	0.815 1 2-1.75x9.5	Maximum Shear Stress Ratio Section used for this span	=	0.709 : 1 2-1.75x9.5
fb : Actual	$f_{ij} = f_{ij}$	2,388.03psi	fv : Actual	=	232.29 psi
FB : Allowable	=	2,931.26psi	Fv : Allowable	=	327.75 psi
Load Combination Location of maximum on span	=	+D+S 3.000ft	Load Combination Location of maximum on span	=	+D+S 0.000 ft
Span # where maximum occurs	=	Span #1	Span # where maximum occurs	=	Span # 1
Maximum Deflection Max Downward Transient Deflec	tion	0.122 in Ratio	= 589>=360		

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 nota	ition : 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		4	
+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			

est -



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

**Wood Beam** 

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Lic. #: KW-06002886 Description: B19 - HEADERS

Vertical Reactions		Support notation : Far left is #1	Values in KIPS
Load Combination	Support 1	Support 2	i i
L Only	0.160	0.160	
S Only	5.940	5.940	



VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

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**Wood Beam** 

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Lic. #: KW-06002886 Description:

**B20 - HEADERS** 

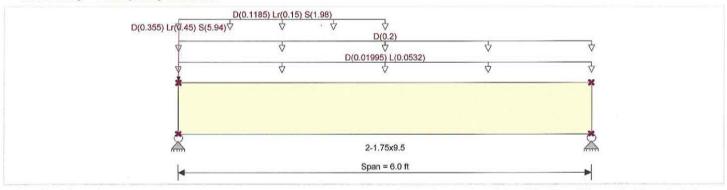
### **CODE REFERENCES**

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

### **Material Properties**

Fb+	2,600.0 psi	E : Modulus of Elastic	city
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
	* AMERICAN AND TOTAL	THE TOTAL OF THE STATE OF THE S	Compression of the Profits
	Fc - PrII Fc - Perp Fv	Fb - 2,600.0 psi Fc - Prll 2,510.0 psi Fc - Perp 750.0 psi Fv 285.0 psi	Fb -         2,600.0 psi         Ebend- xx           Fc - Prll         2,510.0 psi         Eminbend - xx           Fc - Perp         750.0 psi         Fv           285.0 psi         Fv         Fv



### **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				18	Design OK
Maximum Bending Stress Ratio Section used for this span	=	0.489 1 2-1.75x9.5	Maximum Shear Stress Ratio Section used for this span	=	0.492 : 1 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 Location of maximum on span Span # where maximum occurs	=	+D+S 2.321ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	=	+D+S 0.000 ft Span # 1
Maximum Deflection Max Downward Transient Deflect Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	pana	0.062 in Rat 0.000 in Rat 0.079 in Rat 0.000 in Rat	io = 0 <360 io = 906 >= 240.		

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	



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 7 JUL 2017, 3:24PM

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**Wood Beam** 

Lic. #: KW-06002886
Description: B20 - HEADERS

File = d:\ENERCALC Projects\2017-0610.ec6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver.10.17.6.29
Licensee: RICHMOND HOFFMAYER

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	



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 7 JUL 2017, 3:27PM

Steel Beam

Lic. #: KW-06002886

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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

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

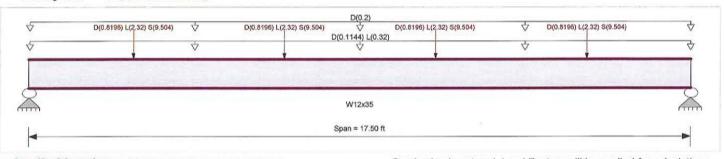
Bending Axis:

Major Axis Bending

Fy: Steel Yield: E: Modulus :

50.0 ksi

29,000.0 ksi



# **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 Va : Applied 114.595 k-ft 24.532 k Mn / Omega: Allowable Vn/Omega: Allowable 75.0 k 127.745 k-ft +D+0.750L+0.750S Load Combination +D+0.750L+0.750S Load Combination 0.000 ft Location of maximum on span 8.750ft Location of maximum on span Span # where maximum occurs Span # where maximum occurs Span #1 Span #1

Maximum Deflection

Max Downward Transient Deflection 0.233 in Ratio = 902>=360 Max Upward Transient Deflection 0.000 in Ratio = 0 < 360 0.375 in Ratio = Max Downward Total Deflection 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
	No N	1/10/10/10/00	747.77	1777.1077.1		1,000,000			100000	01000	100,000,000	100000000	10000000

### **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



VILLAGE NEST JJH

Project ID: 2017-0610 MULTI UNIT PROJECT

Printed: 7 JUL 2017, 3:27PM

Steel Beam

File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29 Licensee: RICHWOND HOFFWAYER

Description:

Lic. #: KW-06002886

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		



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 7 JUL 2017, 3:32PM

19,

Steel Column

File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29 Licensee: RICHMOND HOFFMAYER

Lic. # : KW-06002886

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 E: Elastic Bending Modulus

36.0 ksi 29,000.0 ksi Overall Column Height Top & Bottom Fixity 10.0 ft 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** 

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 =

Load Combination +D+S
Location of max.above base 9.933 ft
At maximum location values are . . .
Pa : Axial 100.008 k

 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

PASS Maximum Shear Stress Ratio =
Load Combination
Location of max.above base
At maximum location values are . . .

Va : Applied Vn / Omega : Allowable 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

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

Maximum Load Deflections . . .

Along Y-Y 0.1985 in at for load combination : W Only

Along X-X -0.03093 in at

5.034ft above base5.839ft above base

for load combination :+D+S

Load Combination Results

	Maximum Axial +	Maximum Axial + Bending Stress Ratios				atios
Load Combination	Stress Ratio	Status	Location	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

0.9052:1

28.383 k-ft

0.06750 : 1

10.0 ft

3.018 k

44.716 k

+D+0.60W



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

Steel Column Lic. # : KW-06002886 Printed: 7 JUL 2017. 3:32PM File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29 Licensee: RICHMOND HOFFMAYER

Maximum Reactions	5							Note	: Only nor	-zero	reactions a	are lister
Load Combination		Axial Reaction @ Base	X-X Axis F @ Base	Reaction @ Top	k	Y-Y Axis @ Base	Reaction @ Top		d Moments	k-ft		Moments @ Top
			STEE					@ Dase	@ 10	μ	W Dase	@ TOP
D Only +D+L		9.334 16.774	0.018 0.018	0.018 0.018		-0.018 -0.018	0.018 0.018					
+D+Lr		13.362	0.016									
+D+S				0.035		-0.035	0.035					
20070000 <u>7</u> 0		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	FOW	9.334	0.018	0.018		2.982	3.018					
+D+0.750Lr+0.750L+0.4		17.935	0.031	0.031		2.219	2.281					
+D+0.750L+0.750S+0.45	OUW	82.919	0.242	0.242		2.008	2.492					
+0.60D+0.60W	175	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												
ltom	F-4	Axial Reaction	X-X Axis R		k	Y-Y Axis			Moments	k-ft	My - End	Moments @ Top
Item	Extreme Value	@ Base	@ Base	@ Top		@ Base	@ Top	@ Base	@ To	р	@ Base	@ 1 op
Axial @ Base	Maximum	100.008	0.317	0.317		-0.317	0.317					
B	Minimum	400.000	0.047	0.047		5.000	5.000					
Reaction, X-X Axis Base	Maximum	100.008	0.317	0.317		-0.317	0.317					
	Minimum	7.440				20222	020222					
Reaction, Y-Y Axis Base	Maximum	02200000	12022201	12002020		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					
The state of the s	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 Deflection			0.010	0.010		0.010	0.010					
Load Combination	S IOI LOAG COM	Max. X-X Deflect	ion Dista	nce		Max. Y-Y D	aflection	Distance				
D Only		-0.0018 i		0.000	_	-0.002	(1988), C. C. C. C. P. C. J. C. L.					
+D+L						-0.002			ft			
+D+Lr			n 5.8						ft			
			n 5.8			-0.003	in In		ft #			
+D+S		-0.0309 i				-0.031	in		ft			
+D+0.750Lr+0.750L		-0.0030 i				-0.003			ft			
+D+0.750L+0.750S		-0.0236 ii				-0.024	in		ft			
+D+0.60W	-0144	-0.0018 i				0.117	in		ft			
+D+0.750Lr+0.750L+0.4		-0.0030 ii				0.086			ft			
+D+0.750L+0.750S+0.45	OW	-0.0236 ii				0.066			ft			
+0.60D+0.60W		-0.0011 ii	10 MATERIA	202: 1020		0.118	in		ft			
+0.60D		-0.0011 i				-0.001	in		ft			
Lr Only		-0.0016 ii	n 5.83			-0.002	in		ft			
L Only		0.0000 ii				0.000	in		ft			
S Only		-0.0292 in	n 5.83	39 ft		-0.029	in	5.839	ft			
W Only		0.0000 ii	0.00	00 ft		0.199	in	5.034	ft			
	ties : H	SS6x6x3/8										



0.000 in

Project Title: Engineer: Project Descr:

VILLAGE NEST JJH MULTI UNIT PROJECT

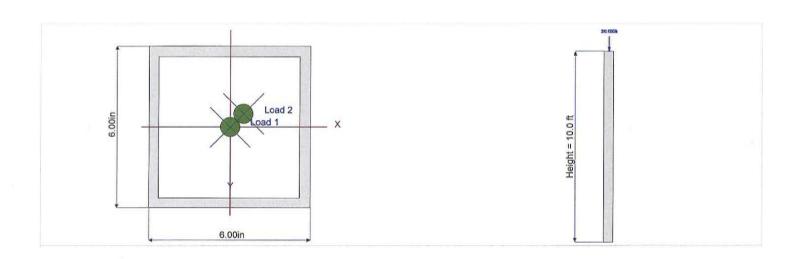
Project ID: 2017-0610

	Printed: 7 JUL 2017, 3:32PM
Steel Column	File = d:\ENERCALC Projects\2017-0610.ec6
Steel Column	ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29
Lic. # : KW-06002886	Licensee: RICHMOND HOFFMAYER

**B21 SUPPORT** Description:

Ycg

Depth	=	6.000 in	l 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	l yy	=	39.500 in^4	C	=	22.100 in^3
Weight	=	27.480 plf	S yy	=	13.200 in^3			
9.93.000/90000000000			R yy	=	2.280 in			
								9



# RICHMOND HOFFMAYER INC. STRUCTURAL ENGINEERING



PROJECT	VILLAGE NEST	JOB	2017-0610
CLIENT	-		
<b>ADDRESS</b>	EDEN, UT		
DESIGN	HUUM		
ENGINEER	J.H.	DATE	6/14/2017
	RICHMOND PE 49428 HOFFMAT	CER SE 3933 CHIEFE	EDA OF

# SIP WALL INFORMATION:

TRY TYPE "S" PANELS

SEISMIL PESIGN CATEGORY DO -TECHOPIL SPECIAL INSCECTION RELOWFED FOR FASTENING AND ANKHOLING OF SHEAR WALLS.

TRY 7-4" TANEL CORE THICKNESS

WONE = (16.60+20L,+264,) 19/2 = 2099 PLF

WUNDES (14.30+40L) 0/2 = 217 PLF

Wines = 2099 + 217 = 2316 PLF

SEE TABLE 5 NEST PAGE, CASE 4326 PLF

The Trimper PLEND HORE INSUL-BERN II

### TABLE 4—UNIFORM TRANSVERSE LOADS FOR FACE SUPPORTED PREMIER TYPE L PANELS<sup>1,2,3</sup> (psf)

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

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

PANEL CORE	PANEL SPAN									
THICKNESS (inches)	8 ft	10 ft	12 ft	16 ft	20 ft	24 ft				
31/2	3,500	2,555	2,450	2,120						
5 <sup>1</sup> / <sub>2</sub>	4,250	4,040	3,375	3,920	2,815					
71/4	4,915	4,325	4,475	4,195	3,495	3,065				
91/4	4,200	4.200	4,200	4,200	3,389	3,247				
1111/4	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.

TABLE 6-ALLOWABLE UNIFORM AXIAL LOADS FOR PREMIER TYPE L PANELS (plf) 1.2.3.4

PANEL CORE						
THICKNESS (inches)	8 ft	10 ft	12 ft	16 ft	20 ft	24 ft
31/2	4,725	3,905	3,095	2,350		
5 <sup>1</sup> / <sub>2</sub>	5,850	5,890	4,280	4,310	2,933	
71/4	6,850	6,110	5,555	5,180	4,835	4,080
91/4	5,470	5,470	5,470	5,470	5,470	4,250
11 <sup>1</sup> / <sub>4</sub>	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.

<sup>&</sup>lt;sup>1</sup>Floor panels must have a minimum <sup>3</sup>/<sub>1-</sub>inch-thick top skin or a minimum <sup>7</sup>/<sub>16</sub>-inch-thick top skin overlaid with minimum <sup>7</sup>/<sub>16</sub>-inch-thick finish flooring perpendicular to the panels.

<sup>&</sup>lt;sup>2</sup>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 11/2-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.

<sup>&</sup>lt;sup>3</sup>Allowable loads with an asterisk, \*, indicates a capacity based on the average peak test load divided by 3.

<sup>&</sup>lt;sup>4</sup>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.  $3\frac{1}{2}$ -inch thick core panels must be limited to a maximum span of 10 feet when used in roof applications.

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

<sup>&</sup>lt;sup>7</sup>7<sup>1</sup>/<sub>4</sub>-inch thick core panels must be limited to a maximum span of 18 feet when used in roof applications.  $^{8}$ 9 $^{1}$ /<sub>4</sub> and 11 $^{1}$ /<sub>4</sub>-inch thick core panels shall be limited to a maximum span of 20 feet when used in roof applications.

<sup>&</sup>lt;sup>1</sup>For the allowable axial load on the fire-resistance-rated assembly, see Section 4.2.6.

<sup>&</sup>lt;sup>2</sup>For combined loads; requirements in Section 4.1 must be applied.

<sup>&</sup>lt;sup>3</sup>The tabulated loads are uniform axial loads applied concentrically to the full thickness of the panels, including panel facings.

<sup>4</sup>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.

<sup>&</sup>lt;sup>1</sup>For the allowable axial load on fire-resistance-rated assembly, see Section 4.2.7.

<sup>&</sup>lt;sup>2</sup>For combined loads; requirements in Section 4.1 must be applied.

<sup>&</sup>lt;sup>3</sup>The tabulated loads are uniform axial loads applied concentrically to the full thickness of the panels, including panel facings.

<sup>&</sup>lt;sup>4</sup>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

		Wal	Loads (ps	f) - End Zor	e (Zone 5)	for 100sf t	o 500 sf ef	fective wir	d area	· · · · · · · · · · · · · · · · · · ·			
Mean Roof		90 MPH			100 MPH			110 MPH			120 MPH		
Height (ft)	Ехр В	Ехр С	Exp D	Exp B	Exp C	Exp D	Ехр В	Exp C	Exp D	Exp B	Ехр С	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		

		Wal	I Loads (ps	f) - End Zor	ne (Zone 5)	for 100sf	to 500sf ef	fective win	d area				
Mean Roof		130 MPH			140 MPH			150 MPH			170 MPH		
Height (ft)	Exp B	Exp C	Exp D	Ехр В	Exp C	Exp D	Exp B	Exp C	Exp D	Ехр В	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	Deflection			Hea	der Span	(ft.)	,.	
Trimmer Studs	Deflection	2'	3'	4'	5'	6'	7'	8'
	L/480	3150	2100	1575	1.260	1050	900	788
1	L/360	3150	2100	1.575	1260	1050	900	788
	L/240	3150	2100	1575	1260	1050	900	788
	L/480	6300	4200	3150 (	2520	2100	1800	1545
2	L/360	6300	4200	3150	2520	2100	1800	1575
	L/240	6300	4200	3150	2520	2100	1800	1575

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

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

e o. Maximum Anowable Axiai C	Jumpression Point Loa	ius (ius) – Type o Pai	1612
Top Plate	1.5" Minimum	3" Minimum	
Configuration	Bearing Width /	Boorina-Width	
Single 2x4 #2 or Better	2040 .		Top Plate –
Hem-Fir Plate	2040	2450	
Single 2x4 #2 or Better			
Hem-Fir Plate	4030	4678	
with 1-1/8 in. wide,	4030	4070	
1.3E Rim Board Cap Plate	The state of the s		
Top plate secured to facings as req	uired in Section 6.3		
2 5	at a feet to a second of the		

Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.

2099 PLF × 19/12

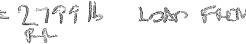


Figure 3: Top Plate Configurations

Continuous

Top Plate

Cap Plate

Table 7: Maximum Allowable Uniform SIP Header Vertical Loads (plf)

Header	Deflection		Header S	Span (ft)	
Depth <sup>3</sup> (in)	Limit <sup>4</sup>	4	6	8	10
	L/480	740	384	228	142
12	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
	L/480	886	629	429	361
24	L/360	886	629	429	361
	L/240	886	629	429	361

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<sup>1,2</sup> Header Deflection Header Span (ft) Depth<sup>3</sup> (in) Limit<sup>4</sup> 10 8 4 6 L/480 345 243 156 99 125 12 L/360 450 295 190 L/240 630 382 236 153 L/480 705 388 254 235 750 302 18 L/360 482 281 L/240 750 482 302 281 L/480 698 556 368 350 24 L/360 896 556 368 350 L/240 896 556 368 350

SIPs wall panel spline minimum 6" from edge of opening

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

Concentrated loads shall be applied concentrically to the top of the panel.

<sup>&</sup>lt;sup>4</sup> Tabulated values are based on the strong-axis of the facing material oriented parallel to the span direction.

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.

Tabulated values are based on the strong-axis of the facing material oriented perpendicular to the direction of header span.

<sup>&</sup>lt;sup>3</sup> Minimum depth of facing above opening.

<sup>&</sup>lt;sup>4</sup> Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code.

tabulated load.

<sup>2</sup> Tabulated values are based on the strong-axis of the facing material oriented perpendicular to the direction of header span.

Minimum depth of facing above opening.
 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	<sub>1</sub> 6/16/2017
	RICHMOND PE 49628 HOFFMAYER SE 3935	SHEET	198 of

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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	77 of

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→ →	Foundation		
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### SEISMIC DESIGN:

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		GOOGLE EARTH LATITUDE LONGTITUDE	N W	=	41.3600			
		USGS NSHMP - 2012 IBC						
		BASED ON SITE CLASS B, Fa = 1.0 AND Fv = 1.0	Ss S1	=	0.806 0.267			
16CBC /								
303.1.5		THQUAKE DESIGN DATA SEISMIC IMPORTANCE FACTOR AND OCCUPANCY OCCUPANCY	I	=	1.00 II		ASCE 7-10 TABLE 11.5-1	
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	3	SITE CLASS SITE COEFFICIENTS	1.00 Fv	=-	D 1.18 1.87		IBC DEFAULT TABLE 11.4-1 TABLE 11.4-2	
	4	SPECTRAL RESPONSE COEFFICIENTS	SMS SM1 SDS SD1	=======================================	0.949 0.498 0.633 0.332	= 2/3 SMS = 2/3 SM1	ASCE 7-10 (11.4-3) ASCE 7-10 (11.4-4)	
				-	200			
Nessel Property	5	SEISMIC DESIGN CATEGORY			D	<u>-</u>	IBC2012 TABLES 1613.5.6(1) & 1613.5.6(2)	
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	Ÿ	A. BEARING WALL SYSTEMS						
		A-LIGHT-FRAMED WALLS SHEATHED WITH WOOD P.	ANELS R	=	6.50		ASCE 7-10 TABLE 12:2-1	
			Ωο	Ţ-	3.00	<del>-</del>	ASCE 7-10 TABLE 12.2-1	
			Cd	= -	4.00	Ť.	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)	
			Cu	=	Cu Ta		ASCE 7-10 TABLE 12.8-1	
				_	1.40 20.00	ft	ASCE 7-10 TABLE 12.6-1	
			hn Ct	_	0.02	п	ASCE 7-10 TABLE 12.8-2	
			×	=	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 0.010	MAX MIN	ASCE 7-10 (12.8-3) ASCE 7-10 (12.8-5)	
			Cs"	_	0.010	WIN,	ASCE 7-10 (12.8-6)	
			Cs US		0.097		Cs OVERIDE 0.000	
		RESPONSE MODIFICATION FACTOR	R	= _	6.50			
	10	ANALYSIS PROCEDURE USED	V	= =	0.0973	IT LATERAL FO	RCE PROCEDURE X-DIRECTION BASE SHEAR FORCE	
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		BASIC SEISMIC-FORCE-RESISTING SYSTEM A. BEARING WALL SYSTEMS						
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			R	=_	6.50	-	ASCE 7-10 TABLE 12.2-1	
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		SEISMIC RESPONSE COEFFICIENT	Cs	=	0.097	= SDS / (R / I)	ASCE 7-10 (12.8-2)	
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			×	_	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 Cel	=	8.00	S	ASCE 7-10 FIG.22-16	
			Cs'	=	0.193 0.010	MAX MIN	ASCE 7-10 (12.8-3) ASCE 7-10 (12.8-5)	
			Cs'''	=	0.010	WIN.	ASCE 7-10 (12.8-6)	
			Cs US		0.097	201770000	Cs OVERIDE 0.000	¥.
		RESPONSE MODIFICATION FACTOR	R	=	6.50			
	10	ANALYSIS PROCEDURE USED					RCE PROCEDURE	
			V	=_	0.0973	W	Y-DIRECTION BASE SHEAR FORCE	



AREA WEIGHTS - 2ND LEVEL:

ROOF

9.00 ft WALL HT.

WEIGHT TABULATIONS:

2Y = 91676.91 lb 2X = 88628.16 lb

The contract of the contract o		THE RESIDENCE OF THE PROPERTY OF THE PERSON	Section and the second section		
K.	METAL ROOF	15.76 psf	_	EXTERIOR	15.23 psf
മ	OPENED BALCONY	7.00 psf	>	INTERIOR	8.85 psf
೦	2nd FLOOR	14.25 psf	3	t	0.00 pst
<u>_</u>	DRIVEWAY LOADS	85.38 psf	×	ı	0.00 psf
ш	20% SNOW LOAD FOR SEISM	52.80 psf	>		0.00 psf
ا علا	GARAGE LOADS	63.50 psf	Z	i	0.00 psf

TOTAL WEIGHT	24000.96 lb	64627.20 lb	0.00 lb	dl 00:0	88628,16 lb	27049.71 lb	64627.20 lb	0.00 lb	0.00 lb	91676.91 lb	0.00 lb	0.00 lb	0.00 lb	0.00 lb	dl 00:0	0.00 lb	0.00 lb	0.00 lb	ql 00:0	al 00:0	0:00 lb	al 00:0	0.00 lb	0.00 lb	ql 00:0	al 00:0	0.00 lb	0.00 lb	al 00:00 al 00:00
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NUMBER	1.00	0.00	0.00			00'00	0.00	0.00	00.00		0.00	00.00		0.00		00.00	00.00	0.00	00.00		0.00	00'00	00.00	00.00		0.00	0.00	0.00	0.00
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		×	×	×		×	×	×	×		×	×	×	×		×	×	×	×		×	×	×	×		×	×	×	×
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AREA WEIGHTS - 1ST LEVEL:

WEIGHT TABULATIONS:

1Y = 18564.00 lb 1X = 18564.00 lb

2ND FLOOR

9.00 ft WALL HT.

WEIGHT TYPES:

15.23 psf 8.85 psf 0.00 psf 0.00 psf 0.00 psf EXTERIOR INTERIOR 15.76 psf 7.00 psf 14.25 psf 85.38 psf 52.80 psf 63.50 psf 2nd FLOOR DRIVEWAY LOADS 20% SNOW LOAD FOR SEISM GARAGE LOADS METAL ROOF OPENED BALCONY ворши

1-3 3 1% 14.25 pst C x 2/00 ft + 16.29 pst V x 1000 ft x 200 pst · x 000 ft x 000 pst · x 000	W# F#	UNIT WEIGHT TYPE	(E)	LENGTH	UNIT WE	UNIT WEIGHT TYPE	T	EIGHT	NUMBER		UNIT WEIGHT	IT TYPE	ľ	HEIGHT	NUMBER		TRIB. WEIGHT	WIDTH		OTAL WEIGHT
+ 700 psf B				3		sf U	×			+	8.85 psf	>	×	ı=	1.00	Name of the last o	9		# == #	17640.00 lb
+ 0.000 psf - x 0.000 ft + 0.000 psf - x 0.000 ft x 0.000 psf - x 0.000 ft x 0.000 psf - x 0.000 psf - x 0.000 ft x 0.000	+		×	5.50 ft +	0.00 p	sf .	×			+	0.00 psf	ı	×		00.00	Ħ			#	924 00 lb
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14.25 psf C	+	0.00 psf -	×	00'00 <b>tf</b> +	0.00 p	sf .	×			+	0.00 psf	ı	×		00.00	11			: #	0.00 B
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x 0.00 ft + 0.00 psf - x 0.00 ft x 0.00 psf - x 0.00 psf																				924.00 lb
x 0.00 ft + 0.00 psf - x 0.00 ft x 0.00 psf - x		0.00 psf -	×	0.00 ft +	0.00 p	ıst .	×	0.00 ft x	00.00	+		ŧ	×	0.00 ft ×	0.00	Ħ			#	d 00 0
- x 0.00 ft + 0.00 psf - x 0.00 ft x 0.00 psf - x 0.00 ps	+		×	0.00 ft +	0.00 p	isf .	×	#		+		1	×	#	00.00	, si			==	al 00:0
x 0.00 ft + 0.00 psf - x 0.00 ft x 0.00 psf - x 0.00 ft x 0.00 psf - x 0.00 ft x 0.00 psf x 0.00 ft x 0.00 psf -	+		×	0.00 ft +	0.00 F	ıst -	×	0.00 ft x	0.00	+		,	×	#	00.0	H		× 0.	#	0.00 lb
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	+		×	0.00 # +		sf -	×		0.00	+		1	×	<b>#</b>	0.00	Ħ			#	0.00 lb
$\begin{array}{cccccccccccccccccccccccccccccccccccc$																			grand the second second	dl 00.0
$\frac{1}{2}$ x 0.00 ft + 0.00 psf - x 0.00 ft x 0.00 + 0.00 psf - x 0.00 ft x 0.00 psf - 0		0.00 psf	×	0.00 ft +	0.00	sf	×	0.00 ft x	00.00	+			×		00.00	II		0	#	0.00 lb
$\sim$ x 0.00 ft + 0.00 psf - x 0.00 ft x 0.00 psf - x 0.00 ft x 0.0	+		×	0.00 ft +	0.00	st -	×	0.00 ft ×	00.00	+	0.00 psf	1	×	0.00 ft ×	00'0	11		× 0	Ħ	0.00 lb
. x 0.00 ft + 0.00 psf - x 0.00 ft x 0.00 + 0.00 psf - x 0.00 ft x	+		×	0.00 ft +	9 00.00	st -	×	0.00 ft ×	0.00	+		ì	×		00.0	Н		0	#	0.00 lb
	+	0.00 psf -	×		0.00 F	sf -	×		0.00	+		r	×	<b>#</b>	00.00	н			#	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 X-DIRECTION SEISMIC RESPONSE COEFICIENT STRENGTH LEVEL BASE SHEAR FORCE - X	Wx Csx Vx	= =	107192.16 lb 0.0973 10434.97 lb	SEE PREVIOUS PAGES Rx =	6.50
	TOTAL BUILDING DEAD WEIGHT - Y DIRECTION Y-DIRECTION SEISMIC RESPONSE COEFICIENT STRENGTH LEVEL BASE SHEAR FORCE - Y	Wy Csy Vy	=	110240.91 lb 0.0973 10731.76 lb	SEE PREVIOUS PAGES Ry =	6.50

ASCE 7-10

12.8.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES

PER EACH ORTHOGONAL DIRECTION x & y

F = Cv V  $Cv = w h^k$   $\Sigma wi hi^k$ 

ASCE 7-10 (12.8-11) ASCE 7-10 (12.8-12)

LEVEL	wx	hx	hx ^ k	wx hx ^ ƙ	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
- 1	0.00	0.00	0.000	0.00	0.000
2	88628.16	20.00	20.000	1772563.12	0.905
1	18564.00	10.00	10.000	185640.00	0.095
SUM	107192.16			1958203.12	1.000

CTOCNOTI		ti
STRENGTH		
SHEAR		DISTRIBUTION
FORCE PER		FACTOR
LEVEL Fx	Wx Csx	Fx/(WxCsx)
0.00	0.00	0.000
0.00	0.00	0.000
0.00	0.00	0.000
0.00	0.00	0.000
9445.72	8627.80	1.095
989.25	1807.17	0.547
10434.97		

Y-DIRECTION BUILDING PERIOD T = 0.2648 s SEE PREVIOIUS PAGES k =  $\frac{1.00}{}$  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	91676.91	20.00	20.000	1833538.12	0.908
1	18564.00	10.00	10.000	185640.00	0.092
SUM	110240.91			2019178.12	1.000

STRENGTH		
SHEAR		DISTRIBUTION
FORCE PER		FACTOR
LEVEL Fy	Wy Csy	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
9745.10	8924.59	1.092
986.66	1807.17	0.546
10731.76		



### **SEISMIC FORCES:**

4TH LEVEL FORCES										
F#	TOTAL TRIB. WEIGHT	DIST. FAC	CTOR	Cs	D	ESIGN SHEAR FO	ORCE (STR	ENGTH LEVEL )	DISTRIBUTION OVERIG	E R OVERIDE
	0.00 plf x	0.00	Х	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	х	0.0000	=	0.00 plf	R =	0.00		
	0.00 plf x	0.00	х	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	×	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	х	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#	NOVEMBER OF THE STREET OF THE PROPERTY OF THE	DIST. FAC	CTOR	Cs	D	ESIGN SHEAR FO	eromonomica acument		DISTRIBUTION OVERIG	E R OVERIDE
	0.00 plf x	0.00	Х	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	х	0.0000	=	0.00 plf	R =	0.00		
	0.00 plf x	0.00	х	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 pif 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	х	0.0000	=	0.00 plf	R =	0.00		
	0.00 plf x	0.00	X	0.0000	=	0.00 plf	R <b>=</b>	0.00		1
	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 15	R =	0.00		
	0.00 pii X	0.00		0.0000		0.00 plf	11 -	0.00		
	•	0.00	-	0.0000		U.UU pir	11 -	0.00		
2ND LEVEL FORCES	ROOF									
F#	ROOF TOTAL TRIB. WEIGHT I	DIST. FAC	CTOR	Cs	D	ESIGN SHEAR FO	RCE ( STRE	ENGTH LEVEL )	DISTRIBUTION OVERID	E R OVERIDE
F# 1 2X	ROOF TOTAL TRIB. WEIGHT 3692.84 plf x	DIST. FAC 1.09	TOR x	Cs 0.0973	Di =	ESIGN SHEAR FO 393.57 plf	PRCE ( STRE R =	ENGTH LEVEL ) 6.50	DISTRIBUTION OVERID	E R OVERIDE
F# 1 2X 2 2Y	ROOF TOTAL TRIB. WEIGHT I 3692.84 plf x 1797.59 plf x	DIST. FAC 1.09 1.09	TOR X X	Cs 0.0973 0.0973	D:====================================	ESIGN SHEAR FO 393.57 plf 191.08 plf	PRCE ( STRE R = R =	NGTH LEVEL ) 6.50 6.50	DISTRIBUTION OVERID	E R OVERIDE
F# 1 2X	ROOF TOTAL TRIB. WEIGHT I 3692.84 plf x 1797.59 plf x 0.00 plf x	DIST. FAC 1.09 1.09 0.00	CTOR X X X	Cs 0.0973 0.0973 0.0000	D) = = =	ESIGN SHEAR FC 393.57 plf 191.08 plf 0.00 plf	RCE ( STRE R = R = R =	ENGTH LEVEL ) 6.50 6.50 0.00	DISTRIBUTION OVERID	E R OVERIDE
F# 1 2X 2 2Y	ROOF TOTAL TRIB. WEIGHT I 3692.84 pif x 1797.59 pif x 0.00 pif x 0.00 pif x	1.09 1.09 1.09 0.00 0.00	X X X X	Cs 0.0973 0.0973 0.0000 0,0000	D) = = = =	ESIGN SHEAR FC 393.57 plf 191.08 plf 0.00 plf 0.00 plf	RCE ( STRE R = R = R = R = R =	ENGTH LEVEL ) 6.50 6.50 0.00 0.00	DISTRIBUTION OVERID	E R OVERIDE
F# 1 2X 2 2Y	ROOF TOTAL TRIB. WEIGHT I 3692.84 pif x 1797.59 pif x 0.00 pif x 0.00 pif x 0.00 pif x	0.00 0.00 0.00 0.00	X X X X X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000	DI = = = = =	ESIGN SHEAR FC 393.57 pif 191.08 pif 0.00 pif 0.00 pif 0.00 pif	RCE ( STRE R = R = R = R = R = R =	ENGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00	DISTRIBUTION OVERID	E R OVERIDE
F# 1 2X 2 2Y	ROOF TOTAL TRIB. WEIGHT I 3692.84 plf x 1797.59 plf x 0.00 plf x 0.00 plf x 0.00 plf x 0.00 plf x	1.09 1.09 1.09 0.00 0.00 0.00 0.00	X X X X X X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000	Di	ESIGN SHEAR FC 393.57 plf 191.08 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf	RCE ( STRE R = R = R = R = R = R = R =	6.50 6.50 0.00 0.00 0.00 0.00	DISTRIBUTION OVERID	E R OVERIDE
F# 1 2X 2 2Y	ROOF TOTAL TRIB. WEIGHT  3692.84 plf x 1797.59 plf x 0.00 plf x	1.09 1.09 1.09 0.00 0.00 0.00 0.00 0.00	X X X X X X X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000	Di = = = = = = = = = = = = = = = = = = =	ESIGN SHEAR FC 393.57 plf 191.08 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf	RCE ( STRE R = R = R = R = R = R = R =	NGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	DISTRIBUTION OVERID	E R OVERIDE
F# 1 2X 2 2Y	ROOF TOTAL TRIB. WEIGHT I 3692.84 pif x 1797.59 pif x 0.00 pif x	0.00 1.09 1.09 0.00 0.00 0.00 0.00 0.00	X X X X X X X X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	Di	ESIGN SHEAR FC 393.57 plf 191.08 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf	RCE ( STRE R = R = R = R = R = R = R = R =	ENGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	DISTRIBUTION OVERID	E R OVERIDE
F# 1 2X 2 2Y	ROOF TOTAL TRIB. WEIGHT I 3692.84 pif x 1797.59 pif x 0.00 pif x	0.00 1.09 1.09 0.00 0.00 0.00 0.00 0.00	CTOR X X X X X X X X X X X X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	D = = = = = = = = = = = = = = = = = = =	ESIGN SHEAR FC 393.57 plf 191.08 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf 0.00 plf	RCE ( STRE R = R = R = R = R = R = R = R = R =	ENGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	DISTRIBUTION OVERID	E ROVERIDE
F# 1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  3692.84 plf	0.00 1.09 1.09 0.00 0.00 0.00 0.00 0.00	CTOR X X X X X X X X X X X X X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	D = = = = = = = = = = = = = = = = = = =	ESIGN SHEAR FC 393.57 plf 191.08 plf 0.00 plf	RCE ( STRE R = R = R = R = R = R = R = R = R = R =	ENGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	DISTRIBUTION OVERID	E R OVERIDE
F# 1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  3692.84 pif x  1797.59 pif x  0.00 pif x	1.09 1.09 0.00 0.00 0.00 0.00 0.00 0.00	X X X X X X X X X X X X X X X X X X X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	Di	ESIGN SHEAR FC 393.57 plf 191.08 plf 0.00 plf	PRCE (STRE R = R = R = R = R = R = R = R = R = R =	NGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	DISTRIBUTION OVERID	E R OVERIDE
F# 1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT I  3692.84 plf x  1797.59 plf x  0.00 plf x	1.09 1.09 1.09 0.00 0.00 0.00 0.00 0.00	X X X X X X X X X X X X X X X X X X X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	Di	ESIGN SHEAR FC 393.57 plf 191.08 plf 0.00 plf	PRCE (STRE R = R = R = R = R = R = R = R = R = R =	ENGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	DISTRIBUTION OVERID	E ROVERIDE
F# 1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT I  3692.84 pif x 1797.59 pif x 0.00 pif x	1.09 1.09 0.00 0.00 0.00 0.00 0.00 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000		ESIGN SHEAR FC 393.57 plf 191.08 plf 0.00 plf	PRCE (STRE R = R R = R R = R R R R R R R R R R R R	ENGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	DISTRIBUTION OVERID	E ROVERIDE
F# 1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT I  3692.84 plf x  1797.59 plf x  0.00 plf x	1.09 1.09 1.09 0.00 0.00 0.00 0.00 0.00	X X X X X X X X X X X X X X X X X X X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	Di	ESIGN SHEAR FC 393.57 plf 191.08 plf 0.00 plf	PRCE (STRE R = R = R = R = R = R = R = R = R = R =	ENGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	DISTRIBUTION OVERID	E ROVERIDE
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  3692.84 pif x  1797.59 pif x  0.00 pif x	1.09 1.09 0.00 0.00 0.00 0.00 0.00 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000		ESIGN SHEAR FC 393.57 plf 191.08 plf 0.00 plf	PRCE (STRE R = R R = R R = R R R R R R R R R R R R	ENGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	DISTRIBUTION OVERID	E ROVERIDE
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT	1.09 0.00 0.00 0.00 0.00 0.00 0.00 0.00	X X X X X X X X X X X X X X X X X X X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	D = = = = = = = = = = = = = = = = = = =	ESIGN SHEAR FC 393.57 plf 191.08 plf 0.00 plf	PRCE (STRE R = R = R = R = R = R = R R R R R R R R	NGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00		
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT I  3692.84 plf x  1797.59 plf x  0.00 plf x  2000 plf x  0.00 plf x	1.09 0.00 0.00 0.00 0.00 0.00 0.00 0.00	X X X X X X X X X X X X X X X X X X X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DI	ESIGN SHEAR FO 393.57 plf 191.08 plf 0.00 plf	RCE (STRE	ENGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00	DISTRIBUTION OVERID	
F#  1 2X 2 2Y	ROOF TOTAL TRIB. WEIGHT I 3692.84 plf x 1797.59 plf x 0.00 plf x	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DI D	ESIGN SHEAR FO 393.57 plf 191.08 plf 0.00 plf	RCE (STRE	ENGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00		
F#  1 2X 2 2Y 1ST LEVEL FORCES  F#  3 1X 4 1Y	ROOF  TOTAL TRIB. WEIGHT I  3692.84 pif x  1797.59 pif x  0.00 pif x  200 pif x  0.00 pif x	1.09 1.09 0.00 0.00 0.00 0.00 0.00 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DD D	ESIGN SHEAR FO  393.57 plf 191.08 plf 0.00 plf	RCE (STRE	NGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00		
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  3692.84 pif x  1797.59 pif x  0.00 pif x  20.00 pif x  10.00 pif x	0.55 0.55 0.55 0.55 0.55	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DD	ESIGN SHEAR FO 393.57 plf 191.08 plf 0.00 plf	RCE (STRE	NGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00		
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT I  3692.84 plf x  1797.59 plf x  0.00 plf x  200 plf x  0.00 plf x	0.55 0.55 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000 0.0000	DE = = = = = = = = = = = = = = = = = = =	ESIGN SHEAR FO 393.57 plf 191.08 plf 0.00 plf	PRCE (STRE	NGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00		
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT  3692.84 plf	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000	DI D	ESIGN SHEAR FO  393.57 plf 191.08 plf 0.00 plf	RCE ( STRE	NGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00		
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT I 1797.59 plf x 0.00 plf x 168.00 plf x 168.00 plf x 0.00 plf x 0.000 plf x 0.0	1.09 0.00 0.00 0.00 0.00 0.00 0.00 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000	DDD	ESIGN SHEAR FO 393.57 plf 191.08 plf 0.00 plf	RCE (STRE	NGTH LEVEL )  6.50  6.50  0.00		
F#  1 2X 2 2Y 1ST LEVEL FORCES  F#  3 1X 4 1Y	ROOF  TOTAL TRIB. WEIGHT  3692.84 pif x  1797.59 pif x  0.00 pif x  200 pif x  0.00 pif x  168.00 pif x  0.00 pif x  0.00 pif x  0.00 pif x	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000	DI D	ESIGN SHEAR FO 393.57 plf 191.08 plf 0.00 plf 39.06 plf 8.93 plf 0.00 plf 0.00 plf 0.00 plf	PRCE (STREET RESERVED	NGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00		
F#  1 2X 2 2Y	ROOF  TOTAL TRIB. WEIGHT	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000	DDD	ESIGN SHEAR FO 393.57 plf 191.08 plf 0.00 plf	PRCE (STRE R = R = R R R R R R R R R R R R R R R R	NGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00		
F#  1 2X 2 2Y 1ST LEVEL FORCES  F#  3 1X 4 1Y	ROOF  TOTAL TRIB. WEIGHT  3692.84 plf	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0900 0.0000	D)	ESIGN SHEAR FO  393.57 plf 191.08 plf 0.00 plf	RCE ( STRE	NGTH LEVEL )  6.50  6.50  0.00		
F#  1 2X 2 2Y 1ST LEVEL FORCES  F#  3 1X 4 1Y	ROOF  TOTAL TRIB. WEIGHT  3692.84 pif x  1797.59 pif x  0.00 pif x  2.00 pif x  0.00 pif x  168.00 pif x  0.00 pif x	0.55 0.55 0.00 0.00 0.00 0.00 0.00 0.00	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0900 0.0000	DI D	ESIGN SHEAR FO 393.57 plf 191.08 plf 0.00 plf	PRCE (STREET REPORTED FOR THE PROPERTY OF THE	NGTH LEVEL )  6.50  6.50  0.00		
F#  1 2X 2 2Y 1ST LEVEL FORCES  F#  3 1X 4 1Y	ROOF  TOTAL TRIB. WEIGHT  3692.84 pif x  1797.59 pif x  0.00 pif x  200 pif x  0.00 pif x  168.00 pif x  0.00 pif x	DIST. FAC 1.09 0.00 0	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000	DE	ESIGN SHEAR FO 393.57 plf 191.08 plf 0.00 plf	PRCE (STREET REPORTED FOR THE PROPERTY OF THE	NGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00		
F#  1 2X 2 2Y 1ST LEVEL FORCES  F#  3 1X 4 1Y	ROOF  TOTAL TRIB. WEIGHT  3692.84 plf x  1797.59 plf x  0.00 plf x  200 plf x  0.00 plf x  773.50 plf x  735.00 plf x  168.00 plf x  0.00 plf x	0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.0	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000	DDD	ESIGN SHEAR FO 393.57 plf 191.08 plf 0.00 plf	PRCE (STREET RECE) RCE (STREET RECE) RCE (STREET RECE) RCE RCR RCR RCR RCR RCR RCR RCR RCR RCR	NGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00		
F#  1 2X 2 2Y 1ST LEVEL FORCES  F#  3 1X 4 1Y	ROOF  TOTAL TRIB. WEIGHT  3692.84 pif x  1797.59 pif x  0.00 pif x  200 pif x  0.00 pif x  168.00 pif x  0.00 pif x	DIST. FAC 1.09 0.00 0	CTOR  X  X  X  X  X  X  X  X  X  X  X  X  X	Cs 0.0973 0.0973 0.0000	DE	ESIGN SHEAR FO 393.57 plf 191.08 plf 0.00 plf	PRCE (STREET REPORTED FOR THE PROPERTY OF THE	NGTH LEVEL ) 6.50 6.50 0.00 0.00 0.00 0.00 0.00 0.00		



### SEISMIC FORCES TO SHEAR LINES:

2ND LEVEL

ROOF

INE		FORCES F#			FORCE			WIDTH		X BAR		T.	_	R DESIGN	R REQ'	0	% LOA	EE	ECT	_	
				,			10		440		,	-						FLL			4722 00 11
1	- 2	F1	(4)	(	393.57 plf		) ×	24.00	×	1.00	'.	2.00	×	6.50	6.50	×	1.00	- 1	1.00	=	4722.86 II
	+	7	*	(			) ×	0.00	×	0.00	′.	0.00	×	0.00	0.00	×	1.00	1	1.00	=	0.00 1
	+		*	(			) x	0.00	×	0.00	1	0.00	×	0.00		×	1.00	- 1	1.00	=	0.00 1
	+		7	(			) ×	0.00	×	0.00	/	0.00	×	0.00	0.00	×	1.00	- /	1.00	=	0.00 1
	+	-	-	(			) ×	0.00	×	0.00	1	0.00	×	0.00	0.00	×	1,00	1	1.00	=	0.00 1
	+			(			) ×	0.00	×	0.00	1	0.00	X	0.00	0.00	×	1.00	1	1,00	**	0.00 1
	+	-			(	-	) ×			0.00	1	0.00	×	0.00	0.00	×	1.00	1	1.00	=_	0.00 1
														6.50							4722.86 It
HEA	RLINE	FORCES														_					
NE		F#			FORCE			WIDTH		X BAR		L		R DESIGN	R REQ'		% LOAI				
2		F1	19	(	393.57 plf		) ×	24.00	×	1.00	1	2.00	X	6.50	6.50	X	1.00	1	1,00	==	4722.86 II
	+	-		(			) ×	0.00	×	0.00	1	0.00	×	0.00	0.00	×	1,00	1	1.00	=	0.00 1
	+	-	-	(			) ×	0.00	×	0.00	1	0.00	×	0.00	0.00	×	1.00	1	1.00	=	0.00 1
	+	160	-	(			) ×	0.00	×	0.00	1	0.00	×	0.00	0.00	×	1.00	1	1.00	=	0.00 18
	+	2	-	(			) x	0.00	×	0.00	1	0.00	×	0.00	0.00	×	1.00	1	1.00	=	0.00 1
	+	- ·		(			) x	0.00	×	0.00	1	0.00	×	0.00	0.00	×	1.00	1	1.00		0.00 18
9	+	-			(	2	)×			0.00	1	0.00	×	0.00		×	1.00	1	1.00	=	0.00 18
					(4		1000							6,50							4722.86 II
IEA	RLINE	FORCES			100001 to 27 P	_	_	MANUAL MA		- 950555555			_		ALTO BE SOUTH		02.04.04.04.04.04.04.04.04.04.04.04.04.04.				
1E	-	F#	1134		FORCE		2000	WIDTH	*	X BAR		L		R DESIGN	R REQ'I		% LOAI	EFF			VVILAMORALAVIV
3		F2		(	191.08 plf		) ×	18.00	×	1.00	1	2.00	×	6.50	6.50	×	1.00	1	1.00	=	1719.72 II
	+	-	(m)	(			) ×	0.00	×	0.00	1	0.00	×	0.00	0.00	×	1.00	1	1.00	=	0.00 1
	+		-	(			) ×	0.00	×	0.00	1	0.00	×	0.00	0.00	X	1.00	1	1.00	=	0.00 1
	+		-	(			) ×	0.00	×	0.00	1	0.00	×	0.00	0.00	X	1.00	1	1,00	=	0.00 1
	+	-	-	(			) ×	0.00	×	0.00	- /	0.00	×	0.00	0.00	×	1.00	1	1.00	=	0.00 lb
	+	-	-	(			) ×	0.00	×	0.00	1	0.00	×	0.00	0.00	x	1.00	1	1.00	=	0.00 18
	+	-		- 8	(	-	) x			0.00	1	0.00	x	0.00	0.00	x	1.00	1	1.00	=	0.00 18
					7.0		#.co							6.50							1719.72 lb
IEA	R LINE I	FORCES																			
VE		F#		1586	FORCE		2000	WIDTH	499	X BAR	11/05	L	100	R DESIGN	R REQ'		% LOAE	EFF	The latest will		ANCHARCAS
		F2	2	(	191.08 plf		) ×	18,00	×	1.00	1	2.00	×	6.50 /	6,50	×	1.00	1	1.00	=	1719.72 lb
4	4	F2	-	(	191.08 plf		) ×	25.00	×	1.00	1	2.00	×	6.50 /	6.50	×	1.00	1	1,00	=	2388.50 lb
4							) x	0.00	×	0.00	1	0.00	×	0.00 /	0.00	×	1.00	1	1.00	=	0.00 lb
4	+	20	-	(			100			0.00	1.0	0.00	×	0.00 /	0.00	×	1.00	1	1.00		0.00 lb
4	+	5	7	(			) x	0.00	X	0.00	10	0.00	^	0.00	0.00						
4	+ + +		-	(			5.25	0.00	×	0.00	,	0.00	x	0.00 /	0.00	×	1.00	1	1.00	=	0.00 lb
4	+ + +	5		(			) x ) x	10.00		55,07,55,0	,		x	/F/5/F 3	0.00	×		1	1.00	=	
4	+ + + +		10.4	(			) × ) × ) ×	0.00	×	0.00	,	0.00	×	0.00 /	0.00	×	1.00	1	1.00		0.00 lb
4	+ + + + +			(	(		) x ) x	0.00	×	0.00	,	0.00	x	0.00 /	0.00		1.00	1		=	0.00 lb 0.00 lb 0.00 lb 4108.23 lb
100	+ + + + + +	FORCES		(	(		) × ) × ) ×	0.00	×	0.00	;	0.00	×	0.00 / 0.00 / 0.00 /	0.00	×	1.00	1	1.00	=	0.00 lb
IEAF	+ + + + + +	F#		(	( FORCE		) × ) × ) × ) ×	0.00 0.00 WIDTH	×	0.00 0.00 0.00	; ;	0.00 0.00 0.00	x x x	0.00 / 0.00 / 0.00 / 6.50	0.00 0.00 0.00	×	1.00 1.00 1.00	1	1.00 1.00	=_	0.00 lt 0.00 lt 4108.23 lt
10.4	+ + + + + +	F# F2		(	191.08 plf		) × ) × ) × ) ×	0.00 0.00 WIDTH 25.00	×	0.00 0.00 0.00 X BAR 1.00	; ;	0.00 0.00 0.00	×××	0.00 / 0.00 / 0.00 / 6.50 /	0.00 0.00 0.00 R REQ'D	× × ×	1.00 1.00 1.00	1	1.00 1.00	= _	0.00 R 0.00 R 4108.23 R
IEAF	+ + + + + R LINE F	F#	10 4 3 5 4 5 4 5 4 5 4 5 4 5 4 5 4 5 4 5 4	( (			) × ) × ) × ) ×	0.00 0.00 WIDTH 25.00 5.50	×	0.00 0.00 0.00 X BAR 1.00 27.75	, , ,	0.00 0.00 0.00 L 2.00 24.00	x x x	0.00 / 0.00 / 0.00 / 6.50 / R DESIGN 6.50 / 6.50 /	0.00 0.00 0.00 R REQ'E 6.50 6.50	× × ×	1.00 1.00 1.00 4.00 1.00	1	1.00 1.00 ECT 1.00 1.00	= _	0.00 lt 0.00 lt 4108.23 lt 2388.50 lt 1215.15 lt
IEAF	+ + + + + + R LINE F	F# F2		( ( (	191.08 plf		) × ) × ) × ) ×	0.00 0.00 WIDTH 25.00	×	0.00 0.00 0.00 X BAR 1.00	', ', ',	0.00 0.00 0.00	×××	0.00 / 0.00 / 0.00 / 6.50 / R DESIGN 6.50 / 6.50 / 0.00 /	0.00 0.00 0.00 R REQ'D	× × ×	1.00 1.00 1.00	1	1.00 1.00 ECT 1.00 1.00	= = = = = = = = = = = = = = = = = = = =	0.00 II 0.00 II 4108.23 II 2388.50 II 1215.15 II 0.00 II
IEAF	+ + + + + + + + + + +	F# F2 F2	1 1 1 1 1	( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	191.08 plf		) × ) × ) × ) ×	0.00 0.00 WIDTH 25.00 5.50	×××	0.00 0.00 0.00 X BAR 1.00 27.75	', ', ',	0.00 0.00 0.00 L 2.00 24.00	x x x	0.00 / 0.00 / 0.00 / 6.50 / R DESIGN 6.50 / 6.50 /	0.00 0.00 0.00 R REQ'E 6.50 6.50	× × ×	1.00 1.00 1.00 4.00 1.00	1	1.00 1.00 ECT 1.00 1.00	= _	0.00 lt 0.00 lt 4108.23 lt 4108.23 lt 2388.50 lt 1215.15 lt 0.00 lt
IEAF	+ + + + + + + + + + + + + + + + + + +	F# F2 F2		( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	191.08 plf	-	) x ) x ) x ) x	0.00 0.00 WIDTH 25.00 5.50 0.00	x x x x	0.00 0.00 0.00 0.00 X BAR 1.00 27.75 0.00	/ / / / / /	0.00 0.00 0.00 0.00	x x x	0.00 / 0.00 / 0.00 / 6.50 / R DESIGN 6.50 / 6.50 / 0.00 /	0.00 0.00 0.00 R REQ'0 6.50 6.50 0.00	x x	1.00 1.00 1.00 1.00 % LOAE 1.00 1.00	1	1.00 1.00 ECT 1.00 1.00	= = = = = = = = = = = = = = = = = = = =	2388.50 II 1215.15 II 0.00 II
IEAF	+ + + + + + + + + + + + + + + + + + +	F# F2 F2		( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	191.08 plf		) x ) x ) x ) x ) x ) x ) x ) x ) x ) x	0.00 0.00 WIDTH 25.00 5.50 0.00 0.00	x x x x x	0.00 0.00 0.00 0.00 X BAR 1.00 27.75 0.00 0.00	' ' ' ' ' ' ' ' ' ' ' ' ' ' ' ' ' ' '	0.00 0.00 0.00 24.00 0.00 0.00	x x x x	0.00 / 0.00 / 0.00 / 6.50 / 6.50 / 6.50 / 0.00 / 0.	0.00 0.00 0.00 R REQ'E 6.50 6.50 0.00 0.00	× × × × × × ×	1.00 1.00 1.00 % LOAE 1.00 1.00	1	1.00 1.00 1.00 1.00 1.00 1.00	= = = = = = = = = = = = = = = = = = = =	0.00 lt 0.00 lt 4108.23 lt 2388.50 lt 1215.15 lt 0.00 lt 0.00 lt
IEAF	+ + + + + + + + + + + + + + + + + + +	F# F2 F2 -	3 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2	((((	191.08 plf		) x ) x ) x ) x ) x ) x ) x ) x ) x ) x	0.00 0.00 WIDTH 25.00 5.50 0.00 0.00	× × × × × ×	0.00 0.00 0.00 0.00 X BAR 1.00 27.75 0.00 0.00	!!!!	0.00 0.00 0.00 24.00 0.00 0.00 0.00	x x x x x x	0.00 / 0.00 / 0.00 / 6.50 / 6.50 / 0.00 / 0.	0.00 0.00 0.00 0.00 6.50 6.50 0.00 0.00	× × × × × × ×	1.00 1.00 1.00 % LOAE 1.00 1.00 1.00	DEFF	1.00 1.00 1.00 1.00 1.00 1.00 1.00	= = = = = = = = = = = = = = = = = = = =	0.00 lb



### SEISMIC FORCES TO SHEAR LINES:

1ST LEVEL

2ND FLOOR

NE	, LIIVE	FORCES F#			FORCE		WIDTH	_	X BAR				P DEGICAL	p peo	רוי	% LOA	DEF	EECT		
				,		100					2.00		R DESIGN	RREC					-	404.00
6	52	F3	V INTER	1	41.22 plf	) X	24.00	×	1.00	1	2.00	×		6.50		1.00		1.00	=	494.62
	+		LINE1	(		4722.86 lb ) x	1.00	×	1.00	/	1.00	×		/ 6.50		( 1.00		1.00	=	4722.86
	+		4	(		) x	0.00	×	0.00	1	0.00	×	0.00	/ 0.00		1,00		1.00	=	0.00
	+			(		) ×	0.00	×	0.00	1	0.00	×		0.00		1.00		1.00	=	0.00
	+	*		(		) x	0.00	×	0.00	1	0.00	×	0.00	/ 0.00	,	1.00	1	1.00	=	0.00
	+	-		(		) x	0.00	×	0.00	1	0.00	×	0.00	/ 0.00		1.00	1	1.00	=	0.00
	+	-			(	- )x			0.00	1	0.00	×	0.00	/ 0.00	1	1.00	1	1.00	=	0.00
					2	100							6.50							5217.49
EA	R LINE	FORCES			FORCE		WEET		VEIR				5 SEGION	5 556		0/ 1 0 4	D.E.E.	FOT		
		F#			FORCE	- V.12	WIDTH	100	XBAR		L	100/	R DESIGN	R REQ		% LOA			4.5	101.00
7	0	F3	Williams	(	41.22 plf	) ×	24.00	X	1.00	1	2.00	×		6,50				1.00	=	494.62
	+		LINE2	(		4722.86 lb ) x	1.00	×	1.00	1	1.00	×		6.50		1.00		1,00	=	4722.86
	+			(		) x	0.00	X	0.00	1	0.00	×		0.00				1.00	=	0.00
	+		*	(		) ×	0.00	×	0.00	1	0.00	×	0.771.77.77	0.00		1.00		1.00	=	0.00
	+			(		) x	0.00	×	0.00	1	0.00	×	0.00	0.00	)	1.00	1	1.00	=	0.00
	+	8		(		) ×	0.00	×	0.00	1	0.00	×	0.00	0.00	,	1.00	1	1.00	=	0.00
	+	2			(	- )×			0.00	1	0.00	×		0.00				1.00	=	0.00
					(3)	9,33					1018100	163	6.50							5217.49
ΕΔI	2 I INE	FORCES		_				_							_	_		_	_	
IE.	LINE	F#			FORCE		WIDTH		X BAR		L	_	R DESIGN	R REQ	D	% LOA	DEFE	ECT	_	
8		F4	- 2	1	39.06 plf	) x	25.00	×	1.00	1	2.00	×	6.50	6.50	,			1.00	=	488.31
	+		LINE4	1		4108.23 lb ) x	1.00	×	1.00	1	1.00	X		6.50				1,00	=	4108.23
	+			1		) x	0.00	×	0.00	,	0.00	×	0.00		,			1.00	-	0.00
	2	3	740	,		)×	0.00		0.00	1	0.00	×		0.00	,			1.00	=	0.00
	7			)		78.70		×		'.					-					
	7			,		)×	0.00	×	0.00	1	0.00	×		7 7 7 7	>			1.00	=	0.00
	+			(	10.	) ×	0.00	×	0.00	1	0.00	×	0.00	700	)			1.00	=	0.00
	+	5			(	- )×			0.00	1	0.00	×	0.00	0.00	>	1,00	1	1.00	=_	0.00
													6,50						_	4596,54
EAF	R LINE	FORCES																		
E		F#			FORCE	NW 275	WIDTH		X BAR		L	1.00	R DESIGN	R REQ	D	% LOA	D EFF	ECT	- 629 III	N/A003-30/F3
9		F4	-	(	39.06 plf	) x	25.00	×	1.00	- 1	2.00	×	6.50	6.50	>	1.00	1	1.00	=	488.31
	+	F5		(	8.93 plf	) x	5.50	×	27.75	1	25.00	×	6.50	6.50	>	1.00	1	1.00	m	54.51
	+		LINE5	(	W.	3603.66 lb ) x	1.00	×	1.00	1	1.00	×	6.50	6.50	,	1.00	1	1.00	=	3603.66
	+	_	ALL DESCRIPTION OF THE PERSON	i		)×	0.00	×	0.00	1	0.00	×	0.00	0.00	>		1	1.00	=	0.00
	+		-	ì		)×	0.00	×	0.00	1	0.00	X	0.00	0.00	>			1.00	=	0.00
	+		35	1		)×	0.00	×	0.00	,	0.00	×	0.00	0.00	,			1.00	=	0.00
	7			,	V		0.00	^	0.00	1	0.00		0.00	0.00				1.00	_	0.00
	7				3	- )×			0.00	9	0.00	X	6.50	0.00	>	1.00		1,00	_	4146.48
	LINE	FORCES										_	0.8395		_		_		_	
ΔΕ	CHAC	F#			FORCE		WIDTH		X BAR	_	L	_	R DESIGN	R REQ	D	% LOA	D EFF	ECT	_	
_			*	(		)×	0.00	×	0.00	1	0.00	×	0.00	0.00	×			1.00	=	0.00
EAF E			2	ì		) ×	0.00	×	0.00	1	0.00	×	0.00	0.00	×			1.00		0.00
_	+			1		)×	0.00	×	0.00	,	0.00	×	0.00	0.00	×			1.00	_	0.00
_	÷					2000	0.00		0.00	,	0.00		0.00					1.00	=	0.00
_	÷			1				×	0.00			×		0.00	×					0.00
_	+ + + + + + + + + + + + + + + + + + + +		- 2	5		) x		**	0.00									4 17.75	-	0.00
_	+ + + +			(		)×	0.00	×	0.00	1	0.00	X	0.00	0.00	×			1.00	=	
_	+ + + +	:	-	(	zi.	) × ) ×		×	0.00	1	0.00	×	0.00	0.00	×	1,00	1	1.00	=	0.00
_	+ + + +			(	Č	)×	0.00			1						1,00	1			



### **REDUNDANCY CHECK:**

IN LIEU OF CHECKING REDUNDANCY ON EACH SHEAR LINE, USE DEFAULT VALUE OF rho = 1.3

STRENGTH DESIGN SHEAR FORCE SUMMARY:

SINENGI	LINE	GN SHEAR FORCE SUI FORCE	rho		V SEISMIC (STRENGTH)	-	<b>φ</b>		V SEISMIC (ASD)
	1 -	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	dl 00.0
	-	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	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	×	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	×	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	Х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	Х	0.70	=	0.00 lb
	_	0.00 lb x 0.00 lb x	1.30 1.30	=	dl 00.0 dl 00.0	X	0.70 0.70	=	0.00 lb 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
	<del>                                     </del>	0.00 lb x	1.30	=	0.00 lb	X	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	dl 00.0
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	Х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	X	0.70	=	dl 00.0
1	-	0.00 lb x 0.00 lb x	1.30 1.30	=	0.00 lb 0.00 lb	X X	0.70 0.70	=	dl 00.0 dl 00.0
		0.00 lb x	1.30	=	0.00 lb	X	0.70	=	0.00 lb
	_	0.00 lb x	1.30	=	0.00 lb	×	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	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
***************************************		0.00 lb x	1.30	=	0.00 lb	X	0.70	=	0.00 lb
	1 2	4722.86 lb x 4722.86 lb x	1.30 1.30	=	6139.72 lb 6139.72 lb	X	0.70 0.70	=	4297.80 lb 4297.80 lb
	3	1719.72 lb x	1.30	=	2235.64 lb	X X	0.70	_	1564.95 lb
	4	4108.23 lb x	1.30	=	5340.70 lb	×	0.70	=	3738.49 lb
	5	3603.66 lb x	1.30	=	4684.75 lb	х	0.70	=	3279.33 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb	х	0.70	=	di 00.0
ROOF	-	0.00 lb x	1.30	=	0.00 lb	×	0.70	=	0.00 lb
R	-	0.00 lb x	1.30	=	0.00 lb	X	0.70	=	0.00 lb
	~	0.00 lb x	1.30	=	0.00 lb	Х	0.70	=	0.00 lb
	-	0.00 lb x	1.30	=	0.00 lb 0.00 lb	X	0.70	=	0.00 lb 0.00 lb
	_	0.00 lb x 0.00 lb x	1.30 1.30	=	0.00 lb	X X	0.70 0.70	=	0.00 lb
	_	x di 00.0	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	х	0.70	=	dl 00.0
	6	5217.49 lb x	1.30	=	6782.73 lb	Х	0.70	=	4747.91 lb
I	7	5217.49 lb x	1.30	=	6782.73 lb	х	0.70	=	4747.91 lb
1	8	4596.54 lb x	1.30	=	5975.50 lb	×	0.70	=	4182.85 lb
1	9	4146.48 lb x	1.30	=	5390.42 lb	X	0.70	=	3773.29 lb
a		0.00 0		=	0.00 lb	Х	0.70	=	0.00 lb 0.00 lb
	-	0.00 lb x	1.30	-		v			
eriodinamientopolas	-	0.00 lb x	1.30	=	0.00 lb	X	0.70	=	
erite Dissassierings and derivers		0.00 lb x 0.00 lb x	1.30 1.30	=	0.00 lb	×	0.70 0.70	=	0.00 lb
OCTAL TRANSPORTED AND AND AND AND AND AND AND AND AND AN		0.00 lb x 0.00 lb x 0.00 lb x	1.30 1.30 1.30		0.00 lb 0.00 lb	X X	0.70 0.70 0.70	=	dl 00.0 dl 00.0
OR		0.00 lb x 0.00 lb x	1.30 1.30	=	0.00 lb	×	0.70 0.70	=======================================	0.00 lb
LOOR		0.00 lb x 0.00 lb x 0.00 lb x 0.00 lb x	1.30 1.30 1.30 1.30	= = =	0.00 lb 0.00 lb 0.00 lb	X X X	0.70 0.70 0.70 0.70	= = =	0.00 lb 0.00 lb 0.00 lb
) FLOOR		0.00 lb x 0.00 lb x 0.00 lb x 0.00 lb x 0.00 lb x	1.30 1.30 1.30 1.30 1.30	= = = =	0.00 lb 0.00 lb 0.00 lb 0.00 lb 0.00 lb 0.00 lb	х х х	0.70 0.70 0.70 0.70 0.70	= = = = =	0.00 lb 0.00 lb 0.00 lb 0.00 lb
2ND FLOOR		0.00 lb x	1.30 1.30 1.30 1.30 1.30 1.30 1.30 1.30	= = = = = = = = = = = = = = = = = = = =	0.00 lb 0.00 lb 0.00 lb 0.00 lb 0.00 lb 0.00 lb 0.00 lb	x x x x	0.70 0.70 0.70 0.70 0.70 0.70 0.70 0.70	= = = = = = = = = = = = = = = = = = = =	0.00 lb 0.00 lb 0.00 lb 0.00 lb 0.00 lb 0.00 lb 0.00 lb
2ND FLOOR		0.00 lb x	1.30 1.30 1.30 1.30 1.30 1.30 1.30 1.30	= = = = =	0.00 lb 0.00 lb 0.00 lb 0.00 lb 0.00 lb 0.00 lb 0.00 lb	x x x x x	0.70 0.70 0.70 0.70 0.70 0.70 0.70 0.70	= = = = = = = = = = = = = = = = = = = =	0.00 lb
2ND FLOOR	-	0.00 lb x	1.30 1.30 1.30 1.30 1.30 1.30 1.30 1.30	= = = = = = = = = = = = = = = = = = = =	0.00 lb	x x x x x x x	0.70 0.70 0.70 0.70 0.70 0.70 0.70 0.70	= = = = = = = = = = = = = = = = = = = =	0.00 lb
2ND FLOOR		0.00 lb x	1.30 1.30 1.30 1.30 1.30 1.30 1.30 1.30		0.00 lb	x x x x x x x x	0.70 0.70 0.70 0.70 0.70 0.70 0.70 0.70		0.00 lb
2ND FLOOR	-	0.00 lb x	1.30 1.30 1.30 1.30 1.30 1.30 1.30 1.30		0.00 lb	x x x x x x x x	0.70 0.70 0.70 0.70 0.70 0.70 0.70 0.70		0.00 lb
2ND FLOOR	-	0.00 lb x	1.30 1.30 1.30 1.30 1.30 1.30 1.30 1.30		0.00 lb	x x x x x x x x x x x x	0.70 0.70 0.70 0.70 0.70 0.70 0.70 0.70		0.00 lb
2ND FLOOR	-	0.00 lb x	1.30 1.30 1.30 1.30 1.30 1.30 1.30 1.30		0.00 lb	x x x x x x x x x x x x x x x x x x x	0.70 0.70 0.70 0.70 0.70 0.70 0.70 0.70		0.00 lb
2ND FLOOR	-	0.00 lb x	1.30 1.30 1.30 1.30 1.30 1.30 1.30 1.30		0.00 lb	x x x x x x x x x x x x	0.70 0.70 0.70 0.70 0.70 0.70 0.70 0.70		0.00 lb



### WIND LOAD DESIGN:

016CBC 609.6.4		E 7-10 CTIONAL PROCEDURE - MWFRS ENCLOSED BUILDING					ASCE 7-10 27.4.1
5-1A	BASI	C WIND SPEEDS FOR RISK CATEGORY I	BUILDING	ss			
	1	BASIC WIND SPEED	V	=	115.00 MPI	1	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	1	=	1.00		ASCE 7-10 TABLE 6-1
LDING	DIRE	CTION - X					
	3	EXPOSURE CATEGORY		C CASE 1			ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3
		HEIGHT ABOVE GROUND LEVEL	z	= CASE 1	20.00 ft		MEAN ROOF HEIGHT
		TERRAIN EXPOSURE CONSTRANT		=	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
	1070					+ 1	
	5	VELOCITY PRESSURE	qz qz	= .00256 Kz Kz =	t Kd V^2 I 25.954 psf		ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT
			cja.		20.004 poi	-	THEODILE AT MEAN TOOL TELOTT
	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
				2			
	9	EXTERNAL PRESSURE COEFFICIENTS	L B		51.00 24.00		ASCE 7-10 FIG 27.4.1 ASCE 7-10 FIG 27.4.1
				=	2.125		ASCE 7-10 FIG 27.4.1
				=	0.392		ASCE 7-10 FIG 27.4.1
			Cp 1	-	0.800	WINDWARD WALL	ASCE 7-10 FIG 27.4.1
			7.0	=	-0.294	LEEWARD WALL	ASCE 7-10 FIG 27.4.1
		1000 LB 0 4 1 0 10 10 10 10 10 10 10 10 10 10 10 10	7.5	-	-0.700	SIDE WALLS	ASCE 7-10 FIG 27.4.1
		PARALLEL TO RIDGE	100000000000000000000000000000000000000	-	-0.900	ROOF 1 0.00 deg	ASCE 7-10 FIG 27.4.1 PITCH 0.00 ::
	10	DECICH WIND LOAD	OP C	= -001/0-	-0.900	ROOF 2 0,00 deg	PITCH 0.00 :
	10	DESIGN WIND LOAD WINDWARD q = qz & qi = qh	I.	= qGCp - qi (Gc =	17.649 -	-4.672 = 2	ASCE 7-10 TABLE 26.11-1 2.321 psf LATERAL FORCE
		LEEWARD q = gh	P		-6.480 -		1.152 psf P WIND 33.47 psf
		SIDE WALLS qi = qh	P	<u> </u>	-15.443 -	(C7)DO/ C (L 2) P	0.114 psf
		ROOF 1 qi = qh	, E	=			
					-19.855 +		4.527 psf WIND -24.53 psf
		ROOF 2 qi = qh	p	=	-19.855 +	-4.672 = -2	1.527 psf LEE -24.53 psf
			p p WALL	=	-19.855 + 22.32 -	-4.672 = -2 -11.15 =	4.527 psf LEE -24.53 psf UPLIFT FORCE
LDING	THE RESIDENCE PROPERTY.	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION	p	-	-19.855 +	-4.672 = -2	1.527 psf 33.47 psf 0.00 psf
ILDING	DIREC	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION	p p WALL	= = C	-19.855 + 22.32 -	-4.672 = -2 -11.15 =	4.527 psf 33.47 psf 0.00 psf ASCE 7-10 26.7.3
ILDING	THE RESIDENCE PROPERTY.	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY	p WALL p ROOF	-	-19.855 + 22.32 - 0.00 -	-4.672 = -2 -11.15 =	4.527 psf 33.47 psf 0.00 psf ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3
ILDING	THE RESIDENCE PROPERTY.	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL	p p WALL p ROOF	C CASE 1	-19.855 + 22.32 - 0.00 -	-4.672 = -2 -11.15 =	1.527 psf 33.47 psf 0.00 psf ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3 MEAN ROOF HEIGHT
LDING	THE RESIDENCE PROPERTY.	ROOF 2 qi = qh  DESIGN WIND LOAD x DIRECTION  CTION - Y  EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT	p p WALL p ROOF z α	C CASE 1	-19.855 + 22.32 - 0.00 -	-4.672 = -2 -11.15 =	1.527 psf 33.47 psf 0.00 psf ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1
LDING	THE RESIDENCE PROPERTY.	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL	p p WALL p ROOF z α zg	C CASE 1	-19.855 + 22.32 - 0.00 -	-4.672 = -2 -11.15 =	1.527 psf 33.47 psf 0.00 psf ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3 MEAN ROOF HEIGHT
LDING	3	ROOF 2 qi = qh  DESIGN WIND LOAD x DIRECTION  CTION - Y  EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE	p p WALL p ROOF z α zg Kz	C CASE 1	-19.855 + 22.32 - 0.00 - 20.00 ft 9.50 900.00 ft 0.90	-4.672 = -2 -11.15 =	1.527 psf 33.47 psf 0.00 psf ASCE 7-10 26.7.3 ASCE 7-10 TABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1
ILDING	3	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR	P p WALL p ROOF z α zg Kz	C CASE 1	-19.855 + 22.32 - 0.00 - 20.00 ft 9.50 900.00 ft 0.90	-4.672 = -2 -11.15 =	ASCE 7-10 26.8.2  LEE -24.53 psf UPLIFT FORCE  LEE -24.53 psf UPLIFT FORCE  LEE -24.53 psf UPLIFT FORCE  ASCE 7-10 26.7.3  ASCE 7-10 7ABLE 6-3  MEAN ROOF HEIGHT  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2
ILDING	3 4 5	ROOF 2 qi = qh  DESIGN WIND LOAD x DIRECTION  CTION - Y  EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE	P p WALL p ROOF z α zg Kz	C CASE 1	-19.855 + 22.32 - 0.00 - 20.00 ft 9.50 900.00 ft 0.90 1.000	-4.672 = -2 -11.15 =	1.527 psf 33.47 psf 0.00 psf ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 26.8.2 ASCE 7-10 (27.3-1)
ILDING	3 4 5	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE	p p WALL p ROOF z α zg Kz Kzt	C CASE 1	-19.855 + 22.32 - 0.00 - 20.00 ft 9.50 900.00 ft 0.90 1.000 t Kd V^2 I 25.954 psf	-4.672 = -2 -11.15 =	ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT
ILDING	3 4 5	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR	p p WALL p ROOF z α zg Kz Kzt	C CASE 1	-19.855 + 22.32 - 0.00 - 20.00 ft 9.50 900.00 ft 0.90 1.000	-4.672 = -2 -11.15 =	1.527 psf 33.47 psf 0.00 psf ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 26.8.2 ASCE 7-10 (27.3-1)
LDING	3 4 5	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE	p p WALL p ROOF z α zg Kz Kzt	C CASE 1	-19.855 + 22.32 - 0.00 - 20.00 ft 9.50 900.00 ft 0.90 1.000 t Kd V^2 I 25.954 psf	-4.672 = -2 -11.15 =	ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT
LDING	3 4 5	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE TOPOGRAPHY FACTOR VELOCITY PRESSURE GUST FACTOR	p p WALL p ROOF  z a a zg Kz Kzt a gz a gz G	C CASE 1	-19.855 + 22.32 - 0.00 - 20.00 ft 9.50 900.00 ft 0.90 1.000 t Kd V^2 I 25.954 psf	-4.672 = -2 -11.15 =	ASCE 7-10 26.8.2
ILDING	3 4 5 6 7 8	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION	p p WALL p ROOF  z a zg kz kz kzt a gz a	C CASE 1	-19.855 + 22.32 - 0.00 - 20.00 ft 9.50 900.00 ft 0.90 1.000 t Kd V^2 I 25.954 psf	-4.672 = -2 -11.15 =	ASCE 7-10 26.7.3 ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 26.8.2 ASCE 7-10 26.8.2 ASCE 7-10 26.9.1 ASCE 7-10 26.9.1 ASCE 7-10 26.9.1 ASCE 7-10 26.9.1 ASCE 7-10 7ABLE 26.11-1 ASCE 7-10 TABLE 26.11-1 ASCE 7-10 TABLE 26.11-1
ILDING	3 4 5 6 7 8	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE	p p WALL p ROOF  z a zg kz kzt qz qz : G GCpi : L B :	C CASE 1 ====================================	-19.855 + 22.32 - 0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000	-4.672 = -2 -11.15 =	ASCE 7-10 26.7.3 ASCE 7-10 26.7.3 ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 26.8.2 ASCE 7-10 (27.3-1) PRESSURE AT MEAN ROOF HEIGHT ASCE 7-10 26.9.1 ASCE 7-10 26.9.1 ASCE 7-10 26.9.1 ASCE 7-10 26.10 ASCE 7-10 1ABLE 26.11-1 ASCE 7-10 FIG 27.4.1 ASCE 7-10 FIG 27.4.1
LDING	3 4 5 6 7 8	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE	P P WALL PROOF  Z a significant significan	C CASE 1  = .00256 Kz Kzt  ENCLOSED	-19.855 + 22.32 - 0.00 - 20.00 ft 9.50 900.00 ft 0.90 1.000 -	-4.672 = -2 -11.15 =	ASCE 7-10 26.7.3  ASCE 7-10 26.7.3  ASCE 7-10 7ABLE 6-3  MEAN ROOF HEIGHT  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 26.8.2  ASCE 7-10 26.8.1  ASCE 7-10 26.9.1  ASCE 7-10 7ABLE 26.11-1  ASCE 7-10 7ABLE 27.4.1  ASCE 7-10 7ABLE 27.4.1  ASCE 7-10 7ABLE 27.4.1
LDING	3 4 5 6 7 8	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE	p p WALL p ROOF  z a g Kz Kzt  gz gz G GCpi = L B L/B h/L	C CASE 1  = .00256 Kz Kzt  ENCLOSED	-19.855 + 22.32 - 0.00	-4.672 = -2 -11.15 = 0.00 =	ASCE 7-10 26.7.3  ASCE 7-10 26.7.3  ASCE 7-10 26.7.3  ASCE 7-10 7ABLE 6-3  MEAN ROOF HEIGHT  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 26.8.1  ASCE 7-10 26.9.1  ASCE 7-10 7ABLE 26.11-1  ASCE 7-10 FIG 27.4.1
LDING	3 4 5 6 7 8	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE	P p WALL p ROOF  Z α 29 Kz Kzt 42 qz qz 15 G  GCpi 1 L 1 B 1 L/B 1 h/L Cp 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	C CASE 1  = .00256 Kz Kzt  = ENCLOSED	-19.855 + 22.32 - 0.00 - 20.00 ft 9.50 900.00 ft 0.90 1.000 t Kd V^2 I 25.954 psf 0.85 0.18 +/- 24.00 51.00 0.471 0.833 0.800	-4.672 = -2 -11.15 = 0.00 =	ASCE 7-10 26.7.3 ASCE 7-10 26.7.3 ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 26.8.2  ASCE 7-10 26.9.1  ASCE 7-10 26.9.1  ASCE 7-10 7ABLE 26.11-1  ASCE 7-10 FIG 27.4.1
LDING	3 4 5 6 7 8	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE	P p WALL p ROOF  z a zg Kz  Kzt  G GCpi = L  B L/B h/L = Cp 1  Cp 2	C CASE 1	-19.855 + 22.32 - 0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000 -t Kd V^2 I 25.954 psf  0.85  0.18 +/- 24.00 51.00 0.471 0.833 0.800 -0.500	-4.672 = -2 -11.15 = 0.00 =	ASCE 7-10 26.7.3 ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 TABLE 27.3-1 ASCE 7-10 26.8.2 ASCE 7-10 26.8.2 ASCE 7-10 26.9.1 ASCE 7-10 26.9.1 ASCE 7-10 7ABLE 26.9-1 ASCE 7-10 7ABLE 26.9-1 ASCE 7-10 7ABLE 27.3-1 ASCE 7-10 7ABLE 27.3-1 ASCE 7-10 7ABLE 26.9-1 ASCE 7-10 7ABLE 26.11-1 ASCE 7-10 7ABLE 26.11-1 ASCE 7-10 7ABLE 26.11-1 ASCE 7-10 7ABLE 27.4.1
LDING	3 4 5 7 8	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS	P p WALL p ROOF  z a	C CASE 1  = .00256 Kz Kzt  ENCLOSED	-19.855 + 22.32 - 0.00 - 20.00 ft 9.50 900.00 ft 0.90 1.000 - 1.000 - 1.000 - 1.000 51.00 0.471 0.833 0.800 -0.500 -0.700	-4.672 = -211.15 = 0.00 =  WINDWARD WALL LEEWARD WALL SIDE WALLS	ASCE 7-10 26.7.3  ASCE 7-10 26.7.3  ASCE 7-10 7ABLE 6-3  MEAN ROOF HEIGHT  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 26.8.2  ASCE 7-10 26.8.1  ASCE 7-10 26.9.1  ASCE 7-10 7ABLE 26.11-1  ASCE 7-10 7ABLE 26.11-1  ASCE 7-10 7ABLE 26.11-1  ASCE 7-10 FIG 27.4.1
LDING	3 4 5 7 8	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE	P p WALL p ROOF  z α zg κz κzt σz	C CASE 1	-19.855 + 22.32 - 0.00	-4.672 = -2 -11.15 = 0.00 =  WINDWARD WALL LEEWARD WALL SIDE WALLS ROOF 1 0.00 deg	1.527 psf 33.47 psf 0.00 psf  ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 26.8.2  ASCE 7-10 26.9.1  ASCE 7-10 7ABLE 26.11-1  ASCE 7-10 FIG 27.4.1
LDING	3 4 5 6 7 8 9	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS	P p WALL p ROOF  z a zg kz kzt qz qz : G GCpi : L : B L/B : h/L : Cp 1 Cp 2 : Cp 3 : Cp 4 : Cp 4 : Cp 4 :	C CASE 1	-19.855 + 22.32 - 0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000 -1.000	-4.672 = -211.15 = 0.00 =  WINDWARD WALL LEEWARD WALL SIDE WALLS	ASCE 7-10 26.7.3  ASCE 7-10 26.7.3  ASCE 7-10 26.7.3  ASCE 7-10 7ABLE 6-3  MEAN ROOF HEIGHT  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 26.8.2  ASCE 7-10 26.9.1  ASCE 7-10 26.10  ASCE 7-10 FIG 27.4.1
LDING	3 4 5 6 7 8 9	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS  NORMAL TO RIDGE ( > 10 deg )	P p WALL p ROOF  z a zg kz kzt qz qz : G GCpi : L : B L/B : h/L : Cp 1 Cp 2 : Cp 3 : Cp 4 : Cp 4 : Cp 4 :	C CASE 1  = .00256 Kz Kzt  ENCLOSED	-19.855 + 22.32 - 0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000 -1.000	WINDWARD WALL LEEWARD WALL SIDE WALLS ROOF 1 0.00 deg ROOF 2 0.00 deg	1.527 psf 33.47 psf 0.00 psf  ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 26.8.2  ASCE 7-10 26.9.1  ASCE 7-10 7ABLE 26.11-1  ASCE 7-10 FIG 27.4.1
LDING	3 4 5 6 7 8 9	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR  VELOCITY PRESSURE  GUST FACTOR  ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS  NORMAL TO RIDGE ( > 10 deg )  DESIGN WIND LOAD WINDWARD q = qx & qi = qh	P p WALL p ROOF  z a a zg Kz  Kzt  G G  GCpi  L a B L/B h/L a Cp 1 Cp 2 Cp 3 Cp 4	C CASE 1  = .00256 Kz Kzt  ENCLOSED  qGCp - ql (Gc)	-19.855 + 22.32 - 0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000  t Kd V^2 I 25.954 psf  0.85  0.18 +/-  24.00 51.00 0.471 0.833 0.800 -0.500 -0.700 -0.900 pi)	WINDWARD WALL LEEWARD WALL SIDE WALLS ROOF 1 0.00 deg ROOF 2 0.00 deg -4.672 = 22	ASCE 7-10 26.7.3  ASCE 7-10 26.7.3  ASCE 7-10 7ABLE 6-3  MEAN ROOF HEIGHT  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 (27.3-1)  PRESSURE AT MEAN ROOF HEIGHT  ASCE 7-10 26.9.1  ASCE 7-10 7BLE 26.11-1  ASCE 7-10 FIG 27.4.1
LDING	3 4 5 6 7 8 9	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR  VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS  NORMAL TO RIDGE ( > 10 deg )  DESIGN WIND LOAD WINDWARD q = qx & qi = qh LEEWARD q = qh	P p WALL p ROOF  z α zg κz κzt α gz qz : G G GCpi : L B L/B h/L Cp 1 Cp 2 Cp 3 Cp 4	C CASE 1  = .00256 Kz Kzt  = ENCLOSED	-19.855 + 22.32 - 0.00	WINDWARD WALL LEEWARD WALL SIDE WALLS ROOF 1 0.00 deg ROOF 2 0.00 deg -4.672 = 22 4.672 = 21	1.527 psf 33.47 psf 0.00 psf  ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 26.8.2  ASCE 7-10 26.9.1  ASCE 7-10 7ABLE 26.11-1  ASCE 7-10 FIG 27.4.1
LDING	3 4 5 6 7 8 9	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR  VELOCITY PRESSURE  GUST FACTOR  ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS  NORMAL TO RIDGE ( > 10 deg )  DESIGN WIND LOAD WINDWARD q = qx & qi = qh LEEWARD q = qh SIDE WALLS qi = qh	P p WALL p ROOF  z α zg κz κzt α gz qz α gz α gz α gz α gz α gz α gz α	C CASE 1  = .00256 Kz Kzt  = NCLOSED  = qGCp - qi (Gcq	-19.855 + 22.32 - 0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000  t Kd V^2 I 25.954 psf  0.85  0.18 +/- 24.00 51.00 0.471 0.833 0.800 -0.500 -0.700 -0.900 -0.900 pi) 17.64911.03015.443 -	WINDWARD WALL LEEWARD WALL SROOF 1 0.00 deg ROOF 2 0.00 deg 4.672 = 4.672 = 4.672 = 4.672 = -20	1.527 psf 33.47 psf 0.00 psf  ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 26.8.2  ASCE 7-10 26.9.1  ASCE 7-10 26.9.1  ASCE 7-10 FIG 27.4.1 ASCE 7-10 TABLE 26.11-1  LATERAL FORCE P WIND 38.02 psf 1.114 psf
LDING	3 4 5 6 7 8 9	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR VELOCITY PRESSURE  GUST FACTOR ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS  NORMAL TO RIDGE ( > 10 deg )  DESIGN WIND LOAD WINDWARD q = qx & qi = qh LEEWARD q = qh SIDE WALLS qi = qh ROOF 1 qi = qh	P p WALL p ROOF  z a zg kz kz kzt qz qz c c c c c c c c c c c c c c c c c	C CASE 1  = .00256 Kz Kzt  = ENCLOSED  = qGCp - qi (Gcq	-19.855 + 22.32 - 0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000 - t Kd V^2 I 25.954 psf  0.85  0.18 + / -  24.00 51.00 0.471 0.833 0.800 -0.500 -0.700 -0.900 pi) 17.64911.03015.44319.855 +	WINDWARD WALL LEEWARD WALL SIDE WALLS ROOF 1 0.00 deg ROOF 2 0.00 deg 4.672 = 4.672 = -24 4.672 = -24 4.672 = -24	ASCE 7-10 26.7.3  ASCE 7-10 26.7.3  ASCE 7-10 7ABLE 6-3  MEAN ROOF HEIGHT  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 26.9-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 26.8.2  ASCE 7-10 26.9.1  ASCE 7-10 26.9.1  ASCE 7-10 1ABLE 26.9-1  ASCE 7-10 1ABLE 26.11-1  ASCE 7-10 16.10  ASCE 7-10 16.27.4.1  ASCE 7-10 16.27.4
LDING	3 4 5 6 7 8 9	ROOF 2 qi = qh DESIGN WIND LOAD x DIRECTION  CTION - Y EXPOSURE CATEGORY  HEIGHT ABOVE GROUND LEVEL TERRAIN EXPOSURE CONSTRANT TERRAIN EXPOSURE CONSTRANT VELOCITY PRESSURE  TOPOGRAPHY FACTOR  VELOCITY PRESSURE  GUST FACTOR  ENCLOSURE CLASSIFICATION INTERNAL PRESSURE  EXTERNAL PRESSURE COEFFICIENTS  NORMAL TO RIDGE ( > 10 deg )  DESIGN WIND LOAD WINDWARD q = qx & qi = qh LEEWARD q = qh SIDE WALLS qi = qh	P p WALL p ROOF  z α zg κz κzt α gz qz α gz α gz α gz α gz α gz α gz α	C CASE 1  = .00256 Kz Kzt  = ENCLOSED	-19.855 + 22.32 - 0.00 -  20.00 ft 9.50 900.00 ft 0.90  1.000  t Kd V^2 I 25.954 psf  0.85  0.18 +/- 24.00 51.00 0.471 0.833 0.800 -0.500 -0.700 -0.900 -0.900 pi) 17.64911.03015.443 -	WINDWARD WALL LEEWARD WALL SIDE WALLS ROOF 1 0.00 deg ROOF 2 0.00 deg 4.672 = 22 4.672 = -15 4.672 = -24 4.672 = -24 4.672 = -24 4.672 = -24 4.672 = -24	1.527 psf 33.47 psf 0.00 psf  ASCE 7-10 26.7.3 ASCE 7-10 7ABLE 6-3 MEAN ROOF HEIGHT ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 26.9-1 ASCE 7-10 TABLE 27.3-1  ASCE 7-10 TABLE 27.3-1  ASCE 7-10 26.8.2  ASCE 7-10 26.8.2  ASCE 7-10 26.9.1  ASCE 7-10 26.9.1  ASCE 7-10 FIG 27.4.1 ASCE 7-10 TABLE 26.11-1  LATERAL FORCE P WIND 38.02 psf 1.114 psf



# HOFFMAYER INC. STRUCTURAL ENGINEERING

WIND FORCES:

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umaw	_	×	SĮ	5.00	) +		×		00.00	- ¥	-24.53 psf x	SIN	0.00 ×	00.0	# ) =	167.36	plf
	2	27	Sť.	5.00	+	-24.53 psf	x SIN		00:00	#	-24.53 psf x	SIN	0.00	00'0	# ) =	190.11	plf
-ei/sow	•		0.00 psf x	0.00	+		×		00'0	<b>#</b>	0.00 psf x	SIN	0.00 ×	00'0	#	00.00	plf
1	,	ı	, st	0.00	+ -		×		0.00	# :	0.00 psf x	SIN	0.00 ×	00'0	# (#	0.00	늄
100		•	0.00 psr x	00.0	+ -		×		00.0	⊭ :	0.00 pst x	Z :	× 00.00	0.00	    = :	0.00	<u>+</u>
0.01	r		3	00.0	+ -		×		000	, # :	0.00 psf x	SIS	× 0.00	0.0	 	00.0	÷:
, =		,	0.00 psr x	0.00	+ -		N X		00:00	, # :	0.00 psf x	N S	× 0.00	0.00	# ( ;	00.0	÷:
ЭC		1	<u>.</u>	8 6	 		× :		00.0	, Ľ 6	0.00 pst x	Z Z	× 00.0	0.00	 	00.00	±d.
OA	. 1		0.00 psf ×	3 8	+ +	0.00	< > 		3 8	: : ≓	0.00 pst x	2 2	x x	0000	 	00.0	= =
		,	0.00 psf x	00.0	+		× ×		200	: ,	0.00 psf x	Z Z	800	00.0	II	80.0	= ==
******	,	1	0.00 psf x	00'0	+		×		00:0	#	0.00 x fsd 0.00	SIN	0.00 ×	00'0	# (#	00.0	i ia
	ı	i	0.00 psf x	00.00	<u> </u>	0.00 psf	×	0.00 0.00	00.00	, # 0		SIS	0.00 x	0.00	# # #	0.00	plf
		\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\	0.00 psi x	0.00	, L		NIO X	ı	00.0	~ 11	pst	N O	× 00.0	0.00	1	0.00	plf
	) 4	<b>₹</b> ≿	38.02 psf ×	10.00	- \ + +	-24.53 psi -24.53 psi	××		0000		-24.53 pst x -24.53 psf x	2 2	× ×	900	  -  -	384.73	iid iid
	75	4	38.02 psf x	10.00	+		×		00'0	#	-24.53 psf x	SIN	0.00 ×	0.00	= ( #	380.23	. <del>.</del>
110	ı	1	0.00 psf x	0.00	+		×		00.00	#	0.00 psf x	SIN	0.00 ×	7.00	# ( <del> </del>	00'0	i di
0.0		t	pst	0.00	+	_	×		00.00	+	0.00 psf x	SIN	0.00 ×	7.00	# )=	0.00	*
L =			0.00 psf x	0.00	+	0.00 ps	×		00:00	, #		Sin	0.00 ×	00.00	# ( #	00.00	≒
ЯC	ı	ı	bst	0.00	+	0.00 pst	×		000	⊭ :		S	0.00 ×	00.0	# ( #	0.00	늦
00	,		0.00 pst x	0.00	+	0.00 pst	×		00.0	⊭ :		N S	0.00 ×	0.00	# ·	00.0	÷:
1 <b>3</b> (	, ,	1 1	0.00 pst x	9 8	+ +	0.00 pst	× × × × ×	2 2	00:0	, = ¢		N S	0.00 x x	9 9	H +	00.0	± :
JNZ			, J	8 8	- \ + +		× >		886	≓ ¢	0.00 psr x	2 2	x :	90.0	 	0.00	= =
Z	1	r	0.00 psf x	000	· +	0.00	× ×	2 2	0000	, , = #		Z Z	× ×	900	_ d=	00.0	<u> </u>
pecessor		,	Jsc	0.00	+	0.00 ps	ı × SI	00.0 N	00.0	; #		S	0.00 ×	00.0	# ====================================		<u> </u>
Name and Address of the Owner, where the Owner, which is the Owner, where the Owner, which is the Owner, where the Owner, which is the Owner, whic	- ATTENDED	-	0.00 psf ×	00.0	+	0.00 ps	st × Si	N 0.00	00'0	ft -	0.00 psf x	SIN	0.00 ×	0.00	# (#	0.00	plf



### WIND FORCES TO SHEAR LINES:

2ND LEVEL

ROOF

SHEAR LINE	FORCES	XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX				COLUMN TO THE OWNER OF THE OWNER	CONTRACTOR THE CO.		tu	and the second second second	and the Vision		·	anno les con		COLUMN TO THE PARTY OF THE PART		
LINE	F#		***********	FORCE		WIDTH	***************************************	X BAR	*********	L		FACTORS			% LOAD EF	FECT		And the second s
1	F1	_	(	167.36 plf	) x	24.00	х	1.00	1	2.00	x	1.00 /	1.00	х	1.00 /	1.00	=	2008.36 lb
+	-	_	ì		) x	0.00	×	0.00	,	0.00	×	1.00 /	1.00	х	1.00 /	1.00	=	0.00 lb
+	_	_	ì		) x	0.00	x	0.00	1	0.00	X	1.00 /	1.00	х	1.00 /	1.00	=	0.00 lb
+	-	-	ì		) x	0.00	X	0.00	i	0.00	X	1.00 /	1.00	X	1.00 /	1.00	=	0.00 lb
+	_	-	ì		) x	0.00	x	0.00	1	0.00	x	1.00 /	1.00	x	1.00 /	1.00	=	0.00 lb
+	-	-	ì		)×	0.00	х	0.00	1	0.00	×	1.00 /	1.00	х	1.00 /	1.00	=	0.00 lb
+	_		`	(	) x			0.00	1	0.00	×	1.00 /	1.00	x	1.00 /	1.00	=	0.00 lb
				,	•													2008.36 lb
ALLEN MONTH OF THE PARTY OF THE						description (description of the control of the cont	obsolute management from		******					HISTORY COSTS	27-107 (V. 1-5-5-5-1			
SHEAR LINE			TTD Complete											NANAO II W			-	
LINE	F#		,	FORCE	3	WIDTH		X BAR	,	L		FACTORS	4.00		% LOAD EF			0000 00 11-
2	F1	-	(	167.36 plf	) x	24.00	X	1.00	1	2.00	Х	1.00 /	1.00	X	1.00	1.00 1.00	=	2008,36 lb 0.00 lb
+	-	-	(		) x	0.00	X	0.00	',	0.00	X	1.00 / 1.00 /	1.00	X	1.00 / 1.00 /	1.00	=	di 00.0
+	-	-	,		) X	0.00	X	0.00	1,	0.00	X		1.00	Х	1.00 /	1.00	=	
+	-	-	(		) x	0.00	X	0.00 0.00	1	0.00	X	1.00 / 1.00 /	1.00	X	1.00 /	1.00	=	0.00 lb 0.00 lb
+	-	-	(		) x ) x	0.00	X X	0.00	',	0.00	X	1.00 /	1.00 1.00	X	1.00 /	1.00	=	0.00 lb
+	-	-	(	(	) X	0.00		0.00	1	0.00	x x	1.00 /	1.00	X	1.00 /	1.00	=	0.00 lb
т	-			(	) X			0.00	′	0.00		1.00 1	1.00	^	1.00 /	1.00		2008.36 lb
																	=	2000.00 10
SHEAR LINE	FORCES	09001041000410101041144		**************************************					*****	***************************************	-	- Mary Construction of the						
LINE	F#			FORCE	***************************************	WIDTH		X BAR		Ĺ		FACTORS			% LOAD EF			
3	F2	-	(	190.11 plf	) x	18.00	X	1.00	/	2.00	х	1.00 I	1.00	Х	1.00 /	1.00	=	1711.02 lb
+	-	-	(		) x	0.00	х	0.00	1	0.00	X	1.00 /	1.00	×	1.00 /	1.00	=	0.00 lb
+	-	-	(		) x	0.00	Х	0.00	1	0.00	X	1.00 <i>I</i>	1.00	х	1.00 /	1.00	=	0.00 lb
+	-	-	(		) x	0.00	х	0.00	1	0.00	Х	1.00 /	1.00	Х	1.00 /	1,00	=	0.00 lb
+	-	-	(		) x	0.00	Х	0.00	/	0.00	X	1.00 /	1.00	х	1.00 /	1.00	=	0.00 lb
+	-	-	(		) x	0.00	х	0.00	/	0.00	Х	1.00 <i>l</i>	1.00	Х	1.00 /	1.00	=	0.00 lb
+	-			(	) x			0.00	/	0.00	Х	1.00 /	1.00	х	1.00 /	1.00	=_	0.00 lb
																	-	1711.02 lb
SHEAR LINE	FORCES		***************************************			Martin and the Commercian	Commence of the		/				/ ************************************	***********				
LINE	F#			FORCE		WIDTH	***************************************	X BAR		L		FACTORS	*********		% LOAD EF	FECT	***************************************	***************************************
4	F2	-	(	190,11 plf	) x	18.00	x	1.00	1	2.00	x	1.00 /	1.00	х	1.00 /	1.00	=	1711.02 lb
+	F2	-	(	190.11 plf	) x	25.00	х	1.00	1	2.00	x	1.00 /	1.00	X	1.00 /	1.00	=	2376.42 lb
+	-	-	(		) x	0.00	х	0.00	1	0.00	x	1.00 /	1.00	х	1.00 /	1.00	=	dl 00.0
+	-	-	(		) x	0.00	x	0.00	1	0.00	×	1.00 /	1.00	х	1.00 /	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	1	0.00	x	1.00 /	1.00	х	1.00 /	1.00	=	0.00 lb
+	-	-	(		) x	0.00	x	0.00	1	0.00	×	1.00 /	1.00	X	1.00 /	1.00	=	0.00 lb
+	-			(	) x			0.00	1	0.00	x	1.00 <i>I</i>	1.00	х	1.00 /	1.00	=	0.00 lb
																	_	4087.45 lb
SHEAR LINE	FORCES							Market Control of the	#1000000000	ALCHENO SECTION SECTIO				MANAGEMENT .		20-1-1-1-1-1-1-1-1-1-1-1-1-1-1-1-1-1-1-1		
LINE	F#	**************************************		FORCE		WIDTH	A-774 - 14-14 - 14 (1-14)	X BAR	· · · · · · · · · · · · · · · · · · ·	Ł		FACTORS			% LOAD EF	ECT	**********	
5	F2	-	(	190.11 plf	) x	25.00	x	1.00	1	2.00	×	1.00 /	1.00	х	1.00 /	1.00	=	2376.42 lb
+	F2	-	ì	190.11 plf	) x	5.50	x	27.75	,	24.00	×	1.00 /	1.00	x	1.00 /	1.00	=	1209.00 lb
+	~	-	ì	e	) x	0.00	x	0.00	1	0.00	X	1.00 /	1.00	x	1.00 /	1.00	=	0.00 lb
+	-	-	ì		) x	0.00	x	0.00	ï	0.00	x	1.00 /	1.00	x	1.00 /	1.00	=	0.00 lb
+	-	-	ì		) x	0.00	x	0.00	1	0.00	×	1.00 /	1.00	x	1.00 /	1.00	=	0.00 lb
+	-	-	ì		) x	0.00	x	0.00	1	0.00	x	1,00 /	1.00	×	1.00 /	1.00	=	0.00 lb
+	-		•	(	) x			0.00	1	0.00	×	1.00 /	1.00	х	1.00 /	1.00	=	0.00 lb
				•	•												_	3585.43 lb
																	-	



# WIND FORCES TO SHEAR LINES: 1ST LEVEL 2ND FLOOR

٧E	F#		00000000000000000000000000000000000000	FORCE		WIDTH		X BAR	L	-	FACTORS	****		% LOAD EFF	ECT	-	
6	F3	_	1	334.73 plf	) x	24.00	х	1.00 /	2.00	×	1.00 /	1.00	×	1.00 /		=	4016.72 lk
` .	+ -	F1	ì	00 1.7 0 pii	2008.36 lb ) x	1.00	×	1.00 /	1.00	×	1.00 /	1.00	x	1.00 /	1.00	=	2008.36 lb
_	+ -		ì		) x	0.00	x	0.00 /	0.00	×	1.00 /	1.00	x	1.00 /	1.00	=	0.00 1
	1.		,		•	0.00		0.00 /			1.00 /			1.00 /	1.00	=	
		-	(		) x		X		0.00	X		1.00	×				0.00 1
4	+ -	-	(		) x	0.00	Х	0.00	0.00	х	1.00 /	1.00	х	1.00 /	1.00	=	0.00 1
+	+ -	-	(		) x	0.00	x	0.00 /	0.00	Х	1.00 /	1.00	Х	1.00 /	1.00	=	0.00
4	+ -			(	) x			0.00 /	0.00	х	1.00 /	1.00	Х	1.00 /	1.00	=	0.00 1
																	6025.08
EADI	INE FORCES		· votabilities in a stronger		interiore in the second se	0	over (Classic)	- Continuentia de la compansión de la comp	*********		*******************	-		and the father than the same and the same an		**********	****
E	F#			FORCE	THE RESERVE THE PROPERTY OF TH	WIDTH	**********	X BAR	L	ar ha-sii - Ainte-ar	FACTORS	Accesses to the second		% LOAD EFF	ECT	eregetes ander	Mosesters and Commission
7	F3	_	(	334.73 plf	) x	24.00	х	1.00 /	2.00	х	1.00 /	1.00	x	1.00 /		=	4016.72
+		F2	ì		2008.36 lb ) x	1.00	X	1.00 /	1.00	x	1.00 /	1.00	x	1.00 /	1.00	=	2008.36 [
4	+ -		ì		) x	0.00	x	0.00 /	0.00	×	1.00 /	1.00	x	1.00 /	1.00	=	0.00 1
		_	,		) x	0.00	x	0.00 /	0.00		1.00 /	1.00	×	1.00 /	1.00	=	0.00 1
1	 L	-	(		,	0.00		0.00 /	0.00	Х	1.00 /			1.00 /	1.00	=	0.00 1
	- ·	-	(		) x		X			×		1.00	X				
+	<del>-</del> -	-	ţ	,	) x	0.00	Х	0.00 /	0.00	х	1.00 /	1,00	X	1.00 /	1.00	=	0.00 1
+	-			(	) ×			0.00 /	0.00	х	1.00 /	1.00	Х	1.00 /	1.00	=	0.00 1
																	6025.08
EAR L	INE FORCES					***************************************	N311347-A-17				****************			Market and a residence with a		<del></del>	
E	F#	CAMBER DOCKS AND MINISTER		FORCE		WIDTH	*****	X BAR	L		FACTORS	**************************************		% LOAD EFF		************	
8	F4	-	(	380.23 plf	) x	25.00	X	1.00 /	2.00	Х	1.00 /	1.00	Х	1.00 /		=	4752.84 II
+		F4	(		4087.45 lb) x	1.00	X	1.00 /	1.00	X	1.00 /	1.00	X	1.00 /	1.00	Ξ.	4087.45 II
+	+ <b>-</b>	-	(		) x	0.00	х	0.00 /	0.00	X	1.00 /	1.00	х	1.00 /	1.00	=	0.00
+	+ -	-	(		) x	0.00	Х	0.00 /	0.00	X	1.00 /	1.00	х	1.00 /	1.00	Ξ	0.00
+		-	(		) x	0.00	x	0.00 /	0.00	×	1.00 /	1.00	х	1.00 /	1.00	=	0.00 #
+		-	<i>(</i> '		, ) x	0.00	x	0.00 /	0.00	x	1.00 /	1.00	х	1.00 /	1.00	=	0.00 #
+	-		,	(	ĺχ			0.00 /	0.00	X	1.00 /	1.00	х	1.00 /	1.00	=	0.00
					,												8840.29 II
																	0040.29 1
ΔRII	INE FORCES		i katentari da katentari			***************************************						Marian de Vistania		opproximate with the little of		**************************************	0040.29 1
E <b>AR LI</b>	INE FORCES			FORCE		WIDTH		X BAR	1.		FACTORS			% LOAD EFF	ECT		0040.29 11
_			(	FORCE 380.23 plf	) X	WIDTH 25.00	X	X BAR 1.00 /	L 2.00	X	FACTORS	1.00	X	% LOAD EFF 1.00 /			
_	F# F4	- -	(	380.23 plf				1.00 /			1.00 /	1.00 1.00	х		1.00	=	4752.84
Ē 9	F# F4 - F5	- - F5	(		) x	25.00 5.50	x	1.00 / 27.75 /	2.00 25.00	Х	1.00 / 1.00 /	1.00	x x	1.00 / 1.00 /	1.00 1.00		4752.84 II 2321.29 II
Ē 9 +	F# F4 - F5	- - F5	(	380.23 plf	) x 3585.43 lb ) x	25.00 5.50 1.00	x x	1.00 / 27.75 / 1.00 /	2.00 25.00 1.00	x x	1.00 / 1.00 / 1.00 /	1.00 1.00	x x x	1.00 / 1.00 / 1.00 /	1.00 1.00 1.00	=	4752.84    2321.29    3585.43
Ē 9 +	F# F4 - F5		( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	380.23 plf	) x 3585.43 lb ) x ) x	25.00 5.50 1.00 0.00	x x x	1.00 / 27.75 / 1.00 / 0.00 /	2.00 25.00 1.00 0.00	x x x	1.00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00	x x x	1.00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00	= = =	4752.84 II 2321.29 II 3585.43 II 0.00 II
Ē 9 +	F# F4 - F5	-	( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	380.23 plf	3585.43 lb ) x ) x ) x ) x	25.00 5.50 1.00 0.00 0.00	x x x x	1.00 / 27.75 / 1.00 / 0.00 / 0.00 /	2.00 25.00 1.00 0.00 0.00	x x x	1.00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00	x x x x	1.00 / 1,00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00	= = =	4752.84 II 2321.29 II 3585.43 II 0.00 II 0.00 II
= 9 + + + +	F# F4 - F5		(	380.23 plf	) x 3585.43 ib ) x ) x ) x ) x	25.00 5.50 1.00 0.00	x x x	1.00 / 27.75 / 1.00 / 0.00 / 0.00 / 0.00 /	2.00 25.00 1.00 0.00 0.00 0.00	x x x x	1.00 / 1.00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00	x x x x x	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00	= = = = = = = = = = = = = = = = = = = =	4752.84 II 2321.29 II 3585.43 II 0.00 II 0.00 II
Ē 9 +	F# F4 - F5	-	( ( ( (	380.23 plf	3585.43 lb ) x ) x ) x ) x	25.00 5.50 1.00 0.00 0.00	x x x x	1.00 / 27.75 / 1.00 / 0.00 / 0.00 /	2.00 25.00 1.00 0.00 0.00	x x x	1.00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00	x x x x	1.00 / 1,00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00	= = =	4752.84    2321.29    3585.43    0.00    0.00    0.00
= 9 + + + +	F# F4 - F5	-	( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	380.23 plf	) x 3585.43 ib ) x ) x ) x ) x	25.00 5.50 1.00 0.00 0.00	x x x x	1.00 / 27.75 / 1.00 / 0.00 / 0.00 / 0.00 /	2.00 25.00 1.00 0.00 0.00 0.00	x x x x	1.00 / 1.00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00	x x x x x	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00	= = = = = = = = = = = = = = = = = = = =	4752.84 lt 2321.29 lt 3585.43 lt 0.00 lt 0.00 lt 0.00 lt 10659.56 lt
= 9 + + + + + + +	F# F4 - F5   	-	( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	380.23 plf 380.23 plf	) x 3585.43 ib ) x ) x ) x ) x	25.00 5.50 1.00 0.00 0.00 0.00	x x x x	1.00 / 27.75 / 1.00 / 0.00 / 0.00 / 0.00 / 0.00 /	2.00 25.00 1.00 0.00 0.00 0.00 0.00	x x x x	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00	x x x x x	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00 1.00	= = = = = = = = = = = = = = = = = = = =	4752.84    2321.29    3585.43    0.00    0.00    0.00
E 9 + + + + + + +	F# F4 - F5   	-	( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	380.23 plf	3585.43 lb ) x 3585.43 lb ) x ) x ) x ) x	25.00 5.50 1.00 0.00 0.00 0.00 0.00	X X X X	1.00 / 27.75 / 1.00 / 0.00 / 0.00 / 0.00 / 0.00 /	2.00 25.00 1.00 0.00 0.00 0.00 0.00	x x x x x	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00	x x x x x	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00 1.00	= = = = = = = = = = = = = = = = = = = =	4752.84 II 2321.29 II 3585.43 II 0.00 II 0.00 II 0.00 II 10659.56 II
E 9 + + + +	F# F4 - F5   	-	( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( ( (	380.23 plf 380.23 plf	) x 3585.43 lb ) x ) x ) x ) x	25.00 5.50 1.00 0.00 0.00 0.00 0.00	x x x x x	1.00 / 27.75 / 1.00 / 0.00 / 0.00 / 0.00 / 0.00 /	2.00 25.00 1.00 0.00 0.00 0.00 0.00	x x x x x	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / FACTORS 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00	× × × × × × ×	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00 1.00	= = = = = = = = = = = = = = = = = = = =	4752.84 II 2321.29 II 3585.43 II 0.00 II 0.00 II 0.00 II 10659.56 II
E 9 + + + + + + +	F# F4 - F5   	-	(	380.23 plf 380.23 plf	) x 3585.43 lb ) x ) x ) x ) x ) x ) x	25.00 5.50 1.00 0.00 0.00 0.00 0.00	x x x x	1.00 / 27.75 / 1.00 / 0.00 / 0.00 / 0.00 / X BAR 0.00 / 0.00 /	2.00 25.00 1.00 0.00 0.00 0.00 0.00	x x x x x	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / FACTORS 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00	x x x x x x	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00	The state of the s	4752.84 II 2321.29 II 3585.43 II 0.00 II 0.00 II 0.00 II 10659.56 II
E 9 + + + + + + +	F# F4 - F5   	-	( ( ( (	380.23 plf 380.23 plf	) x 3585.43 lb ) x ) x ) x ) x ) x ) x ) x	25.00 5.50 1.00 0.00 0.00 0.00 0.00 WIDTH 0.00 0.00	x x x x x	1.00 / 27.75 / 1.00 / 0.00 / 0.00 / 0.00 / X BAR 0.00 / 0.00 / 0.00 /	2.00 25.00 1.00 0.00 0.00 0.00 0.00	x x x x x	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / FACTORS 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00	× × × × × × × × ×	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 6 LOAD EFF 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00	= = = = = = = = = = = = = = = = = = =	4752.84    2321.29    3585.43    0.00    0.00    0.00    0.00    0.00    0.00    10659.56    0.00    0
= 9 + + + + + + +	F# F4 - F5   	-		380.23 plf 380.23 plf	) x 3585.43 lb ) x ) x ) x ) x ) x ) x	25.00 5.50 1.00 0.00 0.00 0.00 0.00 0.00	x x x x	1.00 / 27.75 / 1.00 / 0.00 / 0.00 / 0.00 / 0.00 / X BAR 0.00 / 0.00 / 0.00 /	2.00 25.00 1.00 0.00 0.00 0.00 0.00	x x x x x	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / FACTORS 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00	x x x x x x	1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00	The state of the s	4752.84    2321.29    3585.43    0.00    0.00    0.00    10659.56    0.00    0
= 9 + + + + + + +	F# F4 - F5   	-		380.23 plf 380.23 plf	) x 3585.43 lb ) x ) x ) x ) x ) x ) x ) x	25.00 5.50 1.00 0.00 0.00 0.00 0.00 WIDTH 0.00 0.00	x x x x x	1.00 / 27.75 / 1.00 / 0.00 / 0.00 / 0.00 / X BAR 0.00 / 0.00 / 0.00 /	2.00 25.00 1.00 0.00 0.00 0.00 0.00	x x x x x	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / FACTORS 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00	× × × × × × × × ×	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 6 LOAD EFF 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00	= = = = = = = = = = = = = = = = = = =	4752.84    2321.29    3585.43    0.00    0.00    0.00    10659.56    0.00    0
= 9 + + + + + + +	F# F4 - F5   	-		380.23 plf 380.23 plf	) x 3585.43 lb ) x ) x ) x ) x ) x ) x ) x	25.00 5.50 1.00 0.00 0.00 0.00 0.00 0.00	x x x x x	1.00 / 27.75 / 1.00 / 0.00 / 0.00 / 0.00 / 0.00 / X BAR 0.00 / 0.00 / 0.00 /	2.00 25.00 1.00 0.00 0.00 0.00 0.00	x x x x x	1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / 1.00 / FACTORS 1.00 / 1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00	× × × × × × × × × × × × × × × × × × ×	1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00	= = = = = = = = = = = = = = = = = = =	4752.84    2321.29    3585.43    0.00    0.00    0.00    0.00    0.00    0.00    10659.56    0.00    0
= 9 + + + + + + +	F# F4 - F5   	-		380.23 plf 380.23 plf	) x 3585.43 lb ) x ) x ) x ) x ) x ) x ) x ) x ) x ) x	25.00 5.50 1.00 0.00 0.00 0.00 0.00 0.00	X X X X X	1.00 / 27.75 / 1.00 / 0	2.00 25.00 1.00 0.00 0.00 0.00 0.00 0.00 0.00	x x x x x x	1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00	x x x x x x x x x x x x x x x x x x x	1.00 / 1.00 /	1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00	=	4752.84    2321.29    3585.43    0.00



### WIND FORCE SUMMARY:

ALLOWABLE STRESS DESIGN SHEAR FORCE SUMMARY:

ALLOWAL	LINE	THE SECRET PROPERTY OF THE PRO	R FORCE SUMMAR' D (STRENGTH)	1.	matramentum di		V WIND (ASD)
	LINE	= 0 77114	0.00 lb	X	φ 0.60	-	0.00 lb
		=	0.00 lb	X	0.60	=	0.00 lb
		=	0.00 lb	×	0.60	=	0.00 lb
		=	0.00 lb	x	0.60	=	di 00.0
		==	0.00 lb	x	0.60	=	0.00 lb
1		=	0.00 lb	x	0.60	=	0.00 lb
I		=	0.00 lb	x	0.60	=	0.00 lb
1	_	=	0.00 lb	×	0.60	=	0.00 lb
1		=	0.00 lb	x	0.60	_	di 00.0
'		=	0.00 lb	x	0.60	_	di 00.0
I		=	0.00 lb	x	0.60	1000	0.00 lb
l		_ =	0.00 lb	×	0.60	=	0.00 lb
1		=	0.00 lb	x	0.60	=	0.00 lb
	1	=	0.00 lb	2	0.60	=	0.00 lb
	] -	=	0.00 lb	X X	0.60	=	0.00 lb
		=	0.00 lb	x	0.60	=	0.00 lb
	-	=	0.00 lb	x	0.60	=	0.00 lb
1		=	0.00 lb	x	0.60	=	0.00 lb
<b></b>			0.00 lb	X	0.60	=	0.00 lb
		=	0.00 lb	B	0.60	=	0.00 lb
Ĭ		=	0.00 lb	X X	0.60	=	0.00 lb
Į.	-			8			
1		=	0.00 lb	×	0.60	=	0.00 lb
	-		0.00 lb	X	0.60	=	0.00 lb
	_	=	0.00 lb	Х	0.60	=	0.00 lb
	-	=	0.00 lb	Х	0.60	=	0.00 lb
	-	=	0.00 lb	X	0.60	=	0.00 lb
	-	=	0.00 lb	Х	0.60	=	0.00 lb
	-	<b>=</b>	0.00 lb	Х	0.60	=	dl 00.0
	-	=	0.00 lb	Х	0.60	=	0.00 lb
	-	=	0.00 lb	Х	0.60	=	0.00 lb
	-	=	0.00 lb	Х	0.60	=	0.00 lb
	1 -	=	0.00 lb	Х	0.60	=	0.00 lb
	-	<b>=</b>	0.00 lb	Х	0.60	=	0.00 lb
	1 -	=	0.00 lb	Х	0.60	=	dl 00.0
	-	=	0.00 lb	Х	.0.60	=	0.00 lb
	_	=	0.00 lb	X	0.60	=	0.00 lb
	1	==	2008.36 lb	Х	0.60	=	1205.02 lb
	2	=	2008.36 lb	X	0.60	=	1205.02 lb
1	3	=	1711.02 lb	Х	0.60	=	1026.61 lb
	4	=	4087.45 lb	Х	0.60	=	2452.47 lb
	5	=	3585.43 lb	х	0.60	=	2151.26 lb
	_	=	0.00 lb	X	0.60	=	0.00 lb
	-	=	0.00 lb	Х	0.60	==	0.00 lb
டி	-	=	0.00 lb	Х	0.60		0.00 lb
ROOF	l -	=	0.00 lb	Х	0.60	=	0.00 lb
Ĭ.	-	=	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 0.00 lb	X X	0.60 0.60	=	0.00 lb 0.00 lb
ļ	-	and the second s	Augustus and a supplied to the supplied of the supplied to the			=	CONTRACTOR OF THE PROPERTY OF
	6 7	=	6025,08 lb 6025.08 lb	X	0.60 0.60		3615.05 lb 3615.05 lb
	8	=	8840.29 lb	X		=	
		=		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 dl 00.0	X	0.60		0.00 lb
	-	=	di 00.0	X	0.60	=	0.00 lb
	-	=	2	X	0.60	=	0.00 lb 0.00 lb
Ř	-	=	0.00 lb 0.00 lb	X	0.60	=	
2ND FLOOR	_	=		X	0.60		0.00 lb
교	_	=	0.00 lb 0.00 lb	X	0.60 0.60	=	0.00 lb
è	_		di 00.0 di 00.0	X			0.00 lb
72	_	=		X	0.60	=	0.00 lb
	-	=	0.00 lb	X	0.60	=	0.00 lb
	-	=	0.00 lb	х	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	х	0.60	=	0.00 lb
	-	=	dl 00.0	х	0.60	=	0.00 lb
	-	=	0.00 lb	X	0.60	=	0.00 lb
	-	=	0.00 lb	Х	0.60	=	0.00 lb
	-	=	0.00 lb	X	0.60	=	0.00 lb

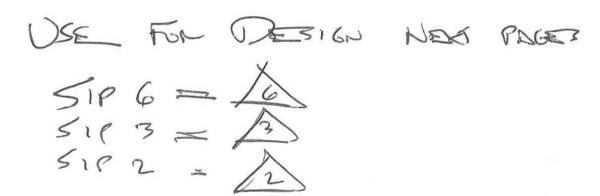
2/3

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)<sup>1,2</sup>

	Framing	M	inimum Facing Connection	ons²	Shear Strengt
Spline Type <sup>3</sup>	Minimum SG <sup>4</sup>	Chord <sup>2</sup>	Plate <sup>2</sup>	Spline <sup>3</sup>	h <sup>5</sup> (plf)
	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
Proof.	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
Block, Surface, or Lumber Spline (Type S, Type L)	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
	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

<sup>&</sup>lt;sup>1</sup> 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)

<sup>&</sup>lt;sup>5</sup> 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.



This listing report is intended to indicate that NTA, Inc. has evaluated the product described and found it to be eligible for labeling. Product not labeled as specified herein is not covered by this report. NTA, Inc. makes no warranty, either expressed or implied, regarding the product covered by this report.



<sup>&</sup>lt;sup>2</sup> Chords, hold-downs and connections to other structural elements must be designed by a registered design professional in accordance with accepted engineering practice.

<sup>&</sup>lt;sup>3</sup> Spline type at interior panel-to-panel joints only, solid chord members are required at each end of each shear wall segment.

<sup>&</sup>lt;sup>4</sup> 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.



### 2013CBC IBC SILL PLATE ANCHOR BOLT DESIGN:

ACI 318-14 CHAPTEF	R 17 phi Vn	>	Vua	CAST-IN-PLACE ANCHOR RODS UNDER TENSION AND SHEAR ONLY - NO TENSION LO. ACI 318-11 D.4.1.1 - MODERATE OR HIGH SEISMIC RISK REGION
APPLIED LOADS				
<del></del>	Vs	=	1332.00 lb O.K.	SERVICE LEVEL SHEAR FORCE BASED ON 0.7 pE FROM A TIMBER SILL PLATE
800000	ρΕ	=	V2012240250 V20	Net street of two cases selected enterings in the adversaries.
SHEAR	Vua	=	1902.86 lb	STRENGTH LEVEL DESIGN FORCE TO ANCHORS
NOTE: TENSION	Nua	=	0.00 lb	NO TENSION LOAD IN SILL PLATE ATTACHMENT
NCHOR 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
	fc	=	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 Ca2	=	12.00 in 5.00 in	MINIMUM ANCHOR BOLT SPACING SPACE FROM BOLT CENTER TO EDGE OF CONCRETE IN THE DIRECTION OF THE LOA
STEEL STRENGTH C ACI 318-11 D.6.1			HEAR L ELEMENT phi	A307 ANCHOR BOLTS REQUIRED
0.0.0.1	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 BREAKC	UT STRENG	TH (	OF ANCHOR IN SHEAR	
ACI 318-11 D.6.2	Vcb		Avc / Avco $\Psi_{ed}$ , V $\Psi_{c,V}$ $V_b$	ACI 318-11 (D-30)
(OI 0 10-11 D.0.2	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 ) SQ	
	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
	Ψc,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	
CI 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 $\Psi_{\text{ed},N}$ $\Psi_{\text{c},N}$ $\Psi_{\text{cp},N}$ $N_{\text{b}}$	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 (fc) 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 6370 56 lb	CAST-IN ANCHOR FACTOR PER ACI 318-11 D.5.2.7 ACI 318-11 (D-4)
	Ncb Vcp	=	6370.56 lb 12741.12 lb	ACI 318-11 (D-4) ACI 318-11 (D-30)
	phi Vcp	-	8918.78 lb	ACI 316-11 (U-30)
	IMMARY			CONCRETE PRODUCTION OF CONTRACT AND AREA OF CONTRAC
SHEAR CAPACITY SU				
SHEAR CAPACITY SU	phi Vn MIN.	=	1903.00 lb	BREAKOUT CONTROLS DESIGN
SHEAR CAPACITY SL		=	1903.00 lb 1903.00 lb	BREAKOUT CONTROLS DESIGN ANCHOR BOLT STRENGTH ASSUMING TIMBER PLATE DUCTILITY > Vua O.K.



## SHEAR WALL SCHEDULE ANCHOR VALUES:

	CI 318-14 CI 318-14 - NORMALIZED FOR WIND (0.7E / 0.6W ASD FORCES) N12 TABLE 11E w/ CD = 1.60 USE MIN: 1,332 lbs S N05 TABLE 11E w/ CD = 1.60 USE MIN: 1,332 lbs S
ALCULATIONS	1332 Ibs / BOLT IN CONCRETE NEAR FOUNDATION EDGE - 1-3/4" MIN ACI 318-14 1554 Ibs / BOLT IN CONCRETE NEAR FOUNDATION EDGE - 1-3/4" MIN ACI 318-14 - NORMALIZED FOR WIND (0.7E / 0.6W ASD FORCES) 1488 Ibs / BOLT - SINGLE SHEAR W/ CONCRETE SIDE - tm = 6" MIN NDS 2012 TABLE 11E W/ CD = 1.60 USE MIN: 1,332 Ibs S 1888 Ibs / BOLT - SINGLE SHEAR W/ CONCRETE SIDE - tm = 6" MIN NDS 2005 TABLE 11E W/ CD = 1.60 USE MIN: 1,332 Ibs S
BOLI / NAIL / VALUES - SEE TABLES BELOW FOR CAL  1.) BOLT CAPACITIES:	5/8" DIAMETER BOLTS IN CONCRETE, VS 5/8" DIAMETER BOLTS IN CONCRETE, Vx 2x TIMBER AGAINST CONCRETE, Z = 3x TIMBER AGAINST CONCRETE, Z =

1,488 lbs W 1,554 lbs W

	145 IDS / INAIL - LHROUGH 3/4" IN THICK PLY - BASED ON REDUCED EMBEDMENT CAPACITIES & 1,60 207 Ibs / INAIL - THROUGH 3/4" IN THICK PLY - BASED ON REDUCED EMBEDMENT CAPACITIES & 1,60	150 lbs / NAIL - THROUGH 3/4" in THICK PLY - BASED ON REDUCED EMBEDMENT CAPACITIES & 1 1,60
2.) NAIL CAPACITIES FOR 3/4" PLY:	2x w/ 20d SILL NAIL CAPACITY =	3x w/ 30d SILL NAIL CAPACITY =

IENT CAPACITIES & + 1.60	1,60
118 lbs / NAIL - THROUGH 3/4" in THICK PLY - BASED ON REDUCED EMBEDMENT CAPACITIES & 1.60	240 lbs / LAG - ts = 1-1/2" NDS 2012 TABLE 11.3 W/ CD =
<ol> <li>NAIL CAPACITIES FOR 1-1/8" PLY: 2x w/ 20d SILL NAIL CAPACITY =</li> </ol>	4.) LAG CAPACITIES: 2x w/ 1/4" DIA, LAG x 4" L CAPACITY =

4.) LAG CAPACITIES:	
2x w/ 1/4" DIA, LAG x 4" L CAPACITY =	240 lbs / LAG - ts = 1-1/2
3x w/ 3/8" DIA, LAG x 6" L CAPACITY =	320 lbs / LAG - ts = 2-1/2

" NDS 2012 TABLE 11J w/ CD = " NDS 2012 TABLE 11J w/ CD =

1.60

1.60

560 lbs / SCREW - BASED ON 100% VALUES PER LARR 25711 & ICC-ES ESR-2236 x CD =	695 lbs / CLIP - BASED ON 160% VALUES FOR F1 ORIENTATION PER LARR 25716
5.) SIMPSON SDS1/4 x 6" SCREWS =	6.) SIMPSON A35 CLIPS w/ 8d x 1-1/2 =

670 lbs / CLIP - BASED ON 160% VALUES FOR ORIENTATION G PER LARR 25717	545 lbs / CLIP - BASED ON 160% VALUES FOR H ORIENTATION
7.) SIMPSON LTP4 CLIPS =	8.) SIMPSON LTP5 CLIPS =

	ERM IREDICED		1hs 206 60 lhs	_	lhc 149 76 lhc
	SHEAR VALUE ISHORT TERM	225 60	272.00 lbs	_	247 60 lbs
	SHEAR VAL	141.00 lbs	170.00 lbs	Ann	
	DIAMTER CHECK	7.72	9.11	LESS THAN 6d	6.04
	EMBED REMAINDER	1.25 in	1.75 in	0.75 in	1.25 in
PLY	PLY THICKNESS	9	0.75 in	0.75 in	0
:0.50) AND:3/4"	PLATE THK.	1.50 in	1.50 in	2.50 in	2.50 in
2005 NDS TABLE 11N (G=	NAIL LENGTH	3.50 in	4.00 in	4.00 in	4.50 in
Z) VALUES FOR SILL PLATES -	REQ'D EMBED = 10 d	1.62 in	1.92 in	1.92 in	2.07 in
NAIL SINGLE SHEAR (	NAIL dia.	0.162 in	0.192 in	0.192 in	0.207 in
COMMON	NAIL d	16.00	20.00	20.00	30.00

	IN NAIL SINGLE SHEAR (Z) VALUES FOR SILL PLATES - 2	י (ב) שבסבסו סוו סוברו בעובס	- ZOUS INDS TABLE TIN (G=	LONA (ne.	IIO PLT					
NAIL d	NAIL dia.	REQ'D EMBED = 10 d	NAIL LENGTH	PLATE THK.	PLY THICKNESS	EMBED REMAINDER	DIAMTER CHECK	SHEAR VALL	JE 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 ir	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 ir	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	<b>LESS THAN 6d</b>	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 SIMPSON SDS SCREWS AND SIMPSON CLIPS LAG SCREWS PER NDS A307 ANDHOR BOLTS x?" MIN. EMBED

A307 ANCHOR BOLTS x 7" MIN. EMBED WIND LOAD RESISMIC VALUES WIND LOAD RESISTANCE VALUES ALL LOAD DURATION INCREASES FOR SEISMIC LOADS ARE LIMITED TO ALL LOAD NAIL AND SCREW DURATION INCREASES FOR WIND LOADS ARE PER NDS 200 1,60 ALL SIMPSON CLIP LOAD DURATION INCREASES FOR WIND LOADS ARE PER DT 160

SEISMIC   SILL   SILL NAILS FOR UP TO 3/4" SHTG   SILL NAILS FOR UP TO 1-1/8" SHT
VPE         SEISMIC         SILL           6         340 pif         2x           3         4         50 pif         3x           3         870 pif         3x           66         680 pif         3x           44         1020 pif         3x           43         1330 pif         3x           22         1740 pif         3x           22         1740 pif         3x

WINI	BESISTANCE SHEAR WAI	ZHICH.	AP WALL SHIMMADY	WYWAYTHING AN INSTRUCTOR IN THE PROPERTY OF TH	CHARLES THE STREET STREET, STR	CHANGE OF THE PROPERTY OF THE					***************************************	
			1 00 H	- 5			A Section of the second section of the second		energische between politige	ne de 1880 en de Carego de C		
	≥	SILL	SILL NAILS FOR	UP 10 3/4" SHTG SILL NAILS FOR UP TO 1-1/8" SHTG	SILL ATTACHMENT w/ SDS SCREWS SILL ATTACHMENT w/ LAGS	w/ SDS SCREW!	SILL ATTACH	AENT w/ LAGS	A35 CLIPS	LTP4 CLIPS	1	LTP5 CLIPS 5/8" ANCHOR BOLT
9	349 plf	Z X	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 6	8.25 in o.c.	23.90 in		ı	51 18 in
4	714 plf	×	30d @ 2.52 in o.c.			9.41 in o.c.			11.68 in	11.26 in	19.00 ri 01.00	26.12 in
က	931 plf	š	ſ	,		7.22 in o.c.			8 96 in	8 64 in	7.02 in	20.02 in
8	1218 plf	×	t	4	SDS 1/4 x 6 8	5.52 in o.c.	3/8 x 6 0	3.15 in o.c.	6.85 in	6.60 in	5.37 in	15.31 in
emberid		nasaena								)		
99	952 plf	ĕ	£	Í	SDS 1/4 x 6	7.06 in o.c.	3/8 x 6		8.76 in	8.45 in	6.87 in	19.59 in
4	1428 plf	×		i .		4.71 in o.c.		@ 2.69 in o.c.	5.84 in	5.63 in	4 58 in	13.06 in
8	1862 plf	×	:	t		3.61 in o.c.			4.48 in	4.32 in	3.51 in	10.02 in
22	2436 <b>plf</b>	χ̈́	_	1 Sandan	SDS 1/4 x 6 @	2.76 in o.c.			3.42 in	3.30 in	2.68 in	7.66 in

SL

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 pif FOR TABLES) ... SEE NEXT PAGE FOR RESULTS



### SHEAR WALL SCHEDULE

CO.		Т	Т	T	Т	Т	Т	T	Т	Т
SEISMIC SHEAR WALL CAPACITY 8		340 nlf	510 plf	665 plf	870 nlf		680 plf	1020 plf	1330 plf	1740 nlf
WIND SHEAR WALL CAPACITY		349 plf	714 plf	931 plf	1218 nlf		952 plf	1428 plf	1862 plf	2436 nlf
A35 CLIP LTP4 CLIP SPACING - TOP PLATE TO PLATE TO FRMG, ABV. <sup>10</sup> FRMG, ABV. <sup>10</sup>		20.0 in o.c.	10.0 in o.c.	8.5 in o.c.	6.5 in o.c.	gray and the same	8.0 in o.c.	5.5 in o.c.	4.0 in o.c.	3.0 in o.c.
A35 CLIP SPACING - TOP PLATE TO FRMG, ABV, <sup>10</sup>		18.00 in o.c.	8.00 in o.c.	8.00 in o.c.	6.50 in o.c.	and the second second	8.50 in o.c.	5.50 in o.c.	4.00 in o.c.	3.00 in o.c.
SDS1/4" x 6" SIMPSON WOOD SCREWS FOR SILL (OPTION)		18.0 in o.c.	9.0 in o.c.	6.0 in o.c.	5.5 in o.c.	Charles of Children age	7.0 in o.c.	4.5 in o.c.	3.5 in o.c.	2.5 in o.c.
SILL LAG SPACING (OPTION)		1/4 in DIA @ 8.0 in o.c.	3/8 in DIA @ 5.0 in o.c.	3/8 in DIA @ 4.0 in o.c.	3/8 in DIA @ 3.0 in o.c.		3/8 in DIA @ 4.0 in o.c.	3/8 in DIA @ 2.5 in o.c.	3/8 in DIA @ 2.0 in o.c.	
SILL NAIL SPACING <sup>5</sup> FOR UP TO 1-1/8" THK. SHEATHING		20d @ 4.0 in o.c.	1	•				1		•
SILL NAIL SPACING <sup>5</sup> FOR UP TO 3/4" THK. SHEATHING		16d @ 5.0 in o.c.	30d @ 3.5 in o.c.	30d @ 2.7 in o.c.			30d @ 2.6 in o.c.	,	•	
FRAMING @ ADJOINING EDGES <sup>©</sup>		2x	3x	3x	3х		3x	3x	3x	3×
SPACING SILL PLATE ANCHOR BOLT SPACING 38 FRAMI AT TO 4 ADJOI PLYWOOD ANCHOR EDGES BOLTS PACING 38 FRAMI		5/8" in DIA. @ 47.0 in o.c.	5/8" in DIA. @ 24.0 in o.c.	5/8" in DIA. @ 16.0 in o.c.	5/8" in DIA. @ 15.3 in o.c.		5/8" in DIA. @ 16.0 in o.c.	5/8" in DIA, @ 12.0 in o.c.	5/8" in 011 @ 8.0 in o.c.	5/8" in DIA. @ 7.7 in o.c.
SILL PLATE TO ANCHOR BOLTS		2x	З×	3×	3×		3x	3x	3x	3×
SPACING AT PLYWOOD EDGES <sup>2</sup>		6" o.c.	4" o.c.	3" o.c.	2" o.c.		6" o.c.	4" o.c.	3" o.c.	2" o.c.
SHEAR WALL NAIL SIZE <sup>1</sup>	STTI	10d	10d	10d	10d	'ALLS 7	10d	10d	10d	10d
MATERIAL THICKNESS STRUCT 1 PLY., UNO	SINGLE SIDED SHEAR WALLS	15/32" PLY	15/32" PLY	15/32" PLY	15/32" PLY	DOUBLE SIDED SHEAR WALLS	15/32" PLY	15/32" PLY	15/32" PLY	15/32" PLY
MARK	SINGLES	9	4	36	7°	DOUBLE	99	446	33	22°

### 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 A1 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 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
  - 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 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. 6.)
- 8) LOAD VALUES ARE BASED ON CBC AND LABC TABLE 2306.41.
  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 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 OF HORIZONTFALLY ACROSS STUDS.

  D) WHERE STUDS ARE SPECED 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 STAGGERED.

  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 CONORETE AT 28 DAYS.

  H) PREADLY PLOIT HOLES FOR SILL THATE 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 NAMETER FOR NAILS LARGER THAN 204.

  K) STRUCTURAL OBSERVATION IS REQUIRED FOR ALL PANNELS.

DATE 7/7/2017 SHEET \_\_\_\_\_\_



## STRAP HOLD DOWN CAPACITY SCHEDULE:

UPLIFT CAPACITY IS BASED ON THE MINIMUM RATING  $\times\,0.75$  SIMPSON STRAPS

HOI D DOWN IMEG	HOLD DOWN IMEG RATING	POST	TEST BATING	I A DEDODT	ILO DEBODITI	ADD 25040 25% BEDITON		-		
					ゴード ファート	LA NELONI ICO NELONI LAKK 299 IU 29% REDUCTION MAA DEFLECTION NAIL SIZE	MAX DEFLECTION		IOTAL NAILS	
		.,								the Franchiston Control
MST37	2710.00 lbs	**	2710.00 lbs	RR25713	ESR-2105	2032.50 lbs	0.030	16d	22	
48	4205.00 lbs	*	4205.00 lbs	RR25713	ESR-2105	3153.75 lbs	0.030	16d	34	
09	4605.00 lbs	**	4605.00 lbs	RR25713	ESR-2105	3453.75 lbs	0.030	16d	34	
72	6505.00 lbs	4×	6505.00 lbs	RR25713	ESR-2105	4878.75 lbs	0.030	16d	48	muneev-
					,					***************************************
		_								-
CMST12-42	10710.00 lbs	×4	9215.00 lbs	RR25713	ESR-2105	6911.25 lbs	0.030	16d	84	
T14-33	7755.00 lbs	<del>*</del>	6490.00 lbs	RR25713	ESR-2105	4867.50 lbs	0.030	16d	99	*******
5-11	2080.00 lbs	4×	1705.00 lbs	RR25713	ESR-2105	1278.75 lbs	0.030	10d	22	EEE010-
										-
ADDLICOS (INTERPORTUBATION CONTRACTOR										

POST ABOVE	HOLD DOWN STRAP	# NAILS ABV.	16" MAX STAP CLEAR SPAN	# NAILS BEL.	POST BELOW
	1 60	#		#	ă.

MST STRAP CALCULATION	ULATION			
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 BASI	E ON 2012 I	NAIL VALUES BASE ON 2012 NDS 11P - 16d w/ G=0.5	.5	



### **HOLD DOWN CAPACITY SCHEDULE:**

HOLDOWN CAPACITIES BASED ON CODE TABLES AND RESEARCH REPORTS

### SIMPSON HOLD DOWN CAPACITIES

SIMPSON	POST	ANCHOR	CATALOG CAPACITY	da	ICC	ICC CAPACITY	ICC STRENGTH	da	LARR	75% CAP.
HOLD DOWN	SIZE	DIAMETER	lbs	in	REPORT	lbs	lbs	in	REPORT	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	CAPACITY	de	F	ICC	ICC CAPACITY	ICC STRENGTH	LARR	LARR CAP
ANCHOR	BOLT	DIAMETER	lbs	in	in	REPORT	lbs	lbs	REPORT	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.	Į.		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		1	25828	45808.00
	1 1/4	1.250	22580.0	14.0	21.00	N.A.			-	-

### HOLD DOWN SUMMARY

SIMPSON	POST	ANCHOR	ANCHOR	75% ASSEMBLY CAPACITY
HOLD DOWN	SIZE	LA CITY	ALTERNATE	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	ds
lbs	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	



HOFFMAYER INC. STRUCTURAL ENGINEERING

0.77 2.86 × 0.0.78 0.0.88 × 0.0.75 0.0.75 0.0.75	INE VS	VSTRENGTH	A	8	O	D	Ε	u.	9	Ŧ	LENGTH	v ASD = 0.7 V/L	TYPE	CAPACITY	I	×	M/H	2 W / H	2 W / H AD INSTED CAPACITY
26.00 ft + 75.0 ft + ft	-	6139.72 lb	20.00 ft +	14.00 ft +	###	# H	##	ff.		ff=	L		SIP6	360.00	10.00	14.00	0.71		360.00
D	2	6139.72 lb	26.00 ft +	7.50 ft +	##	##	f+	ŧ		4=	= 33.50 ft	•	SIP6	360.00	10.00	7.50	133		360.00
D	m		3.50 ft +	4.50 ft +	##	#+	##	H+	+ #+	#	8.00 ft		SIP6	360.00	10.00	3.50	2.86	0.40	252.00
D		5340.70 lb	12.00 ft +	++	##	+ #	##	T.	+ # +	#	= 12.00 ft		9	340.00 plf	10.00	12.00	0.83	200	340 00 nif
D	10	4684.75 lb	3.50 ft +		##	##	##	#	+#	#=	7.00 ft		SIP2	920.00	10.00	3.50	2.86	070	644 no
D   9,00 ft			6.50 ft +	+#	+	##	<b>+</b>	Ť.	+ # +	#=#	= 6.50 ft		SIP2	920.00	00 6	6.50	1.38		920.00
b 12.00 ft		6782.73 lb	+ ¥ 00'6	##	##	##	##	f.	+#+	f i	= 9.00 ft		SIP3	720.00	00 6	000	100		720.00
D	_	5975.50 lb	12.00 ft +	##	+	+ #	##	÷₩	+#+	##	= 12.00 ft		4	510 00 olf	00 6	12 00	0.75		510 00 nif
##     <				3.50 ft +	+ #	+	+#	#	+ # +	#	7.00 ft		SIP2	920,00	9.00	3.50	257	0.78	715.56
		Ω	##	##	+ #	##	##	f.	+ # +	##	#	Jio							2000
## ## ## ## ## ## ## ## ## ## ## ## ##		Q	##	##	##	##	##	# #	+ # +	# #	#	Jio.	ń						
A     A <td></td> <td>Q</td> <td>##</td> <td><del>+</del></td> <td>##</td> <td>##</td> <td>##</td> <td>##</td> <td>+ # +</td> <td>ff.</td> <td>#</td> <td>Jid</td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td>		Q	##	<del>+</del>	##	##	##	##	+ # +	ff.	#	Jid							
A     A <td></td> <td>Q</td> <td>##</td> <td>+#</td> <td>+</td> <td>##</td> <td>##</td> <td>#</td> <td>+#</td> <td>ft=</td> <td>#</td> <td>. Jo</td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td>		Q	##	+#	+	##	##	#	+#	ft=	#	. Jo							
#+ #+ #+ #+ #+ #+ #+ #+ #+ #+ #+ #+ #+ #		Q	##	##	##	f +	##	#	+ # +	##	#	jo							
A+ A		Q	##	##	##	+#	##	H+	+ # +	ft =	#	Jid							
ft f		Q	##	##	+	##	+#	±#	+ ¥	ft=	#	. Ho	- 1						
+ A+		Q	##	##	##	##	##	#	+#	##	#	- to							
		Q	+ #	##	##	##	##	f H	+ #	#	#	Jid	ï						
		Q	##	##	###	##	##	#	+ # +	#=	#	Jid							



# WIND SHEAR FORCES TO SHEAR LINES AND ALLOWABLE STRESS SHEAR WALL FORCES: ALLOW FOR HWY < 3.5/1 PER CBC TA 2306.3.4 LINE VASD

1 ON 11/VV > 3.3/1 PER OBO 1A 2303.3.4	אוייטטט רווי	900.5.4															
ASD	Ą	В	O	D	Ш	4	9	T	LENGTH	VASD = V	= V / L	TYPE	CAPACITY	1	M	H/W/<35	AD II ISTED CADACITY
1205.02 lb	20.00 ft +	14.00 ft +	##	+ <del>1</del>	+ 1	+ 1	+ 11	††			35 44 nlf	SIDE	360.00	ı	44.00	7.4	1100 NO 03100000
1205.02 lb	26.00 ft +	7.50 ft +	#	#	#	#		. 4	03 00		31 70 10		000.000		20.1	- 1.0	Sec. of pil
4008 A1 IN	4 03 0	7						≝ .			55.87 pil	SPO	360.00		7.50	1.33	360.00 plf
1020.01	5.5U II +		+ #	<b>+</b> ⊭	<b>+</b>			₹			28.33 plf	SIP6	360,00		3.50	2.86	360.00 nlf
2452.47 lb	12.00 ft +		+ #	##	##			#	12.00		04.37 of	œ	349 00 plf		12.00	83	340000
2151.26 lb	3.50 ft +		+	±	#			<b>#</b>	7.00		07 32 plf	o Gio	100.0F0		25.50	8 6	048.00 pil
3815 OF IN	A 50 #		4	. d				= 4	00:		in 20. 100	2110	320.00		3.30	7.80	SZU.OU pit
00.000	1000		+	+	+			⊭	(= 6.50		56.16 plf	SIP2	920.00		6.50	1.38	920.00 plf
3615.05 lb	9.00 # +		+ #	+#	#			#	t = 9.00 ft		401.67 plf	SIP3	720.00	00 6	00 6	1.00	720 00 062
5304.17 lb	12.00 ft +		<b>+</b>	#	#			#	12.00		42 01 pF		744.00 214		25 55	- 10	10 00 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
8305 74 lb	250 4		4	4	. 4						- Da 10	r	14.00		2.00	0.73	7.14.00 pit
0000.74 ID	+ 11 00.0		+	+	+==			₽	1.00		13.68 plf	SIP2	920.00		3.50	2.57	920 00 plf
Q	+ #=		##	+ #	###			#	11	₽	ja						
q	<del>+</del>		#	+#	##			#	II	#	. <u>`</u> c	1					
<u>a</u>	<b>+</b>		+#	##	##			#	11	: ==	. <del>'</del>						
<u>a</u>	+		+ ₩	##	##			#	н	: ==	. <u>*</u>						
മ	##	+ <del> </del>	###	##	+ #	#	± ±	#	, ,11	: #	Ē	,	•				
Ω	<b>+</b>		#	##	##			#	H	: <b>#</b>	į	,					
Ω	+		##	##	+#			#	п	: <b>d</b> =	. <u>†</u>	:					
₽	+		##	+#	#			4=	н	: t=	<u> </u>	,					
<u>a</u>	<b>+</b> #		##	##	##			#	п	: d=	ž	,					
a	+		#	<del>+</del>	<b>+</b>			<b>.</b> #	ı	: #	<u> </u>						
				:	•			•		=	<u>5</u> .	,					

84401



SEISMIC OVERTURNING DESIGN:
ASD LEVEL FORCES
USE EQUATION ASCE 7-11 12.4.2.3 #8
(0.6 - 0.14 Sds) D + 0.7 p Qe
Sus = 0.633
0.6 - 0.14 Sds = 0.633
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

USJ	E	LENGTH HEIGHT ADJUSTED ASD OVERTURNING	0.7	WEIGHT	ADD LOAD ADD	LOAD (0.6-0.14Sds) D	WEIGHT ADD LOAD ADD LOAD (0.6-0.14Sds) D RST OVERTURNING +1- LEVER ARM ADDED UPLIFT WALL	- LEVER ARM	MADDED UPLIFT	WALL	NET UPLIFT	HOLDOWN	CAPACITY
			q	blt		lb ft-lb	ft-lb	#	Q	ABV.	Q	TYPE	Q
30.70	30.70	7	1264.1	150.0	370.0	53187.0	-27905.8	-0.50			-1431.1		
			1264.1	150.0	100.0	12529.6	5167.2	-0.50		,	382.8	MST48	3153.8
.7.7.	.7.7.		1282.9	150.0	100.0	43214.4	-9858.3	-0.50			-386.6		
			1282.9	150.0	100.0	3595.9	6026.1	-0.50		ŧ	860.9	MST48	3153.8
			1956.2	150.0	50.0	626.5	6220.2	-0.50		×	2073.4	HDU2-SDS2 5	2628.8
			1956.2	150.0	50.0	1035.6	7767.2	-0.50			1941.8	HD112-SDS2 5	2628 8
10,100	10,100		3115.4	150.0	100.0	9205.4	28179.4	-0.50			2450.4	MST48	3153.8
			4684.8	150.0	100.0	783.1	15613.5	-0.50		c	5204.5	CMST12-45	6911.3
468.48 16396.6			4684.8	100.0	100.0	626.5	15770.2	-0.50		,	5256.7	CMST12-45	6911.3
0.5	0.5		5436.4	100.0	100.0	2160.7	33175.7	-0.50	382.76	18	5912.0	HDU11-SDS2.5	8381.3
7.15	7.15		3593.3	100.0	100.0	4142.4	28197.1	-0.50	860.87	28	4178.2	HDU11-SDS2.5	8381.3
			333.3	100.0	100.0	7364.3	-3365.1	-0.50	2450.39	44	2157.8	HDU4-SDS2.5	3360.0
			635.1	100.0	100.0	626.5	1596.4	-0.50	5204.51	5A	5736.6	HHD014-SDS25	10282.5
			635.1	100.0	100.0	626.5	1596.4	-0.50	5256.72	5B	5788.8	HHDQ14-SDS2.5	10282.5
										9			
										•			



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

CAPACITY	٩		24 62 75	01:00:10	1	3153.75	2628.75	2628.75	3153.75	6911 25	6911.25	0201.60	000	0201.23	3360.00	10282.50	10282.50							
HOLDOWN	TYPE	CONTAINED TO THE PERSON OF THE	MCTAO	04-01	, 10,	MS 148	HDU2-SDS2.5	HDU2-SDS2,5	MST48	CMST12-45	CMST12-45	HD1141,8089 R	HD 14 6062 F	ה אפטפין ויטטון	HUU4-5U5Z.5	HHDQ14-SDS2.5	HHDQ14-SDS2.5	ī	Ť	ſ	ı	i	,	
NET UPLIFT	₽	-2836 50	-724 35	1824 40	1021.40	-217.28	1252.15	1139.93	1193,45	3279.18	3279 18	4654 52	2012.02	20.10	707 2.97	9400.90	9400.90							
WALL	ABV.	***************************************			,		,	•		,	,	á	α	) <	<b>†</b> i	<b>5</b> A	5B	1	.1	1	ı	i	1	
WEIGHT ADD LOAD ADD LOAD 0.6 D RST OVERTURNING+/- LEVER ARMADDED UPLIFT	요	de de contrate de la												1103 15	01.00	32/9.18	3279.18							
- LEVER ARI	<b>#</b>	-0.50	-0.50	0.50	0.00	00.0	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	0 50	0 0	-0.50	-0.50							
VERTURNING+/	유	-55311.67	-9738.17	-41347 64	-1520 95	7750.93	3/30.44	4559.71	13724.67	9837.53	9837.53	27927.10	2476182	17025 36	20000	16365.16	18365.16							
0.6 D RST O	ff-1b	62400.00	14700.00	50700.00	4218 75	72.00.70	00.667	1215.00	10800.00	918.75	918.75	2535.00	4860.00	8640.00	725.00	00.667	735.00							
ADD LOAD	q																							
ADD LOAD	plf	370.0	100.0	100.0	100.0	200	0.00	20.0	100.0	100.0	100.0	100.0	100.0	100.0	100	0.00	100.0							
WEIGHT	jjd	150.00	150.00	150.00	150.00	150.00	00.00	150.00	150.00	150.00	150.00	100.00	100.00	100.00	100.00	0000	100.00							
≥ :	q <sub>i</sub>	354.42	354.42	359.71	359.71	1283 27	1000.1	1283.27	2043.72	3073.22	3073.22	4686.48	3291.31	2138.78	5757 10	2	5457.19							
ASD FORCE OVERLURNING	dl-11	7088.33	4961.83	9352.36	2697.80	4491 44	*******	5//4/1	24524.67	10756.28	10756.28	30462.10	29621.82	25665,36	19100 16	2.00	19100.16							
ASD FORCE	pit	35.44	35.44	35.97	35.97	128.33	100.00	120.33	204.37	307.32	307.32	520.72	365.70	237.64	606.35	0 0	606.35							
	П	10.00	10.00	10.00	10.00	10.00	1000	10.00	10.00	10.00	10.00	9.00	9.00	9.00	00 6		9.00							
_	Manage Manage	∢	മ	⋖	Ω	∢	0	۰ ۵	∢ .	∢ :	മ	⋖	⋖	∢	∢		20							
# LENG-1	11	20.00	14.00	26.00	7.50	3.50	7 50	4.30	12.00	3.50	3.50	6.50	9.00	12.00	3.50		3.50							
# U #	#	_	<del>-</del>	5	7	က	۲,	· ·	4 1	n ı	S.	ဖ	7	œ	တ		מ							



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

PER ASCE 7-10 12.3.4.1  $\,\rho$  = 1.0 FOR DRIFT CALCULATIONS - ADJUST v ASD BY 1 /  $\rho$ (8/th\*3 / Eab + vh/1000Ga + Σ da h/b)
Cd 8xe / I
(8x x 1.00 )
0.0250 hsx
1.30 PER ASCE 7.10 12.3.4.1 8 x x 8 в с д

MPARE	1 0	.⊆	3 00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	2.70	2.70	2.70	2.70	2.70
	×	.⊆	0.59	0.68	0.56	0.93	2.50	2.06	2.13	2.18	2.18	2.34	1.79	162	2.41	2.41
No. of Concession, Name of Street, or other Persons and Persons an	S	; )	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
	δxe	<u>_</u>	0.148	0.171	0.141	0.232	0.625	0.514	0.533	0.546	0.546	0.585	0.448	0.404	0.602	0.602
	Z ds x h/b	.⊆	0.040	0.059	0.031	0.115	0.433	0.327	0.082	0.278	0.278	0.248	0.150	0.117	0.365	0.365
W. Company of the Com	vh/1000Ga	.⊆	0.105	0.105	0.106	0.106	0.162	0.162	0.175	0.210	0.210	0,295	0.269	0.141	0.218	0.218
	8vh^3 /Eab	<u>.</u> ⊆		0.004		0.007	0.022	0.017	0.010	0.052	0.052	0.032	0.017	0.008	0.013	0.013
7	ds SHRINK	<u>.</u> ⊆	0:020	0,050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0,050	0.050
LONGATION	ds CRUSH	<u>.</u> ⊑	0.020	0.020	0.020	0.020	0.020	0,020	0.020	0.020	0,020	0.020	0.020	0,020	0,020	0.020
VERTICAL B	ds SLIP	<u>.</u> ⊑	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0,010	0.010
	Ğa		13.0	13.0	13.0	13.0	13.0	13.0	19.2	24.0	24.0	24.0	19.0	24.0	24.0	24.0
USE	ds D/C	۳		0.003		900.0	0.072	0.067	0.018	0.017	0.017	0.099	0.070	0.076	0.062	0.062
	sp	.⊑		0.030		0.030	0.118	0.118	0.030	0.030	0.030	0.182	0.182	0.154	0.144	0.144
HOLDOWN	D/C	RATIO		0.0934		0.2100	0.6067	0.5682	0.5977	0.5793	0.5793	0.5426	0.3835	0.4940	0.4292	0.4292
0	TaHD	q		3154		3154	2629	2629	3154	6911	6911	8381	8381	3360	10283	10283
ASI	THD/p	Q	-1101	294	-297	662	1595	1494	1885	4003	4003	4548	3214	1660	4413	4413
	A POST	in^2		12.25		12.25	12.25	12.25	12.25	12.25	12.25	12.25	12.25	12.25	39.88	39.88
	HOLDOWN	TYPE	,	MST48	ı	MST48	HDU2-SDS2.5	HDU2-SDS2.5	MST48	CMST12-45	CMST12-45	HDU11-SDS2.5	HDU11-SDS2.5	HDU4-SDS2.5	HHDQ14-SDS2.5	HHDQ14-SDS2.5
STRENGTH	1.4 v / p	Jld	136.1	136.1	138.2	138.2	210.7	210.7	335.5	504.5	504.5	786.6	568.1	375.4	580.5	580.5
	v ASD	plf	126.4	126.4	128.3	128.3	195.6	195.6	311.5	468.5	468.5	730.4	527.5	348.6	539.0	539.0
	ΡĽΥ	Ė			0 SIP6											
		ft	10.00	10.00	10.00	10.0(	10.00	10.00	10.0	10.0	10.00	9.00	9.00	9.00	9.00	9.00
	LENGTH	ft	20.00	14.00	26.00	7.50	3.50	4.50	12.00	3.50	3.50	6.50	9.00	12.00	3.50	3.50
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### SHEAR WALL SUMMARY:

COMPARE SEISMIC AND WIND REQUIREMENTS FOR FINAL RESULTS

LINE	LENGTH	SEISMIC FORCE	WIND FORCE	GOVERNING FORCE	WALL TYPE	HOLD DOWN	WALL CAPACITY	and a constant
#	ft	plf	plf	plf	PER SCHED.	SIMPSON	plf	
1	20.0	126.4	35.4	126.4	SIP6	*	360.0 S	PHONON COLUMN
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	
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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 + Ω0 Qe Sds = \_\_\_\_0.633

HOFFINOND INC. STRUCTURAL ENGINEERING

0.6 - 0.14 Sds Ωo

= 0.633 = 0.511 = 3.00

OVERSTRENGTH	0.6 - 0.14 Sds) D + On Oe	lbs	264.3	2087.0	1324.0	2658.4	5057.8	48197	6701.5	12351.8	12351.8	15327.8	10951.0	6863 7	13852.8	13852.8
	WALL	ABV	NAME AND POST OF THE PERSONS ASSESSED.	,	1			,	,		1	18	2B	4 4	5A	5B
	LEVER ARMIADDED UPLIFT	q	MACONETSIER CONTRACTOR									2096.98	2658.37	6701.51	12351.77	12351.77
	1+/- LEVER ARM	#=	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50
	NET O.T.	#-P	5154.3	28309.2	33761.2	18608.6	15173.5	19278.6	77067.4	37055.3	37055.3	79384.9	70487.2	1864.7	4503.2	4503.2
	(0.6-0.14Sds) D RST	ft-lb .	53187.0	12529.6	43214.4	3595.9	626.5	1035.6	9205.4	783.1	783.1	2160.7	4142.4	7364.3	626.5	626.5
	EIGHT ADD LOAD ADD LOAD (0.6-0.14Sds) D RST	plf	370.0	100.0	100.0	100.0	50.0	50.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
	WEIGHT	plf	150.0	150.0	150.0	150.0	150.0	150.0	150.0	150.0	150.0	100.0	100.0	100.0	100.0	100.0
	Ωo Qe	മ	6731.7	6731.7	6832.2	6832.2	10417.6	10417.6	16590.9	24948.4	24948.4	28951.1	19135.8	1774.8	3382.2	3382.2
	Ωo OVERTURNING	대-IP	58341.2	40838.9	76975.6	22204.5	15800.0	20314.2	86272.8	37838.4	37838.4	81545.7	74629.6	9229.0	5129.6	5129.6
	' ADJUSTED V ASD Ωo OVERT	plf	126.41	126,41	128.29	128.29	195.62	195.62	311.54	468.48	468.48	604.04	389.25	37.03	70.57	70.57
	HEIGHT	¥	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	9.00	9.00	9.00	9.00	9.00
	LENGTH	¥	20.00	14.00	26.00	7.50	3.50	4.50	12.00	3,50	3.50	6.50	9.00	12.00	3.50	3.50
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### RICHMOND HOFFMAYER INC. STRUCTURAL ENGINEERING



 PROJECT
 VILLAGE NEST
 JOB
 2017-0610

 CLIENT

 ADDRESS
 EDEN, UT

 DESIGN
 HUUM

 ENGINEER
 J.H.

 DATE
 6/14/2017

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DIAPHRACUS:
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FX = 3947 2948 / 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
TO ROOF 1913191828 24 120818
)=0.7(394)24 / 18 V25 6' / 70PM
The Way
= 1-81-4)
1) Vy Roof
UMDX = 0.7(191)43/2(24)
= 120 PLP Roof
C LUIND STRENGTS UX
Fx = 42 } 332 628
F 39 3 2-807 5
tours 39 3-0084
V = 0.6 (30) 34 = 11960 0x Vy
244 From 2NP From
"KOP W/ SNOW PERVICES PIT 40/20 FOR ZEAPSF & LG SPOR
REF W/ SNOW PEDDULLES PIT 40/20 FOR 264 PSF \$ 16"SPING ROSE I SNOW PEDDULLES PIT 40/20 FOR 264 PSF \$ 16"SPING ROSE I SNOW OF OSB PLY GIT 40/20 T \$ 6 W/ LOX & 4", 12"
1 10 1 4 4 , 10
200 Front Bre" STWAT I ON THICKEN TEG PIT 48/14
1 100 0 6,12





PROJECT	VILLAGE NEST	JOB2017-0610
CLIENT	æ	
ADDRESS	EDEN, UT	
DESIGN	HUUM	
<b>ENGINEER</b>	J.H.	DATE 6/14/2017
	RICHMOND PE 44628 HOFFMAYER SE	5435 SHEET 22°C OF

CHOPDS & DRASGE:  [MAX CHOPD => (0.6(3-0=)24/6)/4 = 6-041]
USE SINGRE 2x TOR PLOTE W/ 1. 2x RIM PLODED CASP PLOTE SEE PG. #157
LAP CAR (L W) (2) BD NOILS e 6" OL ZINNOIL = 63 11/NAIL × 2 × 12/6 = 25211/6 LAS LAS CAR (R C-84/262 = 2.71 -> 3'
1.3E RIM CAS + 3'LAS PLATE + 3'LAS THE
51P) [] [] [] [] [] [] [] [] [] [] [] [] []

ING. A.
FICHMOND FOFFMAYER IN STRUCTURAL ENGINEERING

STAIR STRINGERS AND TREADS:

PELGOR SHEATHING
PERS PLAN
1/2" PLY OR IX SOLID
5/70/CK RISERS
5/4" PLY READS
1/4" PLY READS

	DEAD LOAD	MAX. RISE	MIN. RUN	FACTORED DEAD LOAD FOR SLOPE	RESIDENTIAL LIVE LOAD	PLACED ON TREADS FOR MAXIMUM STRESS	MAXIMUM NOTCH IN STRINGERS		ALLOWABLE SHEAR STRESS	ALLOWABLE BENDING STRESS	MODULUS OF ELASTICITY		ALLOWABLE SHEAR STRESS	ALLOWABLE BENDING STRESS	MODULUS OF ELASTICITY		ALLOWABLE SHEAR STRESS	ALLOWABLE BENDING STRESS	MODULIS OF FLASTICITY
	10.00 psf	7.50 in	10.00 in	15.54 psf	40.00 psf	300.00 lbs	6.00 in	ERTIES	95.00 psi	870.00 psi	1.60E+06 psi	RTIES	400.00 psi	1700.00 psi	1.30E+06 psi	RTIES	290.00 psi	2900.00 psi	2 OOF+OR pei
9	it	31		Ħ	H	8	я	ER PROP	ŧ	12	11	R PROPE	(I	и	в	ER PROPE	11	n	н
CACA WIND LINE	WD	RISE	RUN	'GW	WL	김	NOTCH	DF#2 TIMBER PROPERTIES	3	먑	ш	LSL LUMBER PROPERTIES	¥	윤	ш	PSL LUMBER PROPERTIES	7	ПP	ш

STRINGER	ct	F'b	<b>МІ</b> ОТН ОЕРТН	)EPTH		MOMENT CAPACITY	SHEAR CAPACITY
SIZE			ω	۵	ò		
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	9.1	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	06.0	783.00 psi	3.00	13.25	7.25	1714.85 ft-lbs	5800.00 lbs
2x14 LSL	66.0	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	0.1	2903.37 psi	3.50	11.88	5.88	4871.40 ft-lbs	5483.33 lbs
4x14 PSL	96.0	2850.80 psi	3.50	14.00	8.00	8869.16 ft-lbs	7466.67 lbs

— SIMPSON H.33 ® EACH STRINGER

2x4 PLATE NAILED TO WOOD FILK.
OR 2x4 P.T.D.F. PLATE POWDER
SHOTT O CONC. FLR. @ 18" o.c.

Zx6 P.T.D.F. PLATE ANCHORED
TO CONC. FLR. W (2) 5/9 © A.B. MIN.

STRINGERS PER PLAN -

J-I/S. WAX

RUN IO" MIN.

-3/4" PLY OR 2x SOLID STOCK TREADS 1/2" PLY OR IX SOLID STOCK RISERS

BEAM PER PLAN -DBL. 2x6 MIN. STRINGERS PER PLAN 2XI4 @ 16" o.c. MIN. ----

		Ь			┗_			١			़					_	_	-	_	μ.			┝		_
	13.00 ft	1173.28	361.01	4×12 DF	1564.38	481.35	DBL 2x14 DF	1759.92	541.51	2x14 LSL	2346.56	722.02	4x12 PSL	2933.21	902.52	4x12 PSL	3128.75	962.69	4x12 PSL	3519.85	1083.03	4x12 PSL	4693.13	1444.04	4x12 PSL
	12.00 ft	999.72	333.24	2x12 LSL	1332.96	444.32	DBL 2x14 DF	1499.58	499.86	DBL 2x14 DF	1999.44	666.48	4x14 DF	2499.30	833.10	4x12 PSL	2665.92	888.64	4x12 PSL	2999.16	999.72	4x12 PSL	3998.88	1332.96	4x12 PSL
	11,00 ft	840.04	305.47	2x14 DF	1120.06	407.29	4x12 DF	1260.06	458.20	4x12 DF	1680.08	610.94	DBL 2x14 DF	2100.11	763.67	4x14 DF	2240.11	814.59	4x12 PSL	2520.13	916.41	4x12 PSL	3360.17	1221.88	4x12 PSL
	10.00 ft	694.25	277.70	2x14 DF	925.67	370.27	DBL 2x12 DF	1041.37	416.55	4x12 DF	1388.50	555.40	DBL 2x14 DF	1735,62	694.25	2x14 LSL	1851.33	740.53	2x14 LSL	2082.75	833,10	4x14 DF	2777.00	1110.80	4x12 PSL
AND SHEAR	9.00 ft	562.34	249.93	2x14 DF	749.79	333.24	2x14 DF	843.51	374.89	2x14 DF	1124.68	499.86	4x12 DF	1405.86	624.82	DBL 2x14 DF	1499,58	666.48	DBL 2x14 DF	1687.03	749.79	DBL 2x14 DF	2249.37	999.72	4x12 PSL
HORIZONTAL PROJECTED SPAN VS. STRIINGER MOMENT AND SHEAR	8.00 ft	444.32	222.16	2x14 DF	592.43	296.21	2x14 DF	666.48	333.24	2x14 DF	888.64	444.32	DBL 2x12 DF	1110.80	555.40	4x12 DF	1184.85	592,43	4x12 DF	1332.96	666.48	DBL 2x14 DF	1777.28	888.64	2x14 LSL
JECTED SPAN VS. S	7.00 ft	340.18	194.39	2x10 LSL	453.58	259.19	2x14 DF	510,27	291.58	2x14 DF	980.36	388.78	2x14 DF	850.46	485.97	2x14 DF	907.15	518.37	DBL 2x12 DF	1020.55	583.17	4x12 DF	1360.73	777.56	DBL 2x14 DF
HORIZONTAL PRO	6.00 ft	249.93	166.62	2x10 LSL	333,24	222.16	2x10 LSL	374.89	249.93	DBL 2x10 DF	499.86	333,24	2x14 DF	624.82	416.55	2x14 DF	666.48	444.32	2x14 DF	749.79	499.86	2x14 DF	999.72	666.48	2x12 LSL
		MOMENT	SHEAR	SIZE	MOMENT	SHEAR	SIZE	MOMENT	SHEAR	SIZE	MOMENT	SHEAR	SIZE	MOMENT	SHEAR	SIZE	MOMENT	SHEAR	SIZE	MOMENT	SHEAR	SIZE	MOMENT	SHEAR	SIZE
TOTAL UNIF.	LOAD	55.5 plf			74.1 plf			83.3 plf			111.1 plf			138.8 plf			148.1 plf			166.6 plf			222.2 plf		
STRINGER	SPACING	12 in o.c.			16 in o.c.			18 in o.c.			24 in o.c.			30 in o.c.			32 in o.c.			36 in o.c.			48 in o.c.		

16.00 ft 444.72 2x14 LSL 2x14 LSL 2x14 LSL 2x14 LSL 392.43 4x12 PSL 382.45 6x12 PSL 385.45 6x12 PSL 4x12 PSL 4x

15 00 ft 156 00 ft 156 20 ft 55 40 belt 2x14 DF 2082.75 55 40 ex14 DF 2343.09 624.82 4x12 PSL 4x12 PSL

1400 ft 1360.73 388.78 288.78 288.78 288.78 288.78 289.79

4x14 PSL	(,	
4x14 PSL		~
4x14 PSL	761 F	JUNE TO BE
4x12 PSL	TX N	メイス
4x12 PSL	#35(	
PSL		

POINT LOAD - APPLY ANYWHERE ON TREAD	MAXIMUM TREAD SPAN	MAXIMUM TREAD SHEAR - LOAD AT SIDE REACTION	MAXIMUM TREAD MOMENT - LOAD AT MID SPAN	DF#2	DF#2	FLAT TREAD - TRUE SIZE = 2" x 8" MIN.	0.K.	0.K,
300.00 lbs	48.00 in	300.00 lbs	3600.00 in-lbs	4.74 in^2	4.11 in <sup>4</sup> 3	2x8 MIN	18.13 in^2	4.83 in^3

TREAD DESIGN

V MAX M MAX A REQ'D S REQ'D TRY 4425 CANOGA AVENUE WOODLAND HILLS CA 91364
PHONE (818) 347-7008 FAX (818) 883-8869
STAIR STRINGERS-3.xis.STRINGERS

### 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 3495 SHEET 73 OF

TSALCONY GUARD:
PESIDENTIAL LOD 20011 1 200 12 1 200 14 200
CLOSET BALUSTRADE SUCH THAT B SCHELL 4"D AND LARGER CAN NOT PASS TOPPOUR 42"
2'n"
MILGHATING 7
+++3"+/_
V = 200 H
$M = 200 \times 45 = 9000 \text{ IN-IL}$ THY CIOID STEEL - $S_y = 16-600 \text{ PSI}$ $S_{PRODED} = 9000/16-600 = 0.5361N^3$
USE TS 2/2 × 1/2 C1010 MELH, WELDED TOBING - WELD - ROLE TO HSS S= 0,5931, AT 11gr

### RICHMOND HOFFMAYER INC. STRUCTURAL ENGINEERING



TIT - 1155 0 1 4 3	Renac
TOLT TO HSS & x 4 x 3/	2360 IL
FB = 200/2 +/- 900-0/4 =	
= 100 +/ 2250	4" + -> -
= 2350 ll \$ 215016	* + -> <
DBL SHEW 5/8" & MB	A307
1 n/a = 7.36 k/But >>2	, 3 k k √
	+5 2/2×1/2×1194
Was:	45/e" \$ M.B.)
$l=4$ $e_{x}=al=45$	
a = 45/4 = 11.25	3 ex 43/8
K = O	3//40
C=0.439 -> AISCTA. 8-4	TO 1/2" SISPOLE
TK AK	3ex4×1/8
$P_{MN} = \frac{2(0.2k)}{0.439(1)4} = 0.23$	PLAN
	H <u>aš</u>
)SE 1 4 SARDLE "& MO	
)SE   1 5 MOLE 1 M. S.	FILLERS to TX

### RICHMOND HOFFMAYER INC.



PROJECT VILLAGE NEST JOB 2017-0610

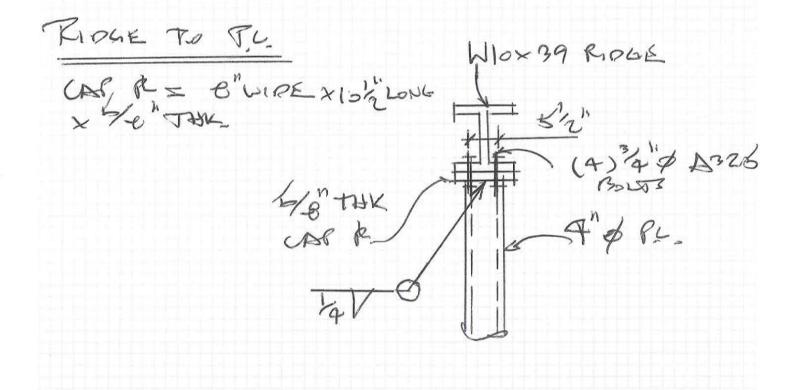
CLIENT 
ADDRESS EDEN, UT

DESIGN HUUM

ENGINEER J.H. DATE 6/14/2017

RICHMOND PE 44628 HOFFMAYER SE 3435 SHEET 737.0F

B3 & B4 TO B6	-> PELDSPAR UVN
TEN AISC TA10-90	1994 BE) B3
(2) 3/4 & ASSES BOLTE 3/6" PE - N THERESE CAP = 21.2 L V	
3/6" TE - N THREATS	中 建国 中
CAR = 21.2 K	
L= 5" MIN OK	W10x39 \ W10x26
Han= 6/e (3/e)	( 3/11) W10×26
20.23 % /4"	A226 POLUE





VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID:

2017-0610

Printed: 10 JUL 2017, 1:27PM

Steel Base Plate

File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Licensee : RICHMOND HOFFMAYE

Lic. #: KW-06002886

Description: P.C. BASE PLATE RIPOLE

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 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: Pipe4STD

2.96 in^2 Depth 4.5 in Area 4.5 in Width 6.82 in^4 lxx Flange Thickness 6.82 in^4 0.221 in lyy

10.50 in

10.50 in

Web Thickness

Plate Dimensions

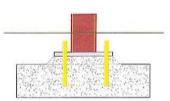
N: Length

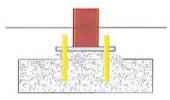
0 in

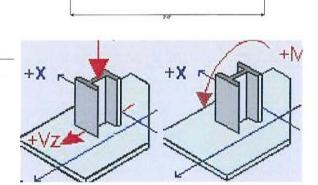
Support Dimensions

Width along "X" 24.0 in Length along "Z" 24.0 in

B: Width 0.750 in Thickness 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
	4 000 50		AND OF SECTION 12		5.50	

" P " = Gravity load, "+" sign is downward. "+" Moments create higher soil pressure at +Z edge.

"+" Shears push plate towards +Z edge.

0.000 k-ft

### **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 Pu : Axial ..... Mu: Moment ......

10 -1/2" x 10 -1/2" x 0 -3/4" 83.578 k

fu: Max. Plate Bearing Stress ....

Mu : Max. Moment ..... fb: Max. Bending Stress .....

Fb: Allowable:

Fy \* Phi

Fp: Allowable: min( 0.85\*fc\*sqrt(A2/A1), 1.7\* fc)\*Phi

Bending Stress Ratio

Bearing Stress Ratio

Bending Stress OK 0.758 ksi 2.550 ksi

0.297

4.512 k-in

32.082 ksi

32.400 ksi

0.990

Bearing Stress OK



VILLAGE NEST JJH

Project ID: MULTI UNIT PROJECT

2017-0610

Printed: 10 JUL 2017, 1:46PM

Steel Base Plate

File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29 Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886

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

7.58 in^2 Depth 6 in Area 6 in 39.5 in^4 Width lxx 0.349 in Flange Thickness lyy 39.5 in^4 Web Thickness in

Plate Dimensions

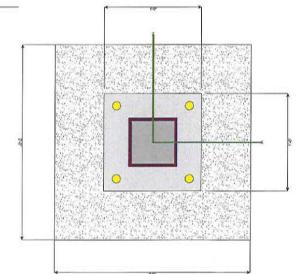
N: Length 12.0 in 12.0 in B: Width Thickness 0.8750 in

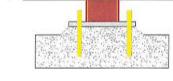
Column assumed welded to base plate.

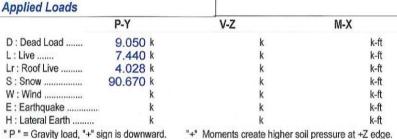
Support Dimensions

Width along "X" 24.0 in 24.0 in Length along "Z"

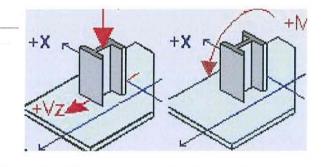








"+" Shears push plate towards +Z edge.



### 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 ..... Mu: Moment ......

159.652 k 0.000 k-ft Mu : Max. Moment ..... fb : Max. Bending Stress ..... Fb: Allowable: Fy \* Phi

Bending Stress Ratio

fu: Max. Plate Bearing Stress .... Fp: Allowable:

> min( 0.85\*fc\*sqrt(A2/A1), 1.7\* fc)\*Phi Bearing Stress Ratio

5.501 k-in 28.737 ksi 32.400 ksi

0.887

Bending Stress OK

1.109 ksi 2.550 ksi

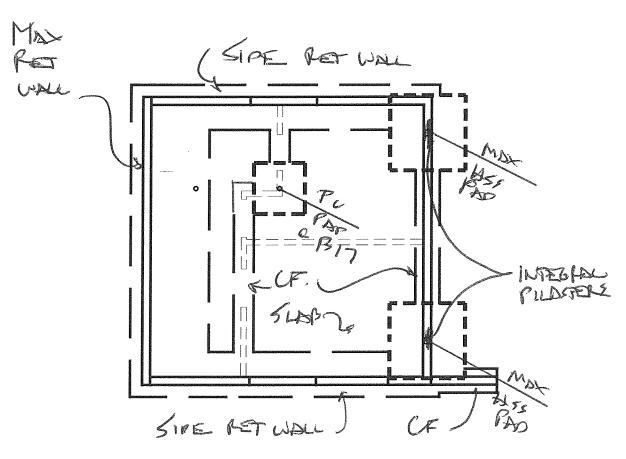
0.435

Bearing Stress OK





**PROJECT** FELDSPAR LOWER JOB 2017-0610 CLIENT VILLAGE NEST EDEN, UTAH ADDRESS DESIGN HUUM **ENGINEER** J.H. DATE 6/16/2017 SHEET 235 OF RICHMOND PE 49628 HOFFMAYER SE 3935



FELDSPAR LVN

FOUNDATION	TLAN	
FELOSPAC		
NTS		





 PROJECT
 FELDSPAR LOWER
 JOB
 2017-0610

 CLIENT
 VILLAGE NEST

 ADDRESS
 EDEN, UTAH

 DESIGN
 HUUM

 ENGINEER
 J.H.

 RICHMOND PE 49628
 HOFFMAYER SE 3935

 SHEET 36
 OF

Max Res	SIPE PET, WALL  POPE PAT TO THE PATE OF TH
	CSIPE RETURN

QUARTZ LVN

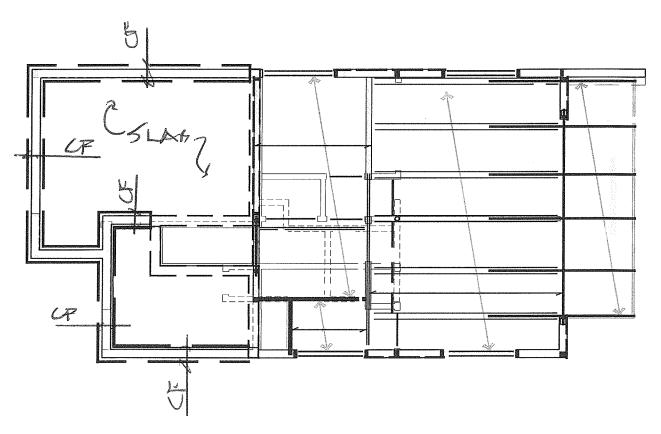
FOUNDATION PLAN

QUARTZ LUN

N.T.S.



PROJECT	FELDSPAR LOWER	JOB	2017-0610
CLIENT	VILLAGE NEST		
ADDRESS	EDEN, UTAH		
DESIGN	HUUM		
ENGINEER	J.H.	DATE	6/16/2017
	RIGHMOND PE 49628 HOFFMAYER SE 3935	SHEET	237 OF



FELDSPAR UVN

FELOSIAL LIVN
N.T.S.



$\mathbb{Z}$	
· /// <i>=</i>	

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 .	72 <i>R</i> of

	SIDE PUST	
SUPLHABLE PET UDIL		Biz
	GROW FET WAY	

PELDSPAR UVN

LOUES FONDATION FLAN FEDSTAL UNI

### 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 4962B	HOFFMAYER SE 3435	OUEET	120 DE	

SOILS REPORT:
1 GES 01628-003 Nov. 9, 2012 01628-015 DEC. 1, 2016
OVER EXCAVATE 24" BELOW FOOTING BOTTOM FOOTINGS ON 24" STRUCTURAL FILL
RALLOW = 2600 PSI? BENDRING CAPACITY
RECOMMENDED 42" BELOW LOWEST APPRIENT GERRE FOR FROST
MIN FOOTING = 20"  MAX FOOTING = 5' CONTINUOUS & 7' SOUBLE
WERE ABRILLIE & GA (LF 2:1
PASSIVE = 360 PCF PASSIVE RETULED BY 1/2 WHEN USED WITH FRICTION
MIN. 5" SLAB VI # A @ 16" OL EDLA WAY OVER 10 MIL VAROR BARRIER ON GROWER ON ARRENTED GRAPPE.

RICHMOND
HOFFMAYER INC.
STRUCTURAL ENGINEERING



PROJECT	VILLAGE NEST	_ JOB	2017-0610	
CLIENT		7/1 113-21-1		
<b>ADDRESS</b>	EDEN, UT			
DESIGN	HUUM			
ENGINEER	J.H.	DATE	6/14/2017	
	RICHMOND PE 49628 HOFFMAYER SE 59	35 CHEET	2 A OF	

CONTINUOUS FOOTINGS:	
MAX LONG = (15.e + 20 + 264) 1/2 + 15.2 × 20'	- ROSP Haus
+(14-3+40)1.33 = 2625 (LF	2NP FLOA
20" WIPE X AT" BEOW GRARE WIGHT = 2500 PSIZ X 20/12 = 4167PLZ	
JSE 20" WIDE X AR" BROW GRAF WITH SOUS FOOTINGS WITH 4 EDUS WAY & 24" 50 -	) <u>L</u>



0.00 in

VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

File = d:\ENERCALC Projects\2017-0610.ec6

14

Printed: 11 JUL 2017, 3:33PM

### Cantilevered Retaining Wall

Lic. # : KW-06002886

Retained Height

Slope Behind Wall

Wall height above soil

Height of Soil over Toe

Water height over heel

Vertical component of active

Lateral soil pressure options:

Description:

Criteria

MAX RETAINING WALL - FELDSPAR & QUARTZ LVN

9.00 ft

1.00 ft

2.00:1

0.0 ft

30.00 in

### 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

Calculations per ACI 318-14, ACI 530-11, IBC 2015, CBC 2016, ASCE 7-10

ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29 Licensee: RICHMOND HOFFMAYER

### 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

USED for Soil Pressure.

USED for Sliding Resistance.

USED for Overturning Resistance.

### **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

for passive pressure

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 =	2.73 OK
Sliding =	1.49 Ratio < 1.5!
Total Bearing Load =resultant ecc. =	8,913 lbs 3.18 in
Soil Pressure @ Toe = Soil Pressure @ Heel =	1,036 psf OK 1,706 psf OK
Allowable = Soil Pressure Less Than All	2,500 psf
ACI Factored @ Toe = ACI Factored @ Heel =	1,089 psf 1,792 psf
Footing Shear @ Toe = Footing Shear @ Heel = Allowable =	27.2 psi OK 21.8 psi OK 75.0 psi
Sliding Calcs (Vertical Compone Lateral Sliding Force = less 50 % Passive Force = - less 100% Friction Force = -	
Added Force Req'd =for 1.5 : 1 Stability =	0,0 lbs OK 33.0 lbs NG

S	tem Construction	7	Top Stem	2nd	
	Design Height Above Ftg	ft =	Stem OK 4.00	Stem OK 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 ——————————————————————————————————	=	0.263	0.806	
	Total Force @ Section	lbs =	1,280.0	3,947.2	
	MomentActual	ft-I =	2,133.3	12,274.9	
	MomentAllowable	ft-I =	8,121.3	15,222.0	
	ShearActual	psi =	17.2	53.2	
	ShearAllowable	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		is assessed worth.	1000 11000	
	fc	psi =	2,500.0	2,500.0	
	Fy	psi =	60,000.0	60,000.0	

### Load Factors

 Dead Load
 1.200

 Live Load
 1.600

 Earth, H
 1.600

 Wind, W
 1.600

 Seismic, E
 1.000



=

=

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

Project ID:

File = d:\ENERCALC Projects\2017-0610.ec6

Licensee: RICHMOND HOFFMAYER

ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

17-0610

Printed: 11 JUL 2017, 3:33PM

### **Cantilevered Retaining Wall**

**Footing Dimensions & Strengths** 

f'c = 2,500 psi Footing Concrete Density

2.00

Lic. #: KW-06002886

Total Footing Width

Key Distance from Toe

Footing Thickness

Description:

Toe Width Heel Width

Key Width Key Depth

Min. As %

Cover @ Top

MAX RETAINING WALL - FELDSPAR & QUARTZ LVN

4.75 ft

1.75

6.50

12.00 in

12.00 in

12.00 in

5.00 ft 60,000 psi 150.00 pcf

0.0018

@ 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	Ň
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

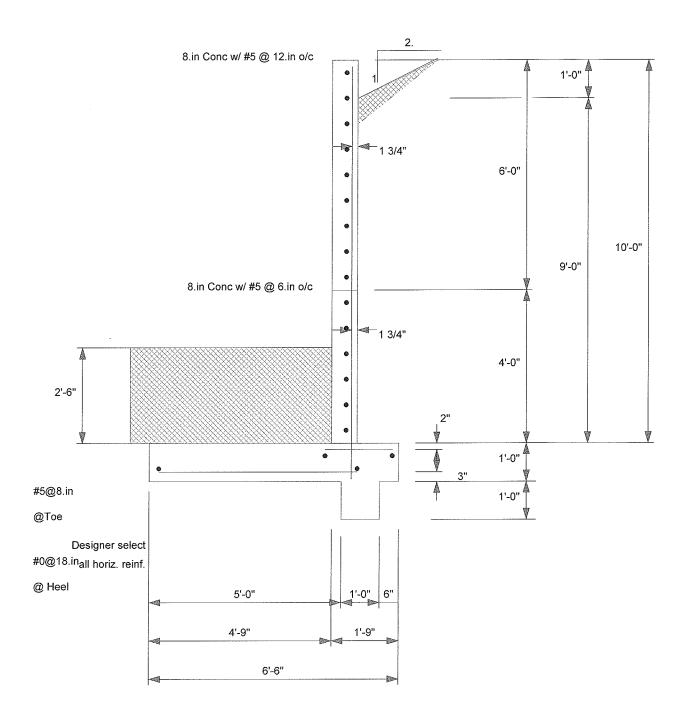
Other Acceptable Sizes & Spacings

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

		0	VERTURN	ING	MARKET LA			RI	ESISTING	
Item		Force lbs	Distanc ft	е	Moment ft-lb	_		Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	3,556.1	3.5	1	12,495.6	Soil Over Heel	=	1,072.5	5.96	6,390.3
Surcharge over Heel	=					Sloped Soil Over Heel	=	32.3	6.14	198.1
Toe Active Pressure	=	-245.0	1.17	7	-285.8	Surcharge Over Heel	=			
Surcharge Over Toe	=					Adjacent Footing Load	i=1			
Adjacent Footing Load	=					Axial Dead Load on Stem	=	600.0	5.08	3,050.0
Added Lateral Load	=					* Axial Live Load on Stem	=	2,000.0	5.08	10,166.7
Load @ Stem Above Soil	=					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
	_	Andrew Company	<del></del>	-		Earth @ Stem Transitions	=			
Total	=	3,311.1	O.T.M.	=	12,209.8	Footing Weight	=	975.0	3.25	3,168.8
Resisting/Overturning	Ratio		=		2.73	Key Weight	=	150.0	5.50	825.0
Vertical Loads used	for S	oil Pressure	= 8	913.4	lbs	Vert. Component	=	1,777.4	6.50	11,553.2
Vertical component of activ	ve pre	ssure used f	for soil pre	ssure		Tota	al =	6,913.4 I	bs R.M. =	33,371.1

<sup>\*</sup> Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.





0.00 in

VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID:

File = d:\ENERCALC Projects\2017-0610.ec6

244

2017-0610

Printed: 11 JUL 2017, 3:32PM

### **Cantilevered Retaining Wall**

Lic. # : KW-06002886

Retained Height

Slope Behind Wall

Wall height above soil

Height of Soil over Toe

Water height over heel

Vertical component of active

Lateral soil pressure options:

Description:

Criteria

MAX RETAINING WALL - FELDSPAR UVN

9.00 ft

0.00 ft

0.00:1

30.00 in

0.0 ft

### Soil Data

2,500.0 psf Allow Soil Bearing Equivalent Fluid Pressure Method Heel Active Pressure 40.0 psf/ft 40.0 psf/ft Toe Active Pressure Passive Pressure 360.0 psf/ft 110.00 pcf Soil Density, Heel Soil Density, Toe 110.00 pcf 0.450 Friction Coeff btwn Ftg & Soil = Soil height to ignore

Calculations per ACI 318-14, ACI 530-11, IBC 2015, CBC 2016, ASCE 7-10

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### Surcharge Loads

Surcharge Over Heel = 100.0 psf Used To Resist Sliding & Overturning Surcharge Over Toe = 0.0 psf NOT Used for Sliding & Overturning

USED for Soil Pressure.

USED for Sliding Resistance.

USED for Overturning Resistance.

### **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 1.73 OK Sliding 4,275 lbs 14.35 in Total Bearing Load ...resultant ecc. Soil Pressure @ Toe 2,185 psf OK Soil Pressure @ Heel = 0 psf OK 2,500 psf Allowable Soil Pressure Less Than Allowable ACI Factored @ Toe ACI Factored @ Heel 2,662 psf 0 psf  $\equiv$ 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 1.822.5 lbs less 50 % Passive Force less 100% Friction Force 1,830.0 lbs = 0.0 lbs OK Added Force Reg'd ....for 1.5 : 1 Stability 0.0 lbs OK L

oad Factors —	
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

### Lateral Load Applied to Stem

for passive pressure

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

0.0 lbs Adjacent Footing Load = Footing Width 0.00 ft 0.00 in Eccentricity = Wall to Ftg CL Dist 0.00 ft = Footing Type Line Load Base Above/Below Soil 0.0 ft = at Back of Wall 0.300 Poisson's Ratio =

Stem Construction	- 1	Top Stem	2nd	
Design Height Above Ftg	ft =	Stem OK 4.00	Stem OK 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 —			2000	
fb/FB + fa/Fa	=	0.254	0.655	
Total Force @ Section	lbs =	1,090.9	2,915.6	
MomentActual	ft-I =	2,060.6	9,965.7	
MomentAllowable	ft-I =	8,121.3	15,222.0	
ShearActual	psi =	14.7	39.3	
ShearAllowable	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	A100.1 217	1100 5000 500 400 5	WW 800 54 54 5	
f'c	psi =	2,500.0	2,500.0	
Fy	psi =	60,000.0	60,000.0	



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 11 JUL 2017, 3:32PM

### Cantilevered Retaining Wall

Lic. #: KW-06002886

Description:

MAX RETAINING WALL - FELDSPAR UVN

File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Licensee: RICHMOND HOFFMAYER

### **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 150.00 pcf
Footing Concrete Density	=	
Min. As %	=	0.0018
Cover @ Top 2.00	@ Bt	m.= 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	2
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

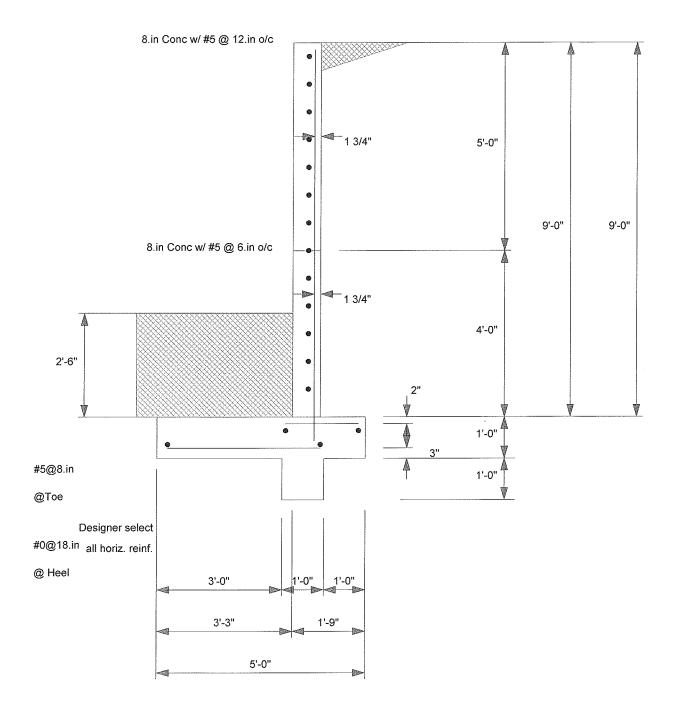
Other Acceptable Sizes & Spacings

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

		0	VERTURN	ING				R	ESISTING	
Item		Force lbs	Distanc ft	е	Moment ft-lb	_		Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	2,000.0	3.33	3	6,666.7	Soil Over Heel	=	1,072.5	4.46	4,781.6
Surcharge over Heel	=	363.6	5.00	)	1,818.2	Sloped Soil Over Heel	=			
Toe Active Pressure	=	-245.0	1.17	7	-285.8	Surcharge Over Heel	=	108.3	4.46	483.0
Surcharge Over Toe	=					Adjacent Footing Load	=			
Adjacent Footing Load	$\dot{x}=\dot{x}$					Axial Dead Load on Stem	=	200.0	3.58	716.7
Added Lateral Load	=					* Axial Live Load on Stem	=	200.0	3.58	716.7
Load @ Stem Above Soil	=					Soil Over Toe	=	893.8	1.63	1,452.3
						Surcharge Over Toe	=			
						Stem Weight(s)	=	900.0	3.58	3,225.0
	_		7	-		Earth @ Stem Transitions	=			
Total	=	2,118.6	O.T.M.	=	8,199.0	Footing Weight	i=1	750.0	2.50	1,875.0
Resisting/Overturning	Ratio		=		1.59	Key Weight	=	150.0	3.50	525.0
Vertical Loads used		oil Pressure	= 4	274.6	6 lbs	Vert. Component	=	200000	5.00	
						Tota	al =	4,074.6	lbs R.M.=	13,058.6

<sup>\*</sup> Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.





0.00 in

VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID:

File = d:\ENERCALC Projects\2017-0610.ec6

017-0610

Printed: 11 JUL 2017; 3:31PM

### **Cantilevered Retaining Wall**

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Lic. #: KW-06002886

Description:

Retained Height

Slope Behind Wall

Wall height above soil

Height of Soil over Toe

Water height over heel

Vertical component of active

Lateral soil pressure options:

Criteria

SIDE RETAINING WALLS - FELDSPAR & QUARTZ LVN & UVN

7.50 ft

1.00 ft

0.00:1

30.00 in

0.0 ft

### 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

Calculations per ACI 318-14, ACI 530-11, IBC 2015, CBC 2016, ASCE 7-10

ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29 Liteensee: RICHIVIOND HOFFMAYER

### 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

USED for Soil Pressure.

USED for Sliding Resistance.

USED for Overturning Resistance.

### 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**

Dead Load

Live Load Earth, H

Wind, W

Seismic, E

Wall Stability Ratios Overturning	=	2.06 OK
100 (0 d d d d d d d d d d d d d d d d d	1777	2.06 OK
Sliding	=	2.21 OK
Total Bearing Load	=	5,456 lbs
resultant ecc.	=	5,456 lbs 3.34 in
Soil Pressure @ Toe	=	2,102 psf OK
Soil Pressure @ Heel	=	808 psf OK
Allowable	=	2,500 psf
Soil Pressure Less	Than A	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 C	ompon	ent 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 —		

1.200

1.600

1.600 1.600

1.000

### Lateral Load Applied to Stem

for passive pressure

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

0.0 lbs Adjacent Footing Load 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 0.0 ft at Back of Wall 0.300 Poisson's Ratio =

tem Construction		Top Stem	2nd	
Design Height Above Ftg	ft =	Stem OK 4.00	Stem OK 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	
MomentActual	ft-I =	457.3	4,333.3	
MomentAllowable	ft-I =	8,121.3	8,121.3	
ShearActual	psi =	5.3	21.5	
ShearAllowable	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	317	0504.0810.4.070		
fc	psi =	2,500.0	2,500.0	
Fy	psi =	60,000.0	60,000.0	



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

Project ID:

File = d:\ENERCALC Projects\2017-0610.ec6

Licensee: RICHMOND HOFFMAYER

ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

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## Cantilevered Retaining Wall

**Footing Dimensions & Strengths** 

2,500 psi

Lic. #: KW-06002886

Total Footing Width

Key Distance from Toe

Footing Concrete Density

Footing Thickness

Description:

Toe Width

Heel Width

Key Width

Key Depth

Min. As %

Cover @ Top

SIDE RETAINING WALLS - FELDSPAR & QUARTZ LVN & UVN

**Footing Design Results** 

Actual 1-Way Shear

2.00 ft
1.75
3.75
12.00 in
12.00 in
0.00 in
1.75 ft
60,000 psi

1.75 60,000 150.00 pcf 0.0018

2.00 @ Btm.= 3.00 in

		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

24.19

Allow 1-Way Shear 75.00 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

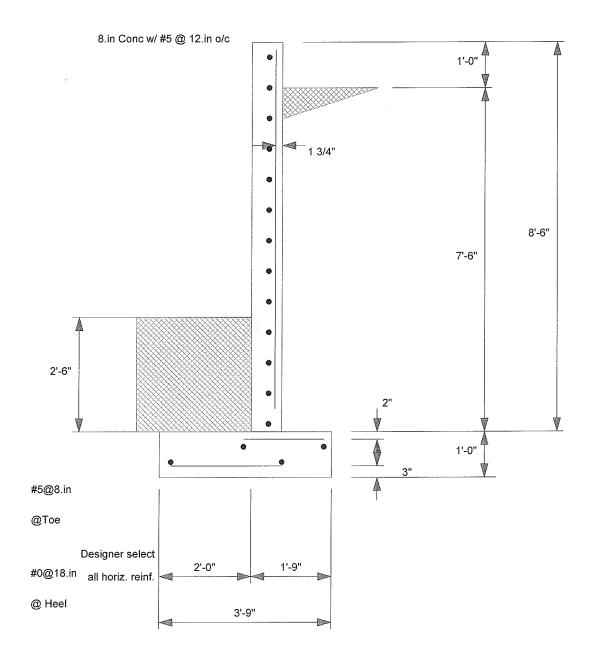
1.61 psi

75.00 psi

### Summary of Overturning & Resisting Forces & Moments

		0	VERTURNIN	G				Е	RESIS	STING	70.01 N
Item		Force lbs	Distance ft	20000	<b>oment</b> t-lb	_		Force lbs		Distance ft	Moment ft-lb
Heel Active Pressure	=	1,445.0	2.83		4,094.2	Soil Over Heel	=	893.8		3.21	2,867.4
Surcharge over Heel	=				***************************************	Sloped Soil Over Heel	=				
Toe Active Pressure	=	-245.0	1.17		-285.8	Surcharge Over Heel	=				
Surcharge Over Toe	=					Adjacent Footing Load	$t=t_{0}$				
Adjacent Footing Load	=					Axial Dead Load on Stem	=	600.0		2.33	1,400.0
Added Lateral Load	=					* Axial Live Load on Stem	=	2,000.0		2.33	4,666.7
Load @ Stem Above Soil	=					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	=				
Total	=	1,200.0	O.T.M.	=	3,808.3	Footing Weight	=	562.5		1.88	1,054.7
Resisting/Overturning	Ratio		=	2.06	ĺ	Key Weight	=			2.25	
Vertical Loads used		oil Pressure	= 5,45	6.3 lb	S	Vert. Component	=			3.75	
						Tota	al =	3,456.3	lbs	R.M. =	7,855.5

<sup>\*</sup> Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.





VILLAGE NEST

MULTI UNIT PROJECT

Project ID: 2017-0610

7-0010

Printed: 11 JUL 2017, 3:30PM

## **Cantilevered Retaining Wall**

Lic. # : KW-06002886

Description:

BOTTOM STEM WALLS - FELDSPAR & QUARTZ LVN & UVN

File = d:\ENERCALC Projects\2017-0610.ec6
ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29
Ligensee: RICHMOND HOFFWAYER
ARTZ LVN & UVN

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 act Lateral soil pressure option USED for Soil I USED for Slidin USED for Over	ons: Pressu ng Res	istance.

Surcharge Loads		
Surcharge Over Heel Used To Resist Sliding	= & Ovo	40.0 psf
Surcharge Over Toe	=	0.0 psf

#### Surcharge Over Toe = 0 NOT Used for Sliding & Overturning 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	=	0.00 OV
Overturning		2.88 OK 6.11 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 Soil Pressure Less	= Then /	2,500 psf
	Than A	A 1 ( A 1( A)( A 1( A 1
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 C	ompon	ent Used)
Lateral Sliding Force	=	327.7 lbs
less 50 % Passive Force	= .	1,102.5 lbs
less 100% Friction Force	= .	900.Q lbs
Added Force Reg'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

Soil Data			
Allow Soil Bearing	=	2,500.0 p	sf
Equivalent Fluid Pressure Me	thod	d	
Heel Active Pressure	=	40.0 p	sf/ft
Toe Active Pressure	=	40.0 p	sf/ft
Passive Pressure	=	360.0 p	sf/ft
Soil Density, Heel	=	110.00 p	cf
Soil Density, Toe	=	110.00 p	cf
Friction Coeff btwn Ftg & Soil	=	0.450	
Soil height to ignore for passive pressure	=	0.00 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

## Calculations per ACI 318-14, ACI 530-11, IBC 2015, CBC 2016, ASCE 7-10

Adjacent Footing Load		
Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	i = 1	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

Stem Construction		Top Stem	
Design Height Above Ftg Wall Material Above "Ht"	ft = =	Stem OK 0.00 Concrete	
Thickness	in =	8.00	
Rebar Size	=	# 5	
Rebar Spacing	in =	12.00	
Rebar Placed at  Design Data	=	Edge	
fb/FB + fa/Fa	=	0.086	
Total Force @ Section	lbs =	405.1	
MomentActual	ft-l =	702.2	
MomentAllowable	ft-I =	8,121.3	
ShearActual	psi =	5.5	
ShearAllowable	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			
fc	psi =	2,500.0	
Fy	psi =		



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

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## Cantilevered Retaining Wall

Lic. #: KW-06002886

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BOTTOM STEM WALLS - FELDSPAR & QUARTZ LVN & UVN Description:

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	
fc = 2,500 psi Footing Concrete Density	Fy_=	60,000 psi 150.00 pcf	
Min. As %	=	0.0018	
Cover @ Top 2.00	@ Bt	tm.= 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	

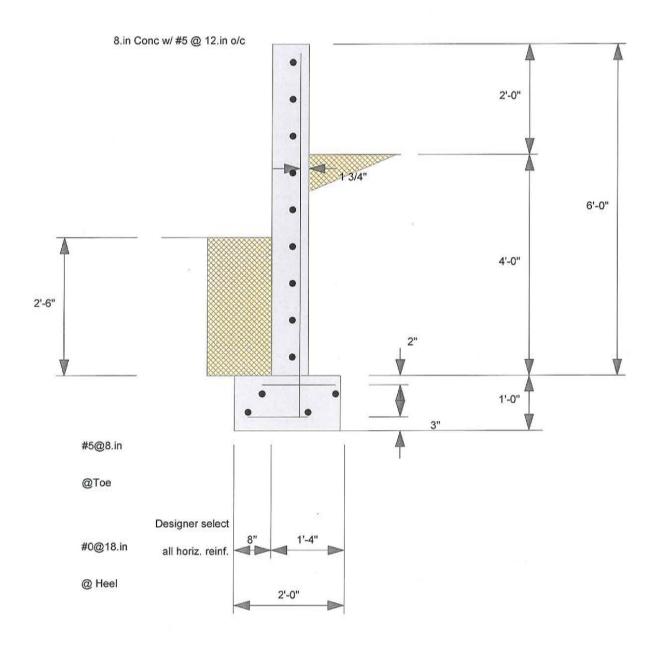
Other Acceptable Sizes & Spacings

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

		0	VERTURN	ING						RESI	STING	
Item		Force lbs	Distanc ft		Mo	<b>ment</b> Ib	_		Force lbs		Distance ft	Moment ft-lb
Heel Active Pressure	=	500.0	1.67	7		833.3	Soil Over Heel	=	291.9		1.67	486.9
Surcharge over Heel	=	72.7	2.50	)		181.8	Sloped Soil Over Heel	=				
Toe Active Pressure	=	-245.0	1.17	7		-285.8	Surcharge Over Heel	=	26.5		1.67	44.3
Surcharge Over Toe	=						Adjacent Footing Load	i=0				
Adjacent Footing Load	=						Axial Dead Load on Stem	=	600.0		1.00	602.0
Added Lateral Load	=						* Axial Live Load on Stem	=	1,000.0		1.00	1,003.3
Load @ Stem Above Soil	=						Soil Over Toe	=	184.3		0.34	61.7
							Surcharge Over Toe	=				
							Stem Weight(s)	=	600.0		1.00	602.0
	_		-	-			Earth @ Stem Transitions	=				
Total	=	327.7	O.T.M.	=		729.3	Footing Weight	=	300.0		1.00	300.0
Resisting/Overturning	Ratio		=		2.88		Key Weight	=			2.25	
Vertical Loads used			= 3,	002	2.7 lbs		Vert. Component	=			2.00	
							Tota	1 =	2 002 7	lhs	RM =	2 096 9

\* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.





VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

7+3

Printed: 11 JUL 2017, 3:30PM

## **General Footing**

Lic. # : KW-06002886

File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Licensee: RICHMOND HOFFMAYER

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	=	6	0.0 ksi
Éc : Concrete Elastic Modulus	=	3,12	2.0 ksi
Concrete Density	=	14	5.0 pcf
φ Values Flexure	=		.90
Shear	=	0.8	350
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 & sh	ears	:	No
Add Pedestal Wt for Soil Pressure		:	No
Use Pedestal wt for stability, mom & s	shear	:	No
Dimensions			

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

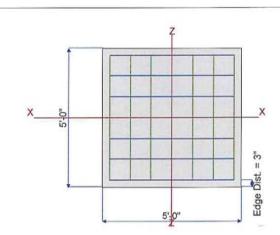
## Increases based on footing Depth Footing base depth below soil surface = 2.0 ft Allow press. increase per foot of depth = ksf when footing base is below = ft

Allowable pressure increase per foot of depth		
when max. length or width is greater than	=	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





Increases based on footing plan dimension

#### Reinforcing

Pedestal dimensions... px : parallel to X-X Axis

pz : parallel to Z-Z Axis Height

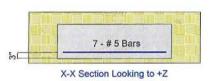
at Bottom of footing

Rebar Centerline to Edge of Concrete...

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



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	Е	Н
P : Column Load OB : Overburden	=	3.807	4.015		49.381			k ksf
	_							k-ft
M-xx M-zz	=							k-ft
V-x	=							K
V-Z	=				4			K



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

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**General Footing** 

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Lic. #: KW-06002886

Description: MAX INTERIOR PAD FOOTING: 4" P.C.

	Min. Ratio	Item	Applied	Capacity	<b>Governing Load Combination</b>
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

Rotation Axis &		Xecc	Zecc	Actual Soil Bearing Stress @ Location				Actual / Allow
Load Combination	Gross Allowable	(ir	n)	Bottom, -Z	Top, +Z	Left, -X	Right, +X	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								

Footing Flexure					100 - 2			
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



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

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**General Footing** 

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Lic. # : KW-06002886 Description:

MAX INTERIOR PAD FOOTING: 4" P.C.

Fo	otin	a F	lexu	re
	Oth	200	CAG	

Footing Flexure									
Flexure Axis & Load Combination	Mu k-ft	Side	Tensio Surfac		Gvrn. As in^2	Actual A	As P	hi*Mn k-ft	Status
Z-Z, +1.20D+0.20S Z-Z, +1.20D+0.20S Z-Z, +0.90D Z-Z, +0.90D One Way Shear	1.806 1.806 0.4283 0.4283	-X +X -X +X	Bottom Bottom Bottom Bottom	0.3888 0.3888	Min Temp Min Temp	% 0.4 % 0.4	1340 1340 1340 1340	28.298 28.298 28.298 28.298	OK OK OK
Load Combination	Vu @ -X	Vu @ ·	+X \	√u@-Z Vu	@ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D +1.20D+0.50Lr +1.20D+0.50S +1.20D+1.60Lr +1.20D+1.60S +1.20D+0.20S +0.90D Two-Way "Punching" Shear	1.48 ps 1.83 ps 8.13 ps 3.05 ps 23.22 ps 4.01 ps 0.95 ps	i i i i	1.48 psi 1.83 psi 8.13 psi 3.05 psi 23.22 psi 4.01 psi 0.95 psi	1.48 psi 1.83 psi 8.13 psi 3.05 psi 23.22 psi 4.01 psi 0.95 psi	1.48 psi 1.83 psi 8.13 psi 3.05 psi 23.22 psi 4.01 psi 0.95 psi	1.48 psi 1.83 psi 8.13 psi 3.05 psi 23.22 psi 4.01 psi 0.95 psi	85.00 ps 85.00 ps 85.00 ps 85.00 ps 85.00 ps 85.00 ps	si 0.02 si 0.10 si 0.04 si 0.27 si 0.05 si 0.01 All units	AND DESCRIPTION
Load Combination		Vu		Phi*Vn		Vu / Phi*Vn			Status
+1.40D +1.20D+0.50Lr +1.20D+0.50S +1.20D+1.60Lr +1.20D+1.60S +1.20D+0.20S +0.90D		6.79 30.20 11.35 86.26 14.9	5 psi 6 psi	170.00 170.00 170.00 170.00 170.00 170.00 170.00 170.00	osi osi osi osi	0.03236 0.03992 0.1776 0.06674 0.5074 0.0877 0.0208			OK OK OK OK OK OK



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

256

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**General Footing** 

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Licensee: RICHMOND HOFFMAYER

Description : INTE

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	2.50 ksi		
fy: Rebar Yield	=	60.0 ksi			
Éc : Concrete Elastic Modulus	=	3,12	2.0 ksi		
Concrete Density	=	14	5.0 pcf		
φ Values Flexure	=	0	.90		
Shear	=	0.8	850		
Analysis Settings	W.				
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		1	No		
Use ftg wt for stability, moments &	shears	:	No		
Add Pedestal Wt for Soil Pressure	L	9	No		
Use Pedestal wt for stability, mom	& shear	1	No		
Dimensions					

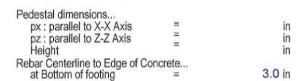
Soil Design Values		
Allowable Soil Bearing	=	2.50 ksf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	180.0 pcf
Soil/Concrete Friction Coeff.	=	0.450

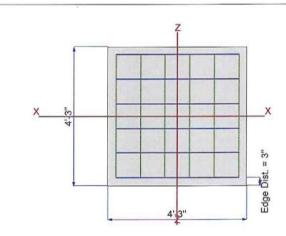
# Increases based on footing Depth Footing base depth below soil surface = 2.0 ft Allow press. increase per foot of depth = ksf when footing base is below = ft

## Increases based on footing plan dimension Allowable pressure increase per foot of depth when max. length or width is greater than = ksf

#### Dimensions

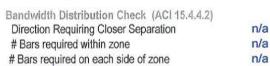
Width parallel to X-X Axis Length parallel to Z-Z Axis Footing Thickness	=	4.250 ft			
Length parallel to Z-Z Axis	=	4.250 ft			
Footing Thickness	=	18.0 in			

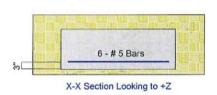




#### Reinforcing

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







**Applied Loads** 

		D	Lr	L	S	W	E	н
P : Column Load OB : Overburden	= -	2.843	2.906	0.280	36.394			k ksf
M-xx M-zz	=							k-ft k-ft
V-x	=							k
V-z	=							k



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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**General Footing** 

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|Lic. # : KW-06002886 | Description : INTERIOR PAD FOOTING: FELDSPAR LVN 4" P.C. UNDER B17

	Min. Ratio	Item	v v	Applied		Capacity	Governir	ng Load Combina	tion
PASS	0.8908	Soil Bearing		2.227 ksf		2.50 ksf	0.000.000.000	bout Z-Z axis	
PASS	n/a	Overturning - X-X		0.0 k-ft		0.0 k-ft	No Ove		
PASS	n/a	Overturning - Z-Z		0.0 k-ft		0.0 k-ft	No Ove	\$107 Y X X X X X X X X X X X X X X X X X X	
PASS	n/a	Sliding - X-X		0.0 k-n		0.0 k	No Slidi	•	
PASS	n/a	Sliding - Z-Z		0.0 k		0.0 k	No Slidi		
PASS	200.00					0.0 k			
	n/a	Uplift		0.0 k			No Uplit		
PASS	0.2707	Z Flexure (+X)		7.723 k-ft/ft		28.527 k-ft/ft		+0.50L+1.60S	
PASS	0.2707	Z Flexure (-X)		7.723 k-ft/ft		28.527 k-ft/ft		+0.50L+1.60S	
PASS	0.2707	X Flexure (+Z)		7.723 k-ft/ft		28.527 k-ft/ft		+0.50L+1.60S	
PASS	0.2707	X Flexure (-Z)		7.723 k-ft/ft		28.527 k-ft/ft	7,777	+0.50L+1.60S	
PASS	0.190	1-way Shear (+X)		16.152 psi		85.0 psi		+0.50L+1.60S	
PASS	0.190	1-way Shear (-X)		16.152 psi		85.0 psi		+0.50L+1.60S	
PASS	0.190	1-way Shear (+Z)	9	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 Res	sults					Machine agrees			
Soil Bearing			101	-10-					
Rotation Axis Load Cor	& nbination	Gross Allowable	Xecc	Zecc (in)	Actus Bottom, -Z	al Soil Bearing Stre Top, +Z	ss @ Loca Left, -X	tion 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		0.2279	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	N 12 12 12 12 12 12 12 12 12 12 12 12 12	2.50	n/a	0.0	2.227	2.227	n/a	n/a	0.891
X-X, +D+0.750		2.50	n/a	0.0	0.3447	0.3447	n/a	n/a	0.138
X-X. +D+0.750	L+0.750S	2.50	n/a	0.0	1.735	1.735	n/a	n/a n/a	0.694 0.051
X-X, +0.60D Z-Z, D Only		2.50 2.50	n/a 0.0	0.0 n/a	0.1274 n/a	0.1274 n/a	n/a 0.2124	0.2124	0.085
Z-Z, D OHIV Z-Z, +D+L		2.50	0.0	n/a	n/a	n/a	0.2124	0.2279	0.003
Z-Z. +D+Lr		2.50	0.0	n/a	n/a	n/a	0.3733	0.3733	0.149
7.7 +D+S		2.50	0.0		n/a	n/a	2 227	2 227	0.891

X-X, +0.60D	2.50		n/a	0.0	0.1274	0.1274	n/a	n/a	0.057		
Z-Z, D Only	2.50		0.0	n/a	n/a	ı n/a	0.2124	0.2124	0.085		
Z-Z. +D+L	2.50		0.0	n/a	n/a	ı 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	ı n/a	2.227	2.227	0.891
Z-Z, +D+0.750Lr+0.750L	2.50		0.0	n/a	n/a	ı n/a	0.3447	0.3447	0.138		
Z-Z, +D+0.750L+0.750S	2.50		0.0	n/a	n/a	ı n/a	1.735	1.735	0.694		
Z-Z, +0.60D	2.50	)	0.0	n/a	n/a		0.1274	0.1274	0.051		
Footing Flexure								DESMINATE VI	URDAY-001		
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	i	0.3888	Min Temp %	0.4376	28.527	OK		
X-X, +1,40D	0.4975	-Z	Bottom	i	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, +1,40D	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



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

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**General Footing** 

File = d:\ENERCALC Projects\2017-\0610.ec6 ENERCALC, INC. 1983-2017, Build:\tau.17.6.29, Ver:\tau.17.6.29 Licensee: RICHIMOND HOFFMAYER

Lic. # : KW-06002886 Description : INTERIO

INTERIOR PAD FOOTING: FELDSPAR LVN 4" P.C. UNDER B17

Fo	oti	na	FI	ex	ure
	vu	ш		CA	ui c

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	ОК
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								

### One way Snear

Load Combination	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	1.04 ps	i 1.04 psi	1.04 psi	1.04 psi	1.04 psi	85.00 ps	i 0.01	0.00
+1.20D+0.50Lr+1.60L	1.39 ps	i 1.39 psi	1.39 psi	1.39 psi	1.39 psi	85.00 ps	i 0.02	0.00
+1.20D+1.60L+0.50S	5.77 ps	i 5.77 psi	5.77 psi	5.77 psi	5.77 psi	85.00 ps	i 0.07	0.00
+1.20D+1.60Lr+0.50L	2.14 ps	i 2.14 psi	2.14 psi	2.14 psi	2.14 psi	85.00 ps	i 0.03	0.00
+1.20D+1.60Lr	2.11 ps	i 2.11 psi	2.11 psi	2.11 psi	2.11 psi	85.00 ps	i 0.02	0.00
+1.20D+0.50L+1.60S	16.15 ps	i 16.15 psi	16.15 psi	16.15 psi	16.15 psi	85.00 ps	i 0.19	0.00
+1.20D+1.60S	16.12 ps	i 16.12 psi	16.12 psi	16.12 psi	16.12 psi	85.00 ps	i 0.19	0.00
+1.20D+0.50Lr+0.50L	1.31 ps	i 1.31 psi	1.31 psi	1.31 psi	1.31 psi	85.00 ps	i 0.02	0.00
+1.20D+0.50L+0.50S	5.69 psi	i 5.69 psi	5.69 psi	5.69 psi	5.69 psi	85.00 ps	i 0.07	0.00
+1.20D+0.50L+0.20S	2.83 psi	i 2.83 psi	2.83 psi	2.83 psi	2.83 psi	85.00 ps	i 0.03	0.00
+0.90D	0.67 psi	i 0.67 psi	0.67 psi	0.67 psi	0.67 psi	85.00 ps	i 0.01	0.00
Two-Way "Punching" Shear	/##### /PPAD						All units	k

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 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



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

259

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**General Footing** 

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Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886 Description: INTER

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 Pro	perties		11/2	sammerou v	Soil D
f'c : Concret	e 28 day strength	=	2	2.50 ksi	Allow
fy: Rebar Y		=	6	0.0 ksi	Incre
	te Elastic Modulus	=	3,12	22.0 ksi	Soil I
Concrete De	ensity	=	14	15.0 pcf	Soil/0
φ Values	Flexure	=	C	0.90	
15	Shear	=	0.	850	Increas
Analysis Se	ttings				Footi
Min Steel %	Bending Reinf.		=		Allow
	Temp Reinf.		=	0.00180	W
Min. Overtu	rning Safety Factor		=	1.50 : 1	
	Safety Factor		=	1.50:1	Increas
	for Soil Pressure		:	No	Allow
Use ftg wt fo	or stability, moments & shears		:	No	
Add Pedest	al Wt for Soil Pressure		:	No	when
Use Pedest	al wt for stability, mom & shear		:	No	
Dimension	•				

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

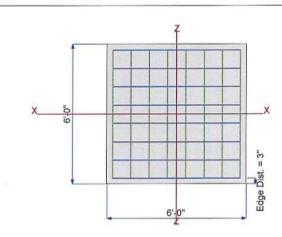
## roreases based on footing Depth Footing base depth below soil surface = 2.0 ft Allow press. increase per foot of depth = ksf when footing base is below = ft

creases based on footing plan dimension Allowable pressure increase per foot of depth	i	
when max. length or width is greater than	=	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

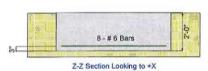




#### Reinforcing

Bars parallel to X-X Axis	-		12 121
Number of Bars	_	200	8.0
Reinforcing Bar Size	=	#	6
Bars parallel to Z-Z Axis			
Number of Bars	=		8.0
Reinforcing Bar Size	=	#	6





Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation

# Bars required within zone

# Bars required on each side of zone

n/a

**Applied Loads** 

		D	Lr	L	S	W	Е	Н
P : Column Load OB : Overburden	= =	6.880	1.120	7.625	60.0			k ksf
M-xx M-zz	=							k-ft k-ft
V-x	=							k
V-z	=							k



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

017-0610

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General Footing

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Description:

INTERIOR PAD FOOTING: QUARTZ LVN 4" P.C. UNDER B16 - NOT AT RIDGE

	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

Rotation Axis &		Xecc	Zecc	Actua	I Soil Bearing S	tress @ Locat	ion	Actual / Allow
Load Combination	Gross Allowable	1,1283	(in)	Bottom, -Z	Top, +Z	Left, -X	Right, +X	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	,511.1	7.6	577770	20070				

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	-7	Bottom	0.5184	Min Temp %	0.5867	53.618	OK
X-X. +1.20D+0.50L+0.50S	5.259	-Z +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, +1.20D 10.50E 10.200 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



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID:

2017-0610

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All units k

## **General Footing**

Two-Way "Punching" Shear

Lic. # : KW-06002886

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INTERIOR PAD FOOTING: QUARTZ LVN 4" P.C. UNDER B16 - NOT AT RIDGE Description:

Footing		

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	3.1110			0.0101	THE LANGE OF	210001	59/715	2000

n Status
.02 0.00
.04 0.00
.09 0.00
.02 0.00
.02 0.00
.18 0.00
.18 0.00
.02 0.00
.07 0.00
.04 0.00
.01 0.00
). ). ). ).

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	170.00 psi	0.06377	OK
+1.20D+1.60L+0.50S	26.03 psi	170.00 psi	0.1531	OK
+1.20D+1.60Lr+0.50L	7.15 psi	170.00 psi	0.04206	OK
+1.20D+1.60Lr	5.18 psi	170.00 psi	0.03049	OK OK OK OK OK OK OK OK OK
+1.20D+0.50L+1.60S	55.75 psi	170.00 psi	0.3279	OK
+1.20D+1.60S	53.78 psi	170.00 psi	0.3164	OK
+1.20D+0.50Lr+0.50L	6.52 ps	170.00 psi	0.03832	OK
+1.20D+0.50L+0.50S	21.70 ps	170.00 psi	0.1277	OK
+1.20D+0.50L+0.20S	12.42 ps	170,00 psi	0.07304	OK
+0.90D	3.19 psi	170.00 psi	0.01879	OK



VILLAGE NEST

JJH MULTI UNIT PROJECT Project ID: 2017-0610

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2.50 ksf No

180.0 pcf

2.0 ft

ksf ft

ksf ft

0.450

## **General Footing**

Lic. #: KW-06002886

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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**

IV		erial Pro		=	-	2.50 ksi	Soil Design Values Allowable Soil Bearing
			28 day strength			2.50 (1.50 (	
	ty	: Rebar Yi	eld	=		0.0 ksi	Increase Bearing By Footing Weight
	Ec	: Concrete	e Elastic Modulus	=		22.0 ksi	Soil Passive Resistance (for Sliding)
		oncrete De		=	14	15.0 pcf	Soil/Concrete Friction Coeff.
	φ	Values	Flexure	=	C	0.90	
	7		Shear	=	0.	850	Increases based on footing Depth
A	nal	vsis Set	ttings				Footing base depth below soil surface
			Bending Reinf.		=		Allow press. increase per foot of depth
			Temp Reinf.		=	0.00180	when footing base is below
	Mi	n. Overturi	ning Safety Factor		=	1.50 : 1	
	Mi	n. Sliding	Safety Factor		=	1.50 ; 1	Increases based on footing plan dimension
			or Soil Pressure		1	No	Allowable pressure increase per foot of depth
	Us	e ftg wt for	r stability, moments & shears		1	No	when you lead the sould the secretar these
			l Wt for Soil Pressure		0	No	when max. length or width is greater than
	Us	e Pedesta	l wt for stability, mom & shear		:	No	
	CARON	the comments and					

#### **Dimensions**

Width parallel to X-X Axis	=	6.250 ft
Length parallel to Z-Z Axis	= 1	6.250 ft
Footing Thickness	=	24.0 in

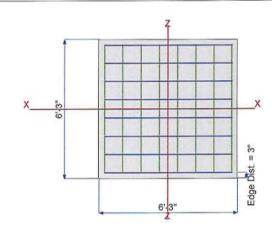


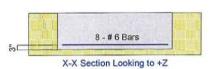
#### Reinforcing

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

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**

A CONTRACTOR OF THE PARTY OF TH		D	Lr	L	S	W	Е	Н
P : Column Load OB : Overburden	= -	9.0	1.0	9.0	80.0			k ksf
M-xx M-zz	=		4					k-ft k-ft
V-x	=							k
V-z	=							K



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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**General Footing** 

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Lic. #: KW-06002886

Description: INTERIOR PAD FOOTING: QUARTZ LVN 4" P.C. UNDER B16 - UNDER RIDGE

	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
etailed Res	sults	S TREE OF WORKING ON THE	instantional Mass	STATE STATE	
il Bearing			WH Sec	7.767 70.761 50000 100 100	
ALL STATE OF THE STATE OF	144		Vana 7000	Actual Cail Dearing Ctro	on @ Location Aut

Soil Bearing		V	-		. O. II B			
Rotation Axis & Load Combination	Gross Allowable	Xecc (	Zecc in)	Bottom, -Z	I Soil Bearing S Top, +Z	Left, -X	Right, +X	Actual / Allov 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		7.170	7.003%			NEW (NEW YORK)	184000000	200000

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 -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



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

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**General Footing** 

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Lic. #: KW-06002886 Description:

INTERIOR PAD FOOTING: QUARTZ LVN 4" P.C. UNDER B16 - UNDER RIDGE

**Footing Flexure** 

Flexure Axis & Load Combination	<b>Mu</b> k-ft	Side	Tensio Surfac		'd	Gvrn. As in^2	Actua in^2		Phi* k-i		Status
Z-Z. +1.40D	1.575	-X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z, +1.40D	1.575	+X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z, +1.20D+0.50Lr+1.60L	3.213	+X -X +X -X +X -X +X -X +X -X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z, +1.20D+0.50Lr+1.60L	3.213	+X	Bottom		184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z. +1.20D+1.60L+0.50S	8.150	-X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z, +1.20D+1.60L+0.50S	8.150	+X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z, +1.20D+1.60Lr+0.50L	2.113	-X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z, +1.20D+1.60Lr+0.50L	2.113	+X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z. +1.20D+1.60Lr	1.550	-X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z, +1.20D+1.60Lr	1.550	+X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z, +1.20D+0.50L+1.60S	17.913	-X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z, +1.20D+0.50L+1.60S	17.913	+X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z, +1.20D+1.60S	17.350	-X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z, +1.20D+1.60S	17.350	+X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z, +1.20D+0.50Lr+0.50L	1.975	-X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z, +1.20D+0.50Lr+0.50L	1.975	+X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z, +1.20D+0.50L+0.50S	6.913	-X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z, +1.20D+0.50L+0.50S	6.913	+X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z. +1.20D+0.50L+0.20S	3.913	-X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z. +1.20D+0.50L+0.20S	3.913	+X	Bottom	0.5	184	Min Temp %	6 (	.5632	5	1.543	OK
Z-Z. +0.90D	1.013	-X	Bottom		184	Min Temp %		.5632	5	1.543	OK
Z-Z, +0.90D	1.013	+X	Bottom	0.5	184	Min Temp %	6 0	.5632	5	1.543	OK
One Way Shear			U BOMO BEENIN	CONTRACT	1.044	NO11/200. 02/05/12/02. 33	A)( Y)(X)	10.000.000.000.00	333	A DAIRESCAL	2000-01
Load Combination	Vu @ -X	Vu@			Vu @		/u:Max	Phi Vr		u / Phi*Vn	Status
+1.40D	1.73 psi		1.73 psi	1.73 psi		1.73 psi	1.73 ps		35.00 psi	0.02	0.0
+1.20D+0.50Lr+1.60L	3.54 psi		3.54 psi	3.54 psi		3.54 psi	3.54 psi		isq 00.28	0.04	0.0
+1.20D+1.60L+0.50S	8.97 psi		8.97 psi	8.97 psi		8.97 psi	8.97 psi		35.00 psi	0.11	0.0
+1.20D+1.60Lr+0.50L	2.33 psi		2.33 psi	2.33 psi		2.33 psi	2.33 psi		35.00 psi	0.03	0.0
+1.20D+1.60Lr	1.71 psi		1.71 psi	1.71 psi		1.71 psi	1.71 psi		35.00 psi	0.02	0.0
+1.20D+0.50L+1.60S	19.71 psi		19.71 psi	19.71 psi		19.71 psi	19.71 psi	8	35.00 psi	0.23	0.0
+1.20D+1.60S	19.09 psi		19.09 psi	19.09 psi		19.09 psi	19.09 psi	8	35.00 psi	0.22	0.0
+1.20D+0.50Lr+0.50L	2.17 psi		2.17 psi	2.17 psi		2.17 psi	2.17 psi		35.00 psi	0.03	0.0
+1.20D+0.50L+0.50S	7.61 psi		7.61 psi	7.61 psi		7.61 psi	7.61 psi		35.00 psi	0.09	0.0
+1.20D+0.50L+0.20S	4.31 psi		4.31 psi	4.31 psi		4.31 psi	4.31 psi		35.00 psi	0.05	0.0
+0.90D	1.11 psi		1.11 psi	1.11 psi		1.11 psi	1.11 psi		35.00 psi	0.01	0.0
Two-Way "Punching" Shear	1.11 030		1.11.001	1.11 001		1.11 001	1.11 00		JO.00 DOI	All units	
Load Combination		Vu		Phi*V	n		Vu / Phi*Vr				Status
+1.40D		6.6	4 psi	170.	00 ps	i	0.03903				OK
+1.20D+0.50Lr+1.60L		13.5	isa <b>8</b>		00 ps		0.07961				OK
+1.20D+1.60L+0.50S		34.3	3 psi	170.	00 ps	i	0.202				OK
+1.20D+1.60Lr+0.50L		8.9	0 psi		00 ps		0.05235				OK
+1.20D+1.60Lr		6.5	3 psi	170.	00 ps	i	0.03841				OK
+1.20D+0.50L+1.60S		75.4	6 psi	170.	00 ps	i	0.4439				OK
+1.20D+1.60S		73.0	9 psi	170.	00 ps	i	0.4299				OK
+1.20D+0.50Lr+0.50L		8.3	2 psi	170.	00ps	i	0.04894				OK
+1.20D+0.50L+0.50S		29.1	2 psi	170.	00 ps	i	0.1713				OK
1.20D 10.00L 10.000			- 10 mm								
+1.20D+0.50L+0.20S		16.4	8 psi 7 psi	170.	00ps 00ps	i	0.09695 0.02509				OK OK



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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### **General Footing**

Lic. #: KW-06002886

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Licensee: RICHMOND HOFFMAYER

Description : MAX EXTER

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**

Mate	erial Pro	perties			
		28 day strength	=	2	2.50 ksi
	: Rebar Yie		=	e	60.0 ksi
Éc	: Concrete	Elastic Modulus	=		22.0 ksi
Co	ncrete Der	nsity	=	14	15.0 pcf
φ	Values	Flexure	=	C	0.90
		Shear	=	0.	850
Anal	ysis Set	tings			
		Bending Reinf.		=	
Mi	n Allow %	Temp Reinf.		=	0.00180
Mi	n. Overturr	ning Safety Factor		=	1.50 :
Mi	n. Sliding S	Safety Factor		=	1.50
		or Soil Pressure		1	No
		stability, moments & shears		1	No
Ad	d Pedestal	Wt for Soil Pressure		:	No
Us	e Pedestal	wt for stability, mom & shear		¥	No
Dim	oncions	The second of th			

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

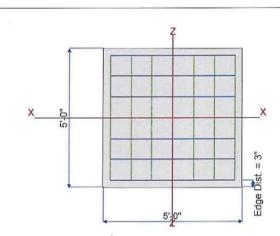
## Increases based on footing Depth Footing base depth below soil surface = 2.0 ft Allow press. increase per foot of depth = ksf when footing base is below = ft

## Increases based on footing plan dimension Allowable pressure increase per foot of depth when max. length or width is greater than ### ksf ### transfer increase per foot of depth ### calculations are allowed by the control of the c

#### **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	1.	
px : parallel to X-X Axis	=	in
pz : parallel to Z-Z Axis	=	in
	=	W(2)
Height		in
Rebar Centerline to Edge of Co	ncrete	25x2x 333
at Bottom of footing	=	3.0 in

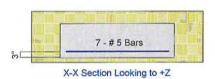


#### Reinforcing

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





**Applied Loads** 

		D	Lr	L	S	W	E	Н
P : Column Load OB : Overburden	= =	3.807	4.015		49.381			k ksf
M-xx M-zz	=							k-ft k-ft
V-x	=							k
V-z	=							k



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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**General Footing** 

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Lic. #: KW-06002886

Description: MAX EXTERIOR PAD FOOTING: 6" HSS

	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
<b>Detailed Res</b>	ults				Martine Martine

Soil Bearing Rotation Axis &		Xecc	Zecc	Actua	Soil Bearing S	Stress @ Loca	tion	Actual / Allow
Load Combination	<b>Gross Allowable</b>	(i		Bottom, -Z	Top, +Z	Left, -X	Right, +X	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								

rooting riexure								
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 -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 -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
Company and the control of the contr								



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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## **General Footing**

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MAX EXTERIOR PAD FOOTING: 6" HSS Description:

Footin	g Fl	lexu	re
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Footing Flexure									
Flexure Axis & Load Combination	<b>Mu</b> k-ft	Side	Tensio Surface		Gvrn. A in^2	As Actual in^2	As	Phi*Mn k-ft	Status
Z-Z, +1.20D+0.20S Z-Z, +1.20D+0.20S Z-Z, +0.90D Z-Z, +0.90D <b>One Way Shear</b>	1.806 1.806 0.4283 0.4283	-X +X -X +X	Bottom Bottom Bottom Bottom	0.388 0.388 0.388 0.388	8 Min Temi 8 Min Temi	0 % 0 0 % 0	.4340 .4340 .4340 .4340	28.298 28.298 28.298 28.298	OK OK OK
Load Combination	Vu @ -X	Vu@	+X \	/u @ -Z Vu	ı @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D +1.20D+0.50Lr +1.20D+0.50S +1.20D+1.60Lr +1.20D+1.60S +1.20D+0.20S +0.90D Two-Way "Punching" Shear	1.48 ps 1.83 ps 8.13 ps 3.05 ps 23.22 ps 4.01 ps 0.95 ps	i i i i	1.48 psi 1.83 psi 8.13 psi 3.05 psi 23.22 psi 4.01 psi 0.95 psi	1.48 psi 1.83 psi 8.13 psi 3.05 psi 23.22 psi 4.01 psi 0.95 psi	1.48 ps 1.83 ps 8.13 ps 3.05 ps 23.22 ps 4.01 ps 0.95 ps	i 1.83 psi i 8.13 psi i 3.05 psi i 23.22 psi i 4.01 psi i 0.95 psi	85.00 85.00 85.00 85.00 85.00 85.00	psi 0.02 psi 0.10 psi 0.04 psi 0.27 psi 0.08	2 0.00 0 0.00 4 0.00 7 0.00 5 0.00 1 0.00 its k
Load Combination		Vu		Phi*Vn		Vu / Phi*Vn			Status
+1.40D +1.20D+0.50Lr +1.20D+0.50S +1.20D+1.60Lr +1.20D+1.60S +1.20D+0.20S +0.90D		6.7 30.2 11.3 86.2 14.9	0 psi 9 psi 0 psi 5 psi 6 psi 1 psi 4 psi	170.00 170.00 170.00 170.00 170.00 170.00 170.00	psi psi psi psi psi	0.03236 0.03992 0.1776 0.06674 0.5074 0.0877 0.0208			OK OK OK OK OK OK



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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Design OK

Steel Beam

Lic. #: KW-06002886

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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** 

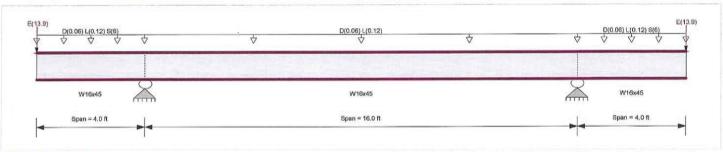
Analysis Method: Allowable Strength Design

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

Bending Axis: Major Axis Bending

Fy : Steel Yield : E: Modulus : 50.0 ksi

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 = 0.020, 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

Maximum Shear Stress Ratio = 0.235:1Maximum Bending Stress Ratio = 0.325:1 Section used for this span Section used for this span W16x45 W16x45 Va : Applied 26.078 k Ma: Applied 66.750 k-ft Vn/Omega: Allowable Mn / Omega: Allowable 111.090 k 205.339 k-ft Load Combination +D+0.750L+0.750S+0.5250E +D+0.750L+0.750S+0.5250E Load Combination 16.000 ft Location of maximum on span Location of maximum on span 16.000ft Span # where maximum occurs Span # where maximum occurs Span #2 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 Combina	ation		Max Stress	Ratios		S	ummary of M	oment Valu	ies			Summary of Shear Values		
Segmen		Span #	М	V	Mmax +	Mmax -	Ма Мах	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.	-1211123-27													
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



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

Steel Beam

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Lic. # : KW-06002886 FELDSPAR UVN LOWER FLOOR - B22 Description:

Load Combin	ation		Max Stre	ss Ratios			Summary of M	noment Valu	ies			Summ	nary of Sh	ear Values
	nt Length	Span#	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsan, 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 =		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	11/2/2012/11/11	577	7/45/7/7/	A140000										
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 =		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 =		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
Dsan, L =		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		DE												
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		E												
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 =		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 =		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 =		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+0.70I	<b>∃</b>													
Dsgn. L =	4.00 ft	1	0.192	0.090		-39.42	39.42	342.92	205.34		1.00	9.98	166.64	111.09
Dsgn. L =		2	0.192	0.090	-0.00	-39.42	39.42	342.92	205.34		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	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	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	Maximu	m Defle	ctions											
Load Comb	ination		Span	Max. "-" Defl	Location	in Span	Load Com	bination			Max	k. "+" Defl	Location	n in Span
+D+0.750	L+0.750S+0.5	250E	1	0.2349		0.000						0.0000		0.000
			2	0.0000		0.000	+D+0.75	50L+0.750S	+0.5250E			-0.2030		8.000
+D+0.750	L+0.750S+0.5	5250E	3	0.2346		4.000						0.0000		8.000
Vertica	I Reactio	ns				Support	notation : Far	left is #1			Values i	n KIPS		
Load Comb	ination	Mark Address V	Support 1	Support 2	Suppor	t3 Sup	port 4							
Overall Ma	Vimum			27 638	27.6	38	ar ne en Aut d'A							

Load Combination	Support 1	Support 2	Support 3
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
17.45 BC177			



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

#### 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
É - Main Rebar	=	29,000.0 ksi
Allow. Reinforcing Limits		ASTM A615 Bars Used
Min. Reinf.	=	1.0 %
Max. Reinf.	=	8.0 %

6.0 ft Overall Column Height Top & Bottom Pinned End Fixity

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

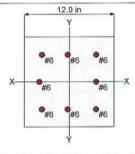
Rebar Edge Cover = 2.0in

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 Combina	ation	+1.20D+0.	50L+1.60S	Maximum SERVICE Load	d Reactions	3	
Location of m	ax.above base		5.960 ft	Top along Y-Y	0.0k	Bottom along Y-Y	0.0 k
Maximum Stre Ratio = (Pu^2	ss Ratio +Mu^2)^.5 / (PhiPn	^2+PhiMn^2)^.5	0.604:1	Top along X-X	0.0k	Bottom along X-X	0.0 k
Pu =	159.580 k	φ * Pn =	264.051 k				
Mu-x = Mu-y =	0.0 k-ft 12.766 k-ft	φ * Mn-x = φ * Mn-y =	0.0 k-ft 20.863 k-ft	Maximum SERVICE Load Along Y-Y for load combination	O.O in at		
Mu Angle = Mu at Angle =	90.0 deg 12.766 k-ft	φMn at Angle =	21.116 k-ft	Along X-X for load combination	O.Oin at	0.0ft above base	
Column Capacit Pnmax : Nomin Pnmin : Nomin Φ Pn, max : U		al Capacity Axial Capacity	509.72 k -211.20 k 265.054 k -137.280 k	General Section Informat ρ : % Reinforcing Reinforcing Area Concrete Area		W. T	Θ = 0.80

#### **Governing Load Combination Results**

Governing Factored	Mon	nent	Dist.	from	Ax	ial Load			Be	nding Anal	ysis k-ft		Uti	lization
Load Combination	X-X	Y-Y	base	ft	Pu	φ * Pn	δх	δx * Mux	δУ	δy * Muy	Alpha (deg)	δMu	φMn	Ratio
+1.40D	1	M2,min	5.9	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	5.9	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	5.9	96	66.08	264.05			1.000	5.29	90.000	5.29	21.12	0.250



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

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Description :

S Only

**B22 SUPPORT** 

Governing Factored	Moment	Dist. from	Axi	ial Load			Be	ending Analy	sis k-ft		1 160	lization
Load Combination	X-X Y-Y	base ft	Pu	φ *Pn	δ×	δx*Mux	δУ	δy * Muy	Alpha (deg)	δ Mu	φMn	Ratio
+1.20D+1.60Lr+0.50L	M2,min	5.96	31.58	264.0			1.000		90.000	2.53	21.12	0.12
+1.20D+1.60Lr	M2,min	5.96	29.08				1.000		90.000	2.33	21.12	0.110
+1,20D+0.50L+1.60S	M2,min		159.58	264.0			1.000		90.000	12.77	21.12	0.60
+1.20D+1.60S	M2,min		157.08	264.0			1.000		90.000	12.57	21.12	0.59
+1.20D+0.50Lr+0.50L	M2,min	5.96	20.58	264.0			1.000		90.000	1.65	21.12	0.07
+1.20D+0.50L+0.50S	M2,min	5.96	60.58	264.0			1.000		90.000	4.85	21.12	0.22
+1.20D+0.50L+0.20S	M2,min	5.96	33.58	264.0			1.000		90.000	2.69	21.12	0.12
+0.90D	M2,min	5.96	9.81	264.0	)		1.000		90.000	0.78	21.12	0.03
Maximum Reactions	2000								te: Only non-			
Load Combination	X-X Axis @ Base	Reaction @ Top		'-Y Axis R ) Base	eaction @ Top	Axial Re @ B		My - E @ Bas	nd Moments se @ Top		//x - End Mo Base (	ments @ Top
D Only		177(0)_01				1	0.900	- STO -				
+D+L						1	5.900					
+D+Lr						2	0.900					
+D+S						10	0.900					
+D+0.750Lr+0.750L						2	2.150					
+D+0.750L+0.750S						8	2.150					
+0.60D							6.540					
Lr Only						1	0.000					
L Only							5.000					
S Only						9	0.000					
<b>Maximum Moment Reactions</b>								Not	e: Only non-	zero read	ctions are	listed.
		Momo	nt Abaud					14-	ment About Y-	Y Axis		
A TO A SHADOW STATE OF THE STAT		1400000	nt About	t X-X Axis								
Load Combination		@ Base	nt Aboui	( X-X Axis @ To	•					@ Top		
D Only		1400000	nt Aboui	1000000	k-ft						k-ft	
D Only +D+L		1400000	nt Aboui	1000000	k-ft k-ft						k-ft	
D Only +D+L +D+Lr		1400000	nt About	1000000	k-ft k-ft k-ft						k-ft k-ft	
D Only +D+L +D+Lr +D+S		1400000	nt About	100000	k-ft k-ft k-ft k-ft						k-ft k-ft k-ft	
D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L		1400000	nt About	100000	k-ft k-ft k-ft k-ft k-ft						k-ft k-ft k-ft k-ft	
D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S		1400000	nt About	100000	k-ft k-ft k-ft k-ft k-ft k-ft						k-ft k-ft k-ft k-ft k-ft	
D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D		1400000	nt About	100000	k-ft k-ft k-ft k-ft k-ft k-ft						k-ft k-ft k-ft k-ft	
D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S		1400000	nt About	100000	k-ft k-ft k-ft k-ft k-ft k-ft						k-ft k-ft k-ft k-ft k-ft k-ft	
D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only		1400000	nt About	100000	k-ft k-ft k-ft k-ft k-ft k-ft k-ft						k-ft k-ft k-ft k-ft k-ft k-ft k-ft	
D Only +D+L +D+L +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only S Only  Maximum Deflections for Loa		@ Base		@Tc	k-ft k-ft k-ft k-ft k-ft k-ft k-ft k-ft	486		@ E	3ase (		k-ft k-ft k-ft k-ft k-ft k-ft k-ft	
D Only +D+L +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only S Only  Maximum Deflections for Loa Load Combination	Max. X-X	@ Base	Dista	@ To	k-ft k-ft k-ft k-ft k-ft k-ft k-ft k-ft	Max. Y-Y De		@ E	ase (		k-ft k-ft k-ft k-ft k-ft k-ft k-ft	
D Only +D+L +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only S Only  Maximum Deflections for Loa Load Combination D Only	Max. X-X 0.000	@ Base  @ Deflection  O in	Dista	@ To	k-ft k-ft k-ft k-ft k-ft k-ft k-ft k-ft	0.00	) in	© E	ace 00 ft		k-ft k-ft k-ft k-ft k-ft k-ft k-ft	
D Only +D+L +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only S Only  Maximum Deflections for Loa Load Combination  D Only +D+L	Max. X-X 0.000 0.000	@ Base  Deflection  in  in	Dista 0.00 0.00	@ To	k-ft k-ft k-ft k-ft k-ft k-ft k-ft k-ft	0.00	) in ) in	© E  Distar  0.0  0.0	ace 00 ft 00 ft		k-ft k-ft k-ft k-ft k-ft k-ft k-ft	
D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only S Only  Maximum Deflections for Loa Load Combination  D Only +D+L +D+Lr	Max. X-X 0.000 0.000 0.000	@ Base  Deflection  in  in  in	Dista 0.00 0.00 0.00	@ To	k-ft k-ft k-ft k-ft k-ft k-ft k-ft k-ft	0.00 0.00 0.00	o in o in o in	Distar 0.0 0.0 0.0	nce 00 ft 00 ft		k-ft k-ft k-ft k-ft k-ft k-ft k-ft	
D Only +D+L +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only S Only  Maximum Deflections for Loa Load Combination  D Only +D+L +D+L +D+Lr +D+S	Max. X-X 0.000 0.000 0.000 0.000	@ Base  Deflection  in  in  in  in  in	Dista 0.00 0.00 0.00 0.00	@ To	k-ft k-ft k-ft k-ft k-ft k-ft k-ft k-ft	0.00 0.00 0.00 0.00	in i	Distar 0.0 0.0 0.0	nce 00 ft 00 ft 00 ft		k-ft k-ft k-ft k-ft k-ft k-ft k-ft	
D Only +D+L +D+L +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only S Only  Maximum Deflections for Loa Load Combination  D Only +D+L +D+L +D+L +D+S +D+0.750Lr+0.750L	Max. X-X 0.000 0.000 0.000 0.000 0.000	@ Base  Deflection  in  in  in  in  in  in	Dista 0.00 0.00 0.00 0.00 0.00	@ To	k-ft k-ft k-ft k-ft k-ft k-ft k-ft k-ft	0.00 0.00 0.00 0.00	in i	Distar 0.0 0.0 0.0 0.0	nce 00 ft 00 ft 00 ft 00 ft		k-ft k-ft k-ft k-ft k-ft k-ft k-ft	
D Only +D+L +D+L +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only S Only  Maximum Deflections for Loa Load Combination  D Only +D+L +D+L +D+Lr +D+S +D+0.750L+0.750S	Max. X-X 0.000 0.000 0.000 0.000 0.000 0.000	@ Base  Deflection  in  in  in  in  in  in  in  in  in	Dista 0.00 0.00 0.00 0.00 0.00 0.00	@ To	k-ft k-ft k-ft k-ft k-ft k-ft k-ft k-ft	0.00 0.00 0.00 0.00 0.00	in	Distar 0.0 0.0 0.0 0.0 0.0	nce 00 ft 00 ft 00 ft 00 ft 00 ft 00 ft		k-ft k-ft k-ft k-ft k-ft k-ft k-ft	
D Only +D+L +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only S Only  Maximum Deflections for Loa Load Combination  D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D	Max. X-X 0.000 0.000 0.000 0.000 0.000 0.000	@ Base  Deflection  in  in  in  in  in  in  in  in  in	Dista 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.	@ To	k-ft k-ft k-ft k-ft k-ft k-ft k-ft k-ft	0.00 0.00 0.00 0.00 0.00 0.00	in i	Distar 0.0 0.0 0.0 0.0 0.0 0.0	nce 00 ft		k-ft k-ft k-ft k-ft k-ft k-ft k-ft	
D Only +D+L +D+L +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D Lr Only L Only S Only  Maximum Deflections for Loa Load Combination  D Only +D+L +D+L +D+Lr +D+S +D+0.750L+0.750S	Max. X-X 0.000 0.000 0.000 0.000 0.000 0.000	@ Base  Deflection  in  in  in  in  in  in  in  in  in	Dista 0.00 0.00 0.00 0.00 0.00 0.00	@ To	k-ft k-ft k-ft k-ft k-ft k-ft k-ft k-ft	0.00 0.00 0.00 0.00 0.00	in i	Distar 0.0 0.0 0.0 0.0 0.0	Once  00 ft		k-ft k-ft k-ft k-ft k-ft k-ft k-ft	

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VILLAGE NEST JJH MULTI UNIT PROJECT

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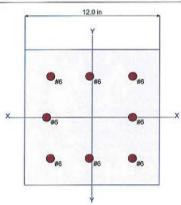
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#### Sketches

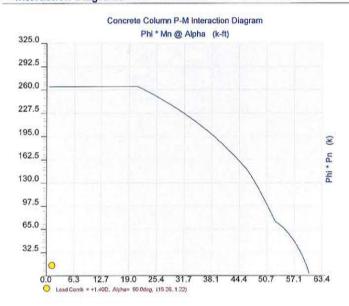


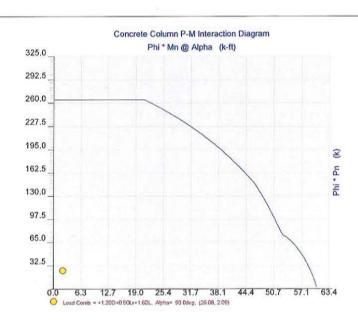




( Axis

#### Interaction Diagrams







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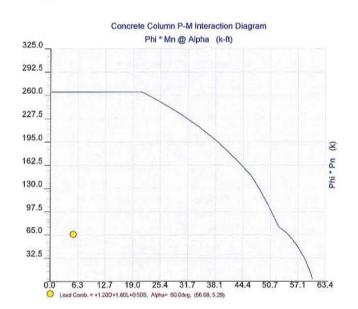
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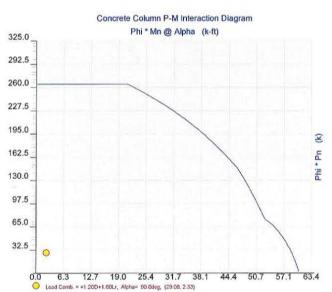
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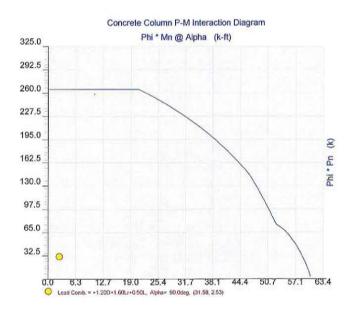
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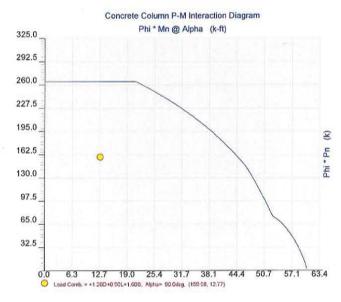
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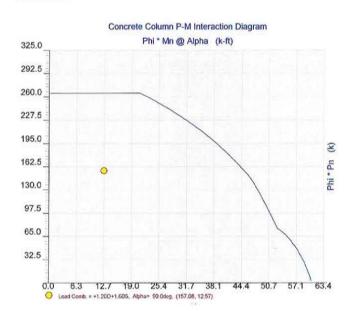
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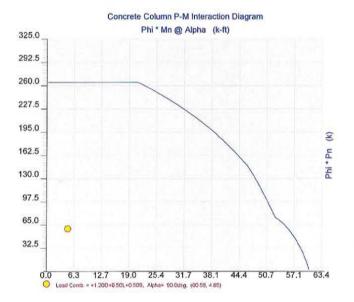
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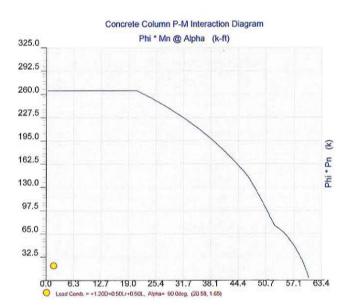
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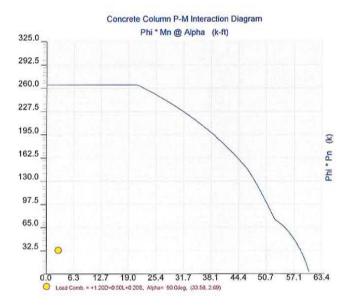
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**B22 SUPPORT** Description:











VILLAGE NEST JJH

JJH MULTI UNIT PROJECT Project ID: 2017-0610

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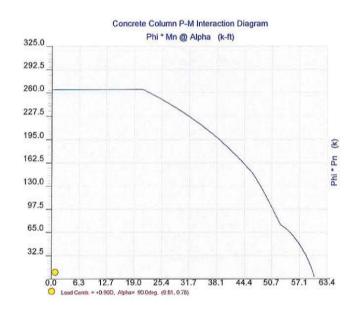
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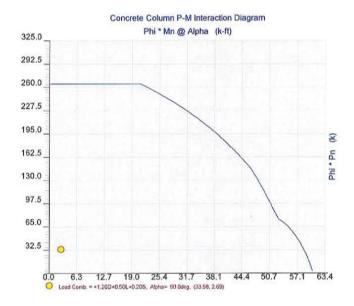
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VILLAGE NEST JJH MULTI UNIT PROJECT

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## **General Footing**

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MAX PAD: FELDSPAR UVN

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#### Code References

Description:

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10

Load Combinations Used: ASCE 7-10

#### **General Information**

Mate	erial Pro	perties			
		28 day strength	=	2	2.50 ksi
	: Rebar Yie		=	6	50.0 ksi
Éc	: Concrete	Elastic Modulus	=		22.0 ksi
Co	oncrete De	nsity	=	14	45.0 pcf
φ	Values	Flexure	=	(	0.90
2		Shear	=	0.	850
Anal	lysis Set	tings			
Mi	n Steel %	Bending Reinf.		=	
Mi	n Allow %	Temp Reinf.		=	0.00180
Mi	n. Overturr	ning Safety Factor		=	1.50 :
Mi	n. Sliding S	Safety Factor		=	1.50 :
Ad	ld Ftg Wt fo	or Soil Pressure		:	No
Us	se ftg wt for	stability, moments & shears			No
		Wt for Soil Pressure		3	No
Us	e Pedesta	I wt for stability, mom & shear			No
1-0/2	And the second second				

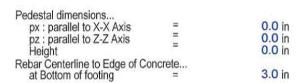
Soil Design Values		
Allowable Soil Bearing	=	2.50 ksf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	180.0 pcf
Soil/Concrete Friction Coeff.	=	0.450

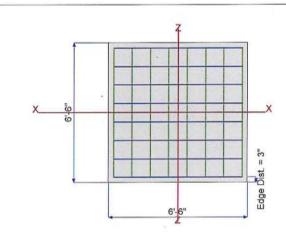
## Increases based on footing Depth Footing base depth below soil surface = 2.0 ft Allow press. increase per foot of depth = 0.0 ksf when footing base is below = 0.0 ft

## Increases based on footing plan dimension Allowable pressure increase per foot of depth when max. length or width is greater than = 0.0 ksf

#### **Dimensions**

Width parallel to X-X Axis	=	6.50 ft
Length parallel to Z-Z Axis	=	6.50 ft
Footing Thickness	=	24.0 in





#### Reinforcing

Bars parallel to X-X Axis	=		8
Number of Bars Reinforcing Bar Size	=	#	6
Bars parallel to Z-Z Axis			120
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





**Applied Loads** 

		D	Lr	L	S	W	E	Н
P : Column Load OB : Overburden	Ī .	10.0 0.0	10.0	5.0 0.0	90.0 0.0	0.0	0.0	0.0 k 0.0 ksf
M-xx M-zz	= =	0.0	0.0	0.0	0.0	0.0	0.0	0.0 k-ft 0.0 k-ft
V-x V-z	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0 k 0.0 k



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 11 JUL 2017, 3:49PM

## **General Footing**

Lic. # : KW-06002886 Description : MAX PA

MAX PAD: FELDSPAR UVN

File = d:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29 Ligensee: RICHIMOND HOFFMAYER

DESIGN SU	MMARY				Design OK
	Min. Ratio	Item	Applied	Capacity	<b>Governing Load Combination</b>
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
etailed Res	sults	T CALLY A WALLESS W	trestammentary (George	467-5-1-01 W. SV	CONTACT STORES STRANG

Rotation Axis &		Xecc	Zecc	Actual	Soil Bearing S	tress @ Loca	Actual / Allow	
Load Combination	Gross Allowable	(i	n)	Bottom, -Z	Top, +Z	Left, -X	Right, +X	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	5507.0	7000	180070	A8.433	15.000	05 AV. 1.0000		2000-2000

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 -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



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

## **General Footing**

Lic. # : KW-06002886

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Licensee: RICHMOND HOFFMAYER

MAX PAD: FELDSPAR UVN Description:

				(-1-		2012	
_ 1	_	_	-	4:	-	 lex	 -
	•	n	n	П		62.8	T E

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actua in^	70 (7) (7)	Phi*Mn k-ft		Status
Z-Z, +1.40D	1.750	-X	Bottom	0.5184	Min Temp %	6	0.5415	49.623		OK
Z-Z. +1.40D	1.750	+X	Bottom	0.5184	Min Temp %	6	0.5415	49.623		OK
Z-Z, +1.20D+0.50Lr+1.60L	3.125	-X	Bottom	0.5184	Min Temp %	ó	0.5415	49.623		OK
Z-Z, +1.20D+0.50Lr+1.60L	3.125	+X	Bottom	0.5184	Min Temp %	ó	0.5415	49.623		OK
Z-Z, +1.20D+1.60L+0.50S	8.125	-X	Bottom	0.5184	Min Temp %	ó	0.5415	49.623		OK
Z-Z, +1.20D+1.60L+0.50S	8.125	+X	Bottom	0.5184	Min Temp %	ó	0.5415	49.623		OK
Z-Z, +1.20D+1.60Lr+0.50L	3.813	-X	Bottom	0.5184	Min Temp %	ó	0.5415	49.623		OK
Z-Z, +1.20D+1.60Lr+0.50L	3.813	+X	Bottom	0.5184	Min Temp %	Ó	0.5415	49.623		OK
Z-Z, +1.20D+1.60Lr	3.50	-X	Bottom	0.5184	Min Temp %	ó	0.5415	49.623		OK
Z-Z, +1.20D+1.60Lr	3.50	+X	Bottom	0.5184	Min Temp %	ó	0.5415	49.623		OK
Z-Z, +1.20D+0.50L+1.60S	19.813	-X	Bottom	0.5184	Min Temp %	o	0.5415	49.623		OK
Z-Z. +1.20D+0.50L+1.60S	19.813	+X	Bottom	0.5184	Min Temp %	o	0.5415	49.623		OK
Z-Z, +1.20D+1.60S	19.50	-X	Bottom	0.5184	Min Temp %	b	0.5415	49.623		OK
Z-Z, +1.20D+1.60S	19.50	+X	Bottom	0.5184	Min Temp %	6	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 %	6	0.5415	49.623		OK
Z-Z, +1.20D+0.50L+0.50S	7.438	-X	Bottom	0.5184	Min Temp %	5	0.5415	49.623		OK
Z-Z, +1.20D+0.50L+0.50S	7.438	+X	Bottom	0.5184	Min Temp %	5	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 %	b	0.5415	49.623		OK
Z-Z, +0.90D	1.125	-X	Bottom	0.5184	Min Temp %	5	0.5415	49.623		OK
Z-Z, +0.90D	1.125	+X	Bottom	0.5184	Min Temp %	5	0.5415	49.623		OK
One Way Shear		5,802	100000000000000000000000000000000000000	0.39360.000.000	27727129-20 52670-080 504	× × × × × × × × × × × × × × × × × × ×	THE CONTROL OF THE CO	2-600-700-0000	-	SANCK.
Load Combination	Vu @ -X	Vu @ +>	√ Vu@	)-Z Vu (	⊕+Z V	u:Max	Phi Vn	Vu / Phi	*Vn	Status
+1.40D	1.99 p	si	1.99 psi	1.99 psi	1.99 psi	1.99 ps	si 85.0	iaq 00	0.02	0.00
+1 20D+0 50Lr+1 60L	3 56 n	ei .	3 56 nei	3 56 ngi	3 56 nei	3.56 ns	95 (	nsi ng	0.04	0.00

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 ps	1.99 psi	1.99 psi	85.00 ps	i 0.02	0.00
+1.20D+0.50Lr+1.60L	3.56 psi	3.56 psi	3.56 ps	3.56 psi	3.56 psi	85.00 ps	i 0.04	0.00
+1.20D+1.60L+0.50S	9.26 psi	9.26 psi	9.26 ps	9.26 psi	9.26 psi	85.00 ps	i 0.11	0.00
+1.20D+1.60Lr+0.50L	4.35 psi	4.35 psi	4.35 ps	4.35 psi	4.35 psi	85.00 ps	i 0.05	0.00
+1.20D+1.60Lr	3.99 psi	3.99 psi	3.99 ps	3.99 psi	3.99 psi	85.00 ps	i 0.05	0.00
+1.20D+0.50L+1.60S	22.58 psi	22.58 psi	22.58 ps	22.58 psi	22.58 psi	85.00 ps	i 0.27	0.00
+1.20D+1.60S	22,22 psi	22.22 psi	22.22 ps	22.22 psi	22.22 psi	85.00 ps	i 0.26	0.00
+1.20D+0.50Lr+0.50L	2.78 psi	2.78 psi	2.78 ps	2.78 psi	2.78 psi	85.00 ps	i 0.03	0.00
+1.20D+0.50L+0.50S	8.48 psi	8.48 psi	8.48 ps	8.48 psi	8.48 psi	85.00 ps	i 0.10	0.00
+1.20D+0.50L+0.20S	4.63 psi	4.63 psi	4.63 ps	4.63 psi	4.63 psi	85.00 ps	i 0.05	0.00
+0.90D	1.28 psi	1.28 psi	1.28 ps	1.28 psi	1.28 psi	85.00 ps	i 0.02	0.00
Two-Way "Punching" Shear							All units	k

Load Combination	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	<b>7.37</b> psi	170.00 psi	0.04337	OK
+1.20D+0.50Lr+1.60L	13.17 psi	170.00 psi	0.07744	OK
+1.20D+1.60L+0.50S	34.23 psi	170.00 psi	0.2013	OK
+1.20D+1.60Lr+0.50L	16.06 psi	170.00 psi	0.09448	OK
+1.20D+1.60Lr	14.74 psi	170.00 psi	0.08673	OK OK
+1.20D+0.50L+1.60S	83.46 psi	170.00 psi	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	170.00 psi	0.1007	OK OK OK
+0.90D	4.74 psi	170.00psi	0.02788	OK



VILLAGE NEST JJH MULTI UNIT PROJECT

Project ID: 2017-0610

Printed: 21 JUL 2017, 10:01AM

#### **Concrete Beam**

Lic. #: KW-06002886

Description: \_\_FELDSPAR UVN LOWER FLOOR - B22 - CONCRETE

File = D:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29 Licensee: RICHMOND HOFFMAYER

REPLAKE

STEEL W/ CONC. BEDA

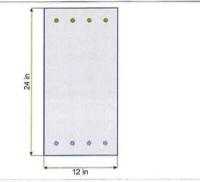
#### **CODE REFERENCES**

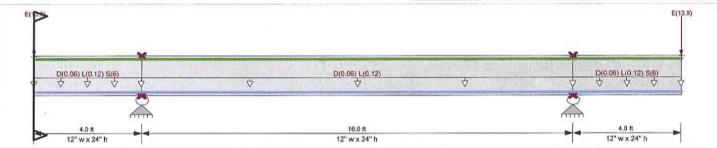
Calculations per ACI 318-14, IBC 2015, ASCE 7-10

Load Combination Set : ASCE 7-10

#### **Material Properties**

	Annual Indiana							_
$f'c$ = $f'c^{1/2} * 7.50$	=	2.50 ksi 375.0 psi	φ	Phi Values		Flexure : Shear :	0.90	
Ψ Density	=	145.0 pcf		β 1	=	470 MATERIA 54	0.850	
λ LtWt Factor	=	1.0						
Elastic Modulus =		3,122.0 ksi	Fy	- Stirrups		4	10.0 ksi	
fy - Main Rebar = E - Main Rebar =		60.0 ksi 29,000.0 ksi		- Stirrups irrup Bar Size i	=	29,00	00.0 ksi 3	
E - Main Rebar =		Number of Resis	ting L	.egs Per Stirru	p =		2	
				and the same and t				





#### **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

Span #2 Reinforcing....

4-#6 at 2.0 in from Bottom, 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

Applied Loads

4-#6 at 2.0 in from Top, from 0.0 to 4.0 ft in this span

4-#6 at 2.0 in from Top, from 0.0 to 16.0 ft in this span

4-#6 at 2.0 in from Top, from 0.0 to 4.0 ft in this span

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**

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

 naximum on span
 0.000 ft

Location of maximum on span 0.000
Span # where maximum occurs Span # 2

Maximum Deflection

Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection

0.163 in Ratio = -0.145 in Ratio = 0.184 in Ratio = -0.152 in Ratio =

Ratio = 520>=18 Ratio = 1267>=18

590>=36

1325>=36

Design OK

#### **Vertical Reactions**

Support notation		Far	left	is	#
------------------	--	-----	------	----	---

V CI LICUI I (CUCLIOII)			A THE PROPERTY OF THE		
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		No.



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

Printed: 21 JUL 2017, 10:01AM

Concrete Beam

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Lic. # : KW-06002886

Description:

FELDSPAR UVN LOWER FLOOR - B22 - CONCRETE

Support notation: Far left is #1 **Vertical Reactions** 

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**

Overall Maximum Benedicite										
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				



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID:

2017-0610

Printed: 21 JUL 2017, 9:59AM

Steel Base Plate Lic. #: KW-06002886

File = D:\ENERCALC Projects\2017-0610.ec6 ENERCALC, INC. 1983-2017, Build:10.17.6.29, Ver:10.17.6.29

Description:

P.C. BASE PLATE

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

Concrete Support f'c

AISC Design Method Load Resistance Factor Design

36.0 ksi 2.50 ksi

Assumed Bearing Area: Bearing Area = P / Fp

Φ c : LRFD Resistance Factor

0.60

Allowable Bearing Fp per J8

4.017 ksi

Column & Plate

Steel Plate Fy

Column Properties

Steel Section: Pipe4STD

Depth 4.5 in 4.5 in Width 0.221 in Flange Thickness

Area 1XX lyy

2.96 in^2 in^4 in^4

in

Plate Dimensions

Web Thickness

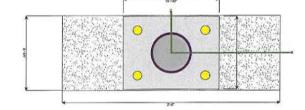
8.0 in N: Length B: Width 10.50 in 0.750 in Thickness

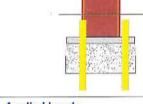
Column assumed welded to base plate.

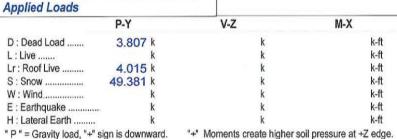


8.250 in Width along "X" Length along "Z' 24.0 in

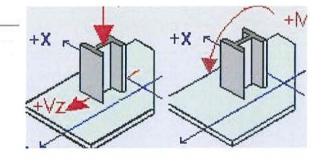








- "+" Moments create higher soil pressure at +Z edge.
  - "+" Shears push plate towards +Z edge.



### **GOVERNING DESIGN LOAD CASE SUMMARY**

Plate Design Summary Design Method

Load Resistance Factor Design

Governing Load Combination Governing Load Case Type Design Plate Size

+1.20D+1.60S Axial Load Only

8" x 10 -1/2" x 0 -3/4"

Pu : Axial ..... Mu: Moment ...... 83.578 k 0.000 k-ft Mu : Max. Moment ..... fb: Max. Bending Stress .....

Fb: Allowable: Fy \* Phi

Bending Stress Ratio

0.038 Bending Stress OK

fu: Max. Plate Bearing Stress ....

Fp: Allowable:

min( 0.85\*fc\*sqrt(A2/A1), 1.7\* fc)\*Phi

Bearing Stress Ratio

1.706 ksi 1.722 ksi

0.173 k-in

1.228 ksi

32.400 ksi

0.991

Bearing Stress OK



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

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Steel Base Plate

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Licensee: RICHMOND HOFFMAYER

Lic. #: KW-06002886

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

36.0 ksi

Steel Plate Fv Concrete Support f'c

2.50 ksi

Assumed Bearing Area : Bearing Area = P / Fp

Allowable Bearing Fp per J8

Φ c : LRFD Resistance Factor

3.814 ksi

0.60

### Column & Plate

#### Column Properties

Steel Section: HSS6x6x3/8

Depth

6 in 6 in

0.349 in

Area lxx

lyy

7.58 in^2 in^4

Width Flange Thickness

Web Thickness

#### Plate Dimensions

N: Length B: Width Thickness

8.0 in 16.0 in

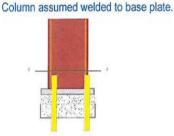
0.8750 in

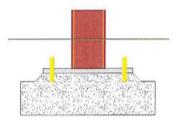
Support Dimensions Width along "X"

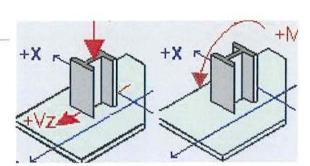
Length along "Z"

8.250 in 24.0 in

in^4







### 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
"D" - Croudty load "y" a	an in down	word	"+" Mamonto create higher ceil	proceure at +7 edge

Moments create higher soil pressure at +Z edge. ' = Gravity load, "+" sign is downward.

"+" Shears push plate towards +Z edge.

#### GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method

Load Resistance Factor Design

Governing Load Combination Governing Load Case Type

+1.20D+0.50L+1.60S 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 ..... fb : Max. Bending Stress .....

Fb: Allowable:

Fy \* Phi

Bending Stress Ratio

0.489

Bending Stress OK

3.033 k-in 15.846 ksi

32,400 ksi

1.385 ksi

1.386 ksi

1.000

fu : Max. Plate Bearing Stress ....

Fp: Allowable:

min( 0.85\*fc\*sqrt(A2/A1), 1.7\* fc)\*Phi Bearing Stress Ratio

Bearing Stress OK



VILLAGE NEST JJH **MULTI UNIT PROJECT** 

Project ID: 2017-0610

#### Concrete Column

Lic. #: KW-06002886

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Licensee: RICHMOND HOFFMAYER

HSS SUPPORT - INTEGRAL w/ WALL Description:

#### 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=	$\equiv$	3,122.0 ksi
Density	=	150.0 pcf
β	=	0.850
fy - Main Rebar	=	60.0 ksi
É - Main Rebar	=	29,000.0 ksi
Allow. Reinforcing Limits	3	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

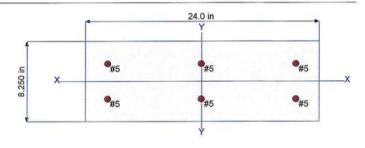
Rebar Edge Cover = 2.0in

Column Reinforcing:

4 - #5 bars @ corners,, 1.0 - #5 bars top &

+1.20D+0.50L+1.60S

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

Location of m	5.960ft		
Maximum Stre Ratio = (Pu^2	ess Ratio !+Mu^2)^.5 / (PhiPn	^2+PhiMn^2)^.5	0.582: 1
Pu =	159.985 k	φ * Pn =	274.767 k
Mu-x =	0.0 k-ft	$\phi * Mn-x =$	O.Ok-ft
Mu-y =	0.0 k-ft	Ψ * 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
Dn & Mn value	located at Du Mus	vector intersection wit	h canacity curve

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

Maximum SERVICE Load Deflections . . .

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

for load combination:

General Section Information .  $_{\text{O}} = 0.650$ 0.80 ρ: % Reinforcing 0.9394 % Rebar < Min of 1.0 %

1.860 in^2 Reinforcing Area Concrete Area 198.0 in^2

#### **Governing Load Combination Results**

Governing Factored Load Combination	Mon	nent	nt Dist. from		Axial Load			Bending Analysis k-ft					Utilization		
	X-X	Y-Y	base	ft	Pu	φ*Pn	δ×	δx*Mux	δУ	δy * Muy	Alpha (deg)	δ Μυ	φ Mn	Ratio	
+1.40D			5.9	96	15.73	274.77					0.000			0.057	
+1.20D+0.50Lr+1.60L			5.9	96	26.49	274.77					0.000			0.096	
+1.20D+1.60L+0.50S			5.9	96	66.49	274.77					0.000			0.242	



VILLAGE NEST JJH

**MULTI UNIT PROJECT** 

Project ID: 2017-0610

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**Concrete Column** 

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Lic. #: KW-06002886 HSS SUPPORT - INTEGRAL w/ WALL Description:

Governing Factored	Moment	Dist. from	Axial k	Axial Load			ending Analysis k-ft				ı	Utilization	
Load Combination	X-X Y-Y	base ft	Pu φ	* Pn	δ×	δx*Mux 8	у	Sy * Muy	Alpha	a (deg)	δMu		Ratio
+1.20D+1.60Lr+0.50L		5.96	31.99	274.77					0	0.000			0.11
+1.20D+1.60Lr		5.96	29.49	274.77					0	0.000			0.10
-1.20D+0.50L+1.60S			159.99	274.77					0	0.000			0.582
+1.20D+1.60S			157.49	274.77						0.000			0.57
+1.20D+0.50Lr+0.50L		5.96	20.99	274.77						0.000			0.07
+1.20D+0.50L+0.50S		5.96	60.99	274.77						0.000			0.22
		5.96	33.99	274.77						0.000			0.12
+1.20D+0.50L+0.20S										0.000			0.03
Ю.90D		5.96	10.11	274.77				410			1400 BU		
Maximum Reactions						4.1.15				-		actions ar	
Load Combination	X-X Axis @ Base	Reaction @ Top		Axis Rea	ction Top	Axial Reacti @ Base		@ Ba	ind Mor se	ments @ Top	k-ft	Mx - End N @ Base	@ Top
D Only	9	G	0 -	0		11.23			20				
+D+L						16.23							
+D+Lr						21.23							
+D+S						101.23							
+D+0.750Lr+0.750L						22.48							
+D+0.750L+0.750S						82.48	38						
+0.60D						6.74							
Lr Only						10.00	00						
L Only						5.00	00						
S Only						90.00	00						
Maximum Moment Reacti	ons					\$ P\$\$ Voc. 1000		7,50				actions ar	e listed.
Load Combination		Mome @ Base	nt About X	(-X Axis @ Top					oment. Base	'-About Y آھ	Y Axis ) Top		
D Only		@ 5000		@ 10p	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 7001 0 7000					k-ft							k-ft	
					k-ft							k-ft	
+0.60D					k-ft							k-ft k-ft	
+0.60D Lr Only												k-ft	
+0.60D Lr Only L Only					k-ft k-ft								
+0.60D Lr Only L Only S Only	Load Combination	s											
+0.60D Lr Only L Only S Only <b>Maximum Deflections for</b>		s K Deflection	Distanc	ре	k-ft	Max. Y-Y Deflec	ction	Dista	nce			200V.01.1	
+0.60D Lr Only L Only S Only <b>Maximum Deflections for</b>	Max. X	X Deflection 00 in	0.000	ft	k-ft	0.000	ction	0.0	000	ft			
+0.60D Lr Only L Only S Only <b>Maximum Deflections for</b> Load Combination	Max. X 0.000 0.000	X Deflection 00 in 00 in	0.000	ft	k-ft	0.000 0.000	in in	0.0	000	ft			
+0.60D Lr Only L Only S Only Maximum Deflections for Load Combination D Only +D+L +D+Lr	Max. X- 0.000 0.000 0.000	X Deflection 00 in 00 in 00 in	0.000 0.000 0.000	ft ft ft	k-ft	0.000 0.000 0.000	in in in	0.0 0.0 0.0	000	ft ft			
+0.60D Lr Only L Only S Only  Maximum Deflections for Load Combination  D Only +D+L +D+Lr +D+S	Max. X- 0.000 0.000 0.000 0.000	X Deflection 00 in 00 in 00 in 00 in	0.000 0.000 0.000 0.000	ft ft ft	k-ft	0.000 0.000 0.000 0.000	in in	0.0 0.0 0.0	000 000 000	ft ft ft			
+0.60D Lr Only L Only S Only  Maximum Deflections for Load Combination  D Only +D+L +D+L +D+S +D+0.750Lr+0.750L	Max. X- 0.000 0.000 0.000 0.000 0.000	X Deflection 00 in 00 in 00 in 00 in 00 in	0.000 0.000 0.000	ft ft ft ft	k-ft	0.000 0.000 0.000 0.000 0.000	in in in	).0 ).0 ).0 ).0	000 000 000 000	ft ft ft ft		,	
+0.60D Lr Only L Only S Only Maximum Deflections for Load Combination  D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S	Max. X- 0.000 0.000 0.000 0.000 0.000 0.000	X Deflection 00 in 00 in 00 in 00 in 00 in 00 in	0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft	k-ft	0.000 0.000 0.000 0.000 0.000 0.000	in in in in in	).0 ).0 ).0 ).0 ).0	000 000 000 000 000	ft ft ft ft ft			
+0.60D Lr Only L Only S Only Maximum Deflections for Load Combination  D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D	Max. X- 0.000 0.000 0.000 0.000 0.000 0.000 0.000	C Deflection OO in	0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft	k-ft	0.000 0.000 0.000 0.000 0.000 0.000 0.000	in in in in	).0 ).0 ).0 ).0 ).0 ).0	000 000 000 000 000 000	ft ft ft ft ft ft		٠	
+0.60D Lr Only L Only S Only Maximum Deflections for Load Combination  D Only +D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S	Max. X- 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	X Deflection 00 in	0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft	k-ft	0.000 0.000 0.000 0.000 0.000 0.000 0.000	in in in in in in	3.0 3.0 3.0 3.0 3.0 3.0 3.0	000 000 000 000 000 000 000	ft ft ft ft ft ft			
+D+L +D+Lr +D+S +D+0.750Lr+0.750L +D+0.750L+0.750S +0.60D	Max. X- 0.000 0.000 0.000 0.000 0.000 0.000 0.000	X Deflection	0.000 0.000 0.000 0.000 0.000 0.000 0.000	ft ft ft ft ft	k-ft	0.000 0.000 0.000 0.000 0.000 0.000 0.000	in in in in in in	3.0 3.0 3.0 3.0 3.0 3.0 3.0	000 000 000 000 000 000 000	ft ft ft ft ft ft			



VILLAGE NEST JJH

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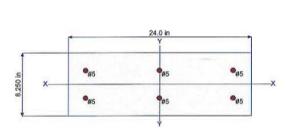
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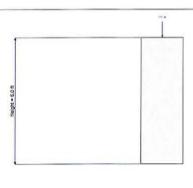
Description:

HSS SUPPORT - INTEGRAL w/ WALL

#### Sketches



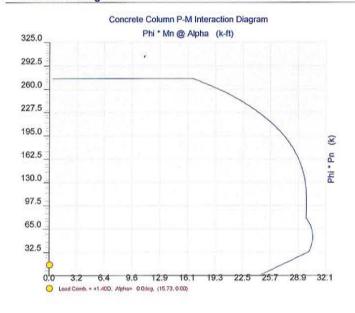


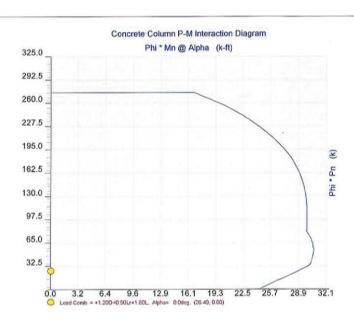


Lecking along X-X Axis

Looking along Y-Y Axis

#### Interaction Diagrams







VILLAGE NEST JJH

MULTI UNIT PROJECT

Project ID: 2017-0610

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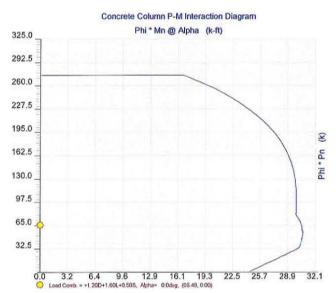
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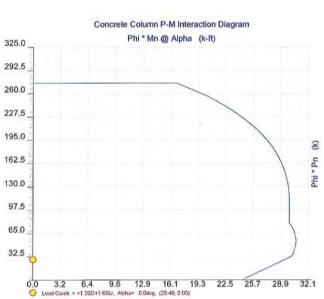
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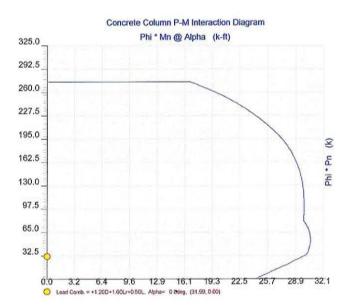
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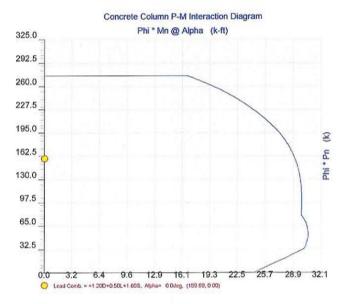
Description : HSS SUPF

HSS SUPPORT - INTEGRAL w/ WALL











VILLAGE NEST JJH

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Project ID: 2017-0610

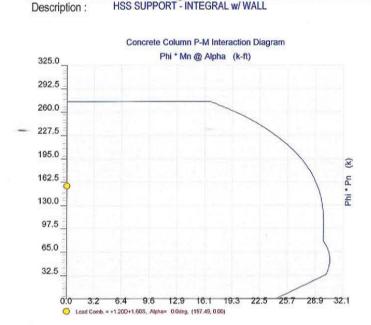
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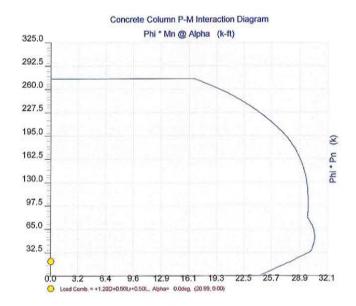
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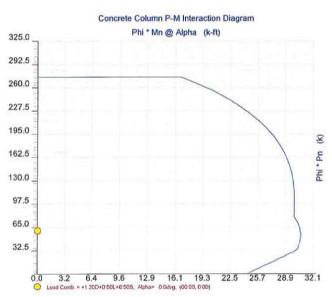
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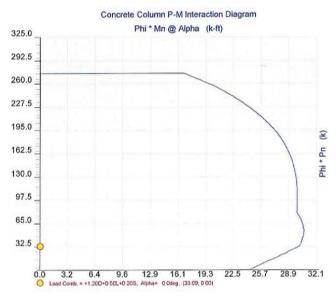
HSS SUPPORT - INTEGRAL w/ WALL

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VILLAGE NEST JJH

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### **Concrete Column**

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Description:

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