

### **Structural Calculations**

For

### **NAC Recreation Center – Park City**

**Project Number: 18121** 

**August 9, 2018** 



Prepared by ARW Engineers 1594 West Park Circle Ogden, Utah 84404





### STRUCTURAL CALCULATIONS

**FOR** 

### **NAC Recreation Center – Park City**

Client: Arch Nexus

Project Number: 17402

### **DESIGN CRITERIA**

**GOVERNING CODE: IBC 2015** 

**GENERAL:** Risk Category = II

**SEISMIC:** Seismic Design Category = D

 $I_E = 1.0$  R = 6.5

 $S_{DS} = 0.624$ 

WIND: Basic Wind Speed = 115 mph

**Exposure Classification = C** 

SOILS: Site Class: D

**Design Allowable Soil Pressure = 1800 psf** 

As per Soils Report by: IGES

**Dated: March 29, 2018** 

### **DESIGN LOADS**

ROOFS: DL = 15 psf SL = 97 psf



### **CALCULATIONS INDEX**

SECTION	PAGE #
Roof Framing	A1 to A38
Lateral Analysis	B1 to B39
Walls	C1 to C22

### ROOF FRAMING



### **ROOF FRAMING**

### ROOF A: DEAD LOADS

Roofing:	2	psf
Batting/Blown Insulation:	2	psf
Sheathing:	2	psf
Framing:	5	psf
Mechanical Ducts/Misc.:	1	psf
Fire Sprinkling:	1	psf
Ceilings:	2	psf
Collateral:		psf
Total Dead Load:	15	psf
Seismic Roof Snow Load:	19.5	psf
Seismic Mass Dead Load:	34.5	psf

## Comments

### ROOF B: DEAD LOADS

Roofing:		psf
Batting/Blown Insulation:		psf
Sheathing:		psf
Framing:		psf
Mechanical Ducts/Misc.:		psf
Fire Sprinkling:		psf
Ceilings:		psf
Collateral:		psf
Total Dead Load:	0	psf
Seismic Roof Snow Load:	0	psf
Seismic Mass Dead Load:	0	psf

Comments		

### LIVE LOADS

20 psf

Comments

### SNOW LOADS

psf	139.0	Ground Snow Load :
	1	Snow Exposure Factor, C <sub>e</sub> :
	1	Snow Load Importance Factor, Is:
	1	Thermal Factor, Ct:
= psf	97	Flat Roof Snow Load :



Project No. \_\_\_\_\_\_Sheet No. \_\_\_\_AZ

Project \_\_\_\_

structural consultants	Tropared By	Date
PLYWOOD BENDING CAPACITY	<u>/_</u>	
ROOF 19/32" 40/20	O SPAN RATING	
F6S = 688 LB-IN/FT	" 90° 4 PLY	
S W = 120 Fbs  Lb Z  W = 143 PSF	Lb = 24 W	
$W_{s} = \frac{20  \text{Fs}}{L_{s}}$ $W_{s} = \frac{20  \text{Fs}}{L_{s}}$ $W_{s} = \frac{219  \text{psf}}{L_{s}}$	Fs = 246 Ls = 24-1.5 = 22.5"	
WL = 1743 EI ALL  VOL = 168 PSF  WL = 1743 EI ATL  LA	$L_{\Delta} = 24 - 1.5 = 22.5^{\circ}$ EI = 247500 $\Delta_{LL} = 24/240 = 0.1^{\circ}$ $\Delta_{LL} = 24/180 = 0.13^{\circ}$	
WIL = 219 PSF  CANTILEVER EAVE $F_b S = \frac{W}{96L^2} \left[ L + a \right]^2 \left[ L - a \right]^2 \left[ L$	a] <sup>2</sup> L= 24"	
$W_b = 204 \text{ psf}$ $W_b = \frac{24 \text{ Fbs}}{L^2}$	a = 12"  Lb = 12"	
W - 114 PSF	Use 12" MAX OVER	CHANG LENGTH

Table A

					Stren	gth			Planar	Shear	Stiffness and Rigidity					
Span Rating		Bending FbS (lb-in/ft of width)  Axial Tension FtA (lb/ft of width)		Compi F	rial ression - A f width)	Shear through the thickness (b.c) Fv.t, (lb/in of shear- resisting panel length)	Planar Shear F <sub>s</sub> (lb/Q) (lb/ft of width)		Bend El (lb-in²/ft o		A: E (lb/ft of w	Rigidity through the thickness G <sub>v</sub> t <sub>v</sub> (lb/in of panel depth				
			Capacities relative to strength axis <sup>(d)</sup>										•			
		0°	90°	0°	90°	0°	0° 90°		0°	90°	0°	90°	0°	90°	0°/90°	
Sheathir	ng Span <sup>®</sup>															
24/0	3-ply	250	54	2,300	600	2,850	2,500	53	156	273	66,000	3,600	3.35	2.90	25,000	
32/16	3-ply	370	92	2,800	1,250	3,550	3,100	62	198	347	126,500	8,100	4.15	3.60	27,000	
	4-ply	407	110	2,800	1,250	5,325	4,650	81	198	479	126,500	17,820	4.15	3.60	35,100	
	5-ply	444	166	3,640	1,625	5,325	4,650	93	215	165	126,500	25,110	4.15	3.60	40,500	
40/20	3-ply	625	150	2,900	1,600	4,200	4,000	68	246	431	247,500	18,000	5.00	4.50	28,500	
	4-ply	688	180	2,900	1,600	6,300	6,000	88	246	595	247,500	39,600	5.00	4.50	37,050	
	5-ply	750	270	3,770	2,080	6,300	6,000	102	267	205	247,500	55,800	5.00	4.50	42,750	
48/24	4-ply	930	270	4,000	1,950	7,500	7,200	98	300	725	440,000	64,900	5.85	5.00	40,300	
	5-ply	1,014	405	5,200	2,535	7,500	7,200	113	325	250	440,000	91,450	5.85	5.00	46,500	
Floor	Span <sup>®</sup>															
20 oc	4-ply	528	168	2,900	1,600	6,300	6,000	87	246	595	231,000	28,600	5.00	4.50	36,400	
	5-ply	576	252	3,770	2,080	6,300	6,000	101	267	205	231,000	40,300	5.00	4.50	42,000	
24 oc	4-ply	704	258	3,350	1,950	7,500	7,200	96	300	725	330,000	57,200	5.85	5.00	39,000	
	5-ply	768	387	4,355	2,535	7,500	7,200	111	325	250	330,000	80,600	5.85	5.00	45,000	
32 oc	5-ply	1,044	684	5,200	3,250	9,450	9,300	120	390	300	715,000	232,500	7.50	7.30	54,000	
48 oc	5-ply	1,920	1,224	7,280	4,745	12,150	10,800	158	501	385	1,265,000	496,000	8.20	7.30	75,750	

The design values in this table correspond with those published in the 2005 edition of the AF&PA American Wood Council's *Allowable Stress Deign (ASD)/LRFD Manual for Engineered Wood Construction* Tables M9.2.1- M9.2.4, which are available from the AF&PA American Wood Council. The appropriate panel grade and construction adjustment factor, C<sub>G</sub>, has already been incorporated into these design values—do not apply the C<sub>G</sub> factor a second time. These values do not apply to Structural I panels. See Tables M9.2.1 – M9.2.4 for the appropriate multipliers for Structural I panels.

Shear through the thickness design capacities are limited to sections two feet or less in width; wider sections may require further reductions.

5-ply applies to plywood with 5 or more layers; for 5-ply/3-layer plywood, use values for 4-ply plywood.

Strength axis is defined as the axis parallel to the face and back orientation of the grain (veneer), which is generally the long panel direction, unless otherwise marked.

Project Title: Engineer: Project Descr:

Project ID:



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### **Wood Beam**

File = Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\Other\18121.ec6 ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.10 Lic. #: KW-06002489

Licensee: ARW ENGINEERS

Description: Activity Room Joists

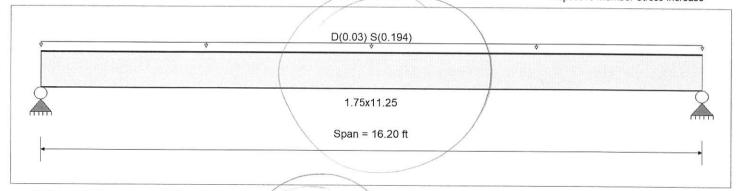
### **CODE REFERENCES**

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+	2600 psi	E : Modulus of Elas	ticity
Load Combination :ASCE 7-10	Fb-	2600 psi	Ebend- xx	2000 ksi
	Fc - Prll	2510 psi	Eminbend - xx	1016.535ksi
Wood Species : Trus Joist	Fc - Perp	750 psi		
Wood Grade : MicroLam LVL 2.0 E	Fv	285 psi		
	Ft	1555 psi	Density	42 pcf
Beam Bracing : Beam is Fully Braced against lateral-torsi	onal buckling		Repetitive Memb	er Stress Increase



### **Applied Loads**

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

Uniform Load: D = 0.0150, S = 0.0970 ksf, Tributary Width = 2.0 ft

DESIGN SUMMARY		Design O	K
Maximum Bending Stress Ratio Section used for this span fb : Actual FB : Allowable	= 0.695 1 Maximum Shear Stress Ratio 1.75x11.25 Section used for this span 2,388.79psi fv : Actual 3,438.50psi Fv : Allowable	= 0.370 1.75x11.2 = 123.10 = 327.75	6 : 1 <b>5</b> 0 psi
Load Combination Location of maximum on span Span # where maximum occurs	+D+S Load Combination = 8.100ft Location of maximum on span = Span # 1 Span # where maximum occurs	+D+; = 15.31 = Span#	S 3ft
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	0.728 in Ratio = 266 >= 240 0.000 in Ratio = 0 < 240 0.841 in Ratio = 231 >= 180 0.000 in Ratio = 0 < 180		

Maximum	Forces &	Stresses for	<b>Load Combinations</b>
INIMATITION	. 0.0000	Oli Cooco IOI	Luad Cullibiliations

Load Combination	Max Stress Ratios										Mor	ment Values		Shear Values		
Segment Length	Span #	M	V	$C_d$	C F/V	Ci	$c_r$	$C_{m}$	$C_t$	C <sub>L</sub>	М	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 16.20 ft	1	0.119	0.064	0.90	1.000	1.00	1.15	1.00	1.00	1.00	0.98	319.93	2691.00	0.22	16.49	256.50
+D+S					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.20 ft	1	0.695	0.376	1.15	1.000	1.00	1.15	1.00	1.00	1.00	7.35	2,388.79	3438.50	1.62	123.10	327.75
+D+0.750S					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = $16.20 \text{ ft}$	1	0.544	0.294	1.15	1.000	1.00	1.15	1.00	1.00	1.00	5.76	1.871.57	3438.50	1.27	96.45	327.75
+0.60D					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.20 ft	1	0.040	0.022	1.60	1.000	1.00	1.15	1.00	1.00	1.00	0.59	191.96	4784.00	0.13	9.89	456.00

Title Block Line 1 You can change this area using the "Settings" menu item and then using the "Printing & Title Block" selection, Title Block Line 6

Project Title: Engineer: Project ID: Project Descr:

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Wood Beam

File = Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\Other\18121.ec6

Lic. #: KW-06002489 Description:

Activity Room Joists at Roof Hoods

Licensee : ARW ENGINEERS

**CODE RÉFERENCES** 

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

Load Combination Set: ASCE 7-10

**Material Properties** 

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

Wood Species : Trus Joist : MicroLam LVL 2.0 E Wood Grade

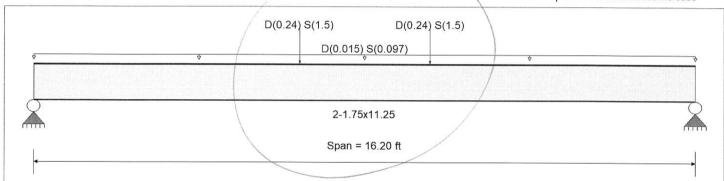
: Beam is Fully Braced against lateral-torsional buckling Beam Bracing

2,600.0 psi E: Modulus of Elasticity 2,600.0 psi

2,000.0 ksi Ebend-xx

Eminbend - xx 1,016.54ksi

42.0 pcf Density Repetitive Member Stress Increase



Fb+

Fb -

Fc - Prll

Fc - Perp

2,510.0 psi

750.0 psi

285.0 psi

1,555.0 psi

### **Applied Loads**

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

=

=

Uniform Load: D = 0.0150, S = 0.0970 ksf, Tributary Width = 1.0 ft

=

Point Load: D = 0.240, S = 1.50 k @ 6.50 ft

Point Load: D = 0.240, S = 1.50 k @ 9.70 ft

### **DESIGN SUMMARY**

Maximum Bending Stress Ratio Section used for this span fb : Actual FB: Allowable Load Combination Location of maximum on span Span # where maximum occurs

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

0.708 1 2-1.75x11.25 2,435.52 psi 3,438.50 psi

+D+S

8.100ft 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

0.708 in Ratio = 274 >=240 0.000 in Ratio = 0 < 240 0.820 in Ratio = 237 >= 180 0.000 in Ratio = 0<180

0.296:1 2-1.75x11.25 97.06 psi 327.75 psi

Design OK

15.313 ft Span #1

+D+S

Maximum Forces & Stresses for Load Combinations

Load Combination									Mor	ment Values	Shear Values					
Segment Length	Span #	M	V	$C_d$	C F/V	Ci	$c_{r}$	$C_{m}$	$C_t$	C <sup>L</sup>	М	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 16.20 ft	1	0.124	0.052	0.90	1.000	1.00	1.15	1.00	1.00	1.00	2.05	333.54	2691.00	0.35	13.26	256.50
+D+S					1.000	1.00	1.15	1.00	1.00	1.00		100,7.5.550.0	0.00	0.00	0.00	0.00
Length = 16.20 ft	1	0.708	0.296	1.15	1.000	1.00	1.15	1.00	1.00	1.00	14.98	2,435.52	3438.50	2.55	97.06	327.75
+D+0.750S					1.000	1.00	1.15	1.00	1.00	1.00	10115.5	_,	0.00	0.00	0.00	0.00

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Wood Roam

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02489					COLUMN TO SERVICE							Licens	ee : AR	W ENGI	NEERS
Activity Roo	m Joists at	Roof Hoo	ds												
	Max Stres	s Ratios								Mor	ment Values			Shear Va	lues
Span #	M	V	$C_d$	C <sub>FN</sub>	Ci	$c_{r}$	$C_{m}$	$c_t$	C <sub>L</sub> -	М	fb	F'b	V	fv	F'v
1	0.555	0.232	1.15	1.000	1.00	1.15	1.00	1.00	1.00	11.75	1,910.03	3438.50	2.00	76.11	327.75
				1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
1	0.042	0.017	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.23	200.13	4784.00	0.21	7.96	456.00
imum De	eflectio	ns													
	5	Span	Max. "-"	Defl	Location	n in Span		Load Co	mbinatio	on		Max. "+"	Defl L	ocation in	Span
		1	8.0	3198		8.159						0.0	000	0.0	000
ctions						Sup	port no	tation : F	ar left is	#1		Values in K	IPS		
				Suppor	t1 Su	pport 2									
				2.6	647	2.647									
				2.2	286	2.286									
				0.3	62	0.362									
				2.6	647	2.647									
				2.0	176	2.076									
				0.2	217	0.217									
				2.2	286	2.286									
	Span #  1  imum De	22489  Activity Room Joists at    Max Stress   M	Activity Room Joists at Roof Hoo    Activity Room Joists at Roof Hoo   Span #   Max Stress Ratios     Max Stress Ratios     M	Activity Room Joists at Roof Hoods     Cd     Activity Room Joists at Roof Hoods     Cd     Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Cd   Activity Room Joists at Roof Hoods     Activity Room Joists at	Activity Room Joists at Roof Hoods   Span #   Max Stress Ratios   No.555   0.232   1.15   1.000   1.	Activity Room Joists at Roof Hoods     Activity Room Joists Ratios     Activity Room Joists Ratios   Activity Room Joists Ratios   Activity Room Joists Ratios   Activity Room Joists Ratios   Activity Room Joists Ratios   Activity Room Joists Ratios   Activity Room Joists Ratios   Activity Room Joists Ratios   Activity Room Joists at Roof Hoods   Activity Room Joists Ratios   Activit	Activity Room Joists at Roof Hoods    Max Stress Ratios   Span #   M   V   C d   C F/V   C i   C r	Activity Room Joists at Roof Hoods    Max Stress Ratios   No.   No	Activity Room Joists at Roof Hoods    Max Stress Ratios   Span #   M   V   C d   C   FW   C i   C r   C m   C t	Activity Room Joists at Roof Hoods    Max Stress Ratios	Activity Room Joists at Roof Hoods    Max Stress Ratios   Span # M V Cd CFN Ci Cr Cm Cd Cd N	Activity Room Joists at Roof Hoods    Max Stress Ratios   Span #   M   V   C   C   C   C   C   C   C   M   C   C	Activity Room Joists at Roof Hoods    Max Stress Ratios   Span # M	Activity Room Joists at Roof Hoods    Max Stress Ratios   Span # M   V   Cd   C   FN   C   Cr   C   Cr   C   M   fb   F'b   V	Activity Room Joists at Roof Hoods   Shear Values   Shear Values

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Project ID:

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**Wood Beam** 

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ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.10

Licensee: ARW ENGINEERS

Lic. # : KW-06002489

Description: Activity Room Joists

eflections					
Span	Max. "-" Defl l	ocation in Span	Load Combination	Max. "+" Defl	Location in Spar
1	0.8408	8.159		0.0000	0.000
		Suppor	t notation : Far left is #1	Values in KIPS	
	Support '	1 Support 2			
	1.81	4 1.814			
	1.57	1 1.571			
	0.24	3 0.243			
	1.81	4 1.814			
	1.42	2 1.422			
	0.14	6 0.146			
	1.57	1 1.571			
	25.0	Span Max. "-" Defl I  1 0.8408  Support 1.81 1.57 0.24 1.81 1.42 0.14	Span         Max. "-" Deft         Location in Span           1         0.8408         8.159           Support           Support 1         Support 2           1.814         1.814           1.571         1.571           0.243         0.243           1.814         1.814           1.422         1.422           0.146         0.146	Span         Max. "-" Defl         Location in Span         Load Combination           1         0.8408         8.159           Support notation : Far left is #1           Support 1           Support 2         1.814           1.571         1.571           0.243         0.243           1.814         1.814           1.422         1.422           0.146         0.146	Span         Max. "-" Defl         Location in Span         Load Combination         Max. "+" Defl           1         0.8408         8.159         0.0000           Support notation : Far left is #1         Values in KIPS           Support 1         Support 2           1.814         1.814           1.571         1.571           0.243         0.243           1.814         1.814           1.422         1.422           0.146         0.146

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Project Title: Engineer: Project ID: Project Descr:

Title Block Line 6

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Design OK

### **Wood Beam**

File = Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\Other\18121.ec6

Lic. # : KW-06002489

Description:

Beam supporting windows above overhead doors

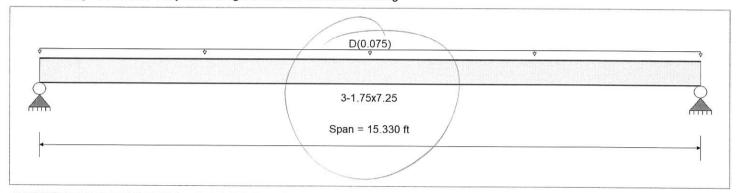
### CODE REFERENCES

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 Elas	ticity
Load Combination ASCE 7-10	Fb -	2,600.0 psi	Ebend- xx	2,000.0ksi
	Fc - Prll	2,510.0 psi	Eminbend - xx	1,016.54ksi
Wood Species : Trus Joist	Fc - Perp	750.0 psi		
Wood Grade : MicroLam LVL 2.0 E	Fv	285.0 psi		
	Ft	1,555.0 psi	Density	42.0 pcf
Beam Bracing : Beam is Fully Braced against lateral-torsion	nal buckling	Control of the contro		por



### **Applied Loads**

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

Beam self weight calculated and added to loads Uniform Load: D = 0.0750, Tributary Width = 1.0 ft

DES	IGN	I SU	<i>IMM</i>	ARY

Maximum Bending Stress Ratio Section used for this span fb : Actual FB : Allowable	= (	7 0.282 1 Ma 3-1.75x7.25 659.94psi 2,340.00psi	ximum Shear Stress Ratio Section used for this span fv : Actual Fv : Allowable	= = =	0.094 : 1 3-1.75x7.25 24.11 psi 256.50 psi
Load Combination Location of maximum on span Span # where maximum occurs	= =	D Only 7.665ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	= =	D Only 0.000 ft Span #1
Maximum Deflection Max Downward Transient Deflectio Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	n	0.000 in Ratio = 0.000 in Ratio = 0.323 in Ratio = 0.000 in Ratio =	0 <240 0 <240 569 >= 180 0 <180		

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stres	s Ratios							8	Morr	ent Values			Shear Va	lues
Segment Length	Span #	M	V	$C_d$	C F/V	Ci	$c_{r}$	$C_{m}$	Ct	CL _	М	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 15.330 ft	1	0.282	0.094	0.90	1.000	1.00	1.00	1.00	1.00	1.00	2.53	659.94	2340.00	0.61	24.11	256.50
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00		000.01	0.00	0.00	0.00	0.00
Length = 15.330 ft	1	0.095	0.032	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.52	395.96	4160.00	0.37	14.47	456.00
<b>Overall Maxin</b>	num De	eflectio	ns									000.00		0.07	11.17	450.00
Load Combination		S	pan	Max. "-"	' Defl	Location	n in Span		oad Co	mbination	1		Max "+"	Defl I	ocation in	Span

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D Only	1	0.3228	7.721		0.0000	0.000

Title Block Line 1 You can change this area using the "Settings" menu item and then using the "Printing & Title Block selection.
Title Block Line 6

Project Title: Engineer: Project ID: Project Descr:

Printed: 8 AUG 2018, 9:37AM

**Wood Beam** 

File = Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\Other\18121.ec6

Lic. #: KW-06002489 Description: Beam supporting windows above overhead doors

Licensee : ARW ENGINEERS

Vertical Reactions		Support notation : Far left is #1	Values in KIPS
Load Combination	Support 1	Support 2	
Overall MAXimum	0.660	0.660	
Overall MINimum	0.396	0.396	
D Only	0.660	0.660	
+0.60D	0.396	0.396	

Title Block Line 1
You can change this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.

Project Title: Engineer: Project ID: Project Descr:

Printed: 8 AUG 2018, 12:42PM

Title Block Line 6

Steel Beam

File = Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\Other\18121.ec6

Lic. # : KW-06002489

Description :/ Angl

Angle supporting windows above overhead doors

### **CODE REFERENCES**

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

### **Material Properties**

Analysis Method: Allowable Strength Design

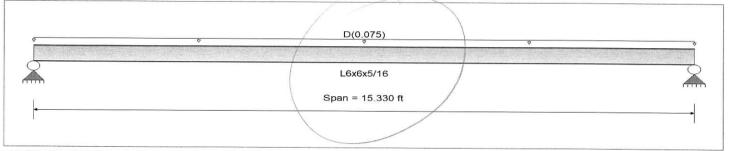
Beam Bracing: Beam is Fully Braced against lateral-torsional buckling

Bending Axis: Major Axis Bending

Fy: Steel Yield: E: Modulus: 36.0 ksi

Design OK

29,000.0 ksi



### **Applied Loads**

+0.60D

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

Beam self weight calculated and added to loading

Uniform Load: D = 0.0750 k/ft, Tributary Width = 1.0 ft

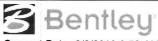
0.402

0.402

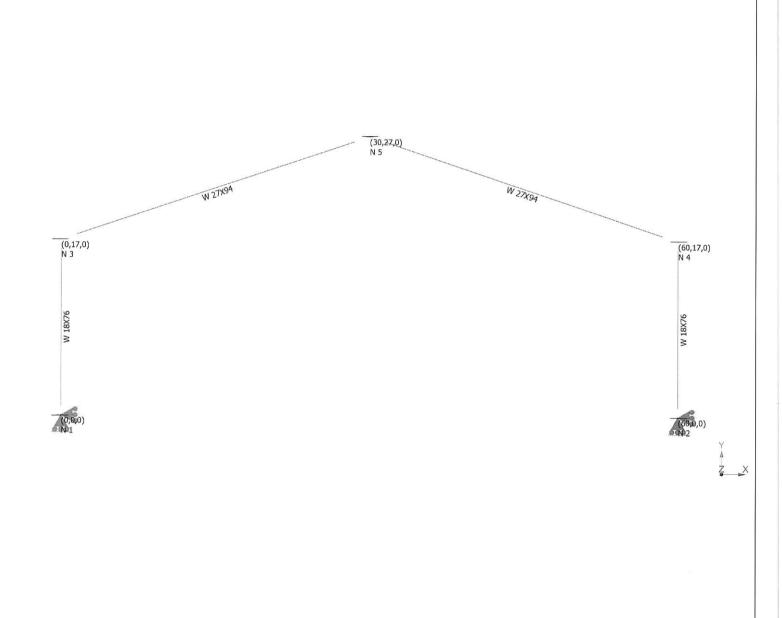
DES	ICN	CI	INA	N/I	AL	VC
DES	IGIY	၁ပ	JIVI	IVI	нr	17

	The control of the co		Design On
Maximum Bending Stress Ratio = Section used for this span Ma : Applied Mn / Omega : Allowable	0.382: 1 Maxin <b>L6x6x5/16</b> 2.567 k-ft 6.722 k-ft	num Shear Stress Ratio = Section used for this span Va : Applied Vn/Omega : Allowable	0.028 : 1 <b>L6x6x5/16</b> 0.6699 k 24.252 k
Load Combination Location of maximum on span Span # where maximum occurs	D Only 7.665ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	D Only 0.000 ft Span # 1
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection Max Upward Total Deflection	0.000 in Ratio = 0.000 in Ratio = 0.289 in Ratio = 0.000 in Ratio =	0 < 360 0 < 360 636 >= 180 0 < 180	

Load Combination		Max Stres	s Ratios		(	Summary of Mo	oment Valu	ies			Sumn	nary of Sh	ear Values
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Only									s score-	PERMIT		2000000	3-
Dsgn. L = 15.29 ft	1	0.382	0.028	2.57		2.57	11.23	6.72	1.00	1.00	0.67	40.50	24.25
Dsgn. L = 0.04 ft	1	0.004	0.028	0.03		0.03	11.23	6.72	1.00	1.00	0.67	40.50	24.25
+0.60D							107.00000000000000000000000000000000000			,,,,,	0.07	10.00	24.20
Dsgn. L = 15.29 ft	1	0.229	0.017	1.54		1.54	11.23	6.72	1.00	1.00	0.40	40.50	24.25
Dsgn. L = 0.04 ft	1	0.003	0.017	0.02		0.02	11.23	6.72	1.00	1.00	0.40	40.50	24.25
<b>Overall Maximum</b>	Deflec	tions										35.5	
Load Combination		Span	Max. "-" Defl	Location	n in Span	Load Comb	oination			Max	. "+" Defl	Location	n in Span
D Only		1	0.2894		7.709						0.0000		0.000
Vertical Reactions	S				Support	notation : Far I	eft is #1			Values in	KIPS		
Load Combination		Support 1	Support 2										
Overall MAXimum		0.670	0.670										
Overall MINimum		0.402	0.402										
D Only		0.670	0.670										



Current Date: 8/2/2018 9:52 AM
Units system: English
File name: Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\RAM\18121 - Activity Room Frame.etz\



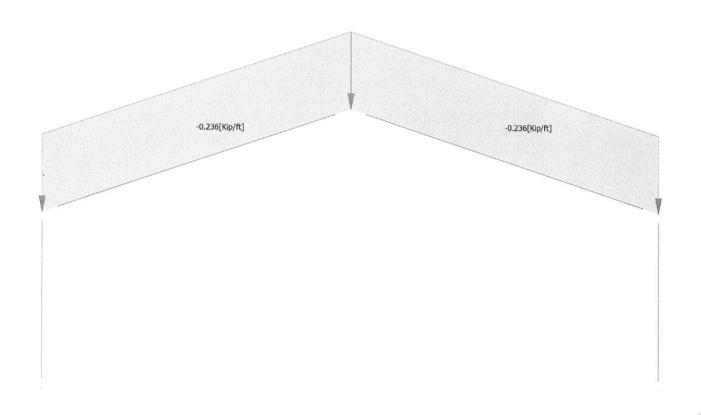




Current Date: 8/2/2018 9:53 AM
Units system: English
File name: Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\RAM\18121 - Activity Room Frame.etz\
Load condition: DL=Dead Load



Distributed user loads - Members



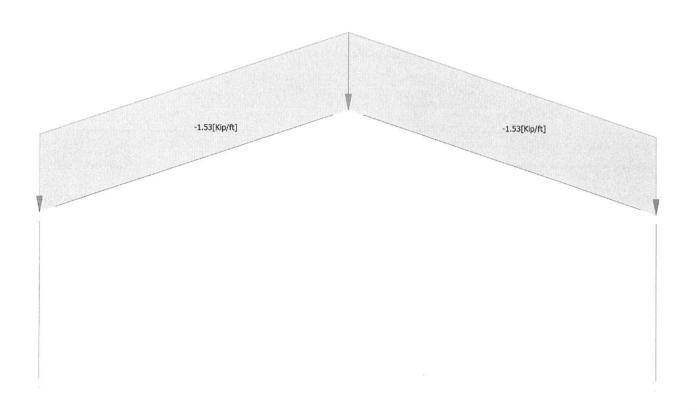




Current Date: 8/2/2018 9:53 AM
Units system: English
File name: Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\RAM\18121 - Activity Room Frame.etz\
Load condition: SL=Snow Load

### Loads

Distributed user loads - Members





Current Date: 8/9/2018 9:38 AM
Units system: English
File name: Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\RAM\18121 - Activity Room Frame.etz\
Load condition: W=Wind Load C&C

Loads

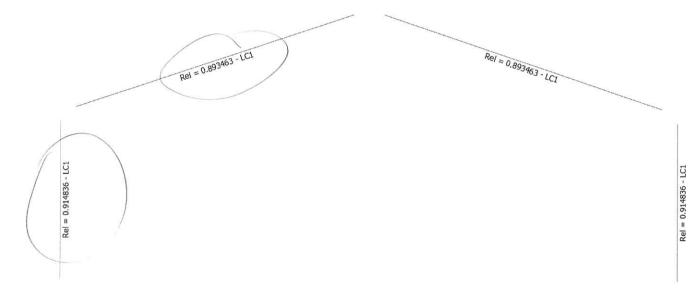
Distributed user loads - Members







Current Date: 8/2/2018 9:53 AM
Units system: English
File name: Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\RAM\18121 - Activity Room Frame.etz\







Current Date: 8/3/2018 1:47 PM
Units system: English
File name: Y.\Projects 2018\text{18121 - NAC Recreation Center\text{Engineering\text{Calculations\text{\text{\text{RAM\text{\tilde{\text{\texi}\text{\text{\texi}\text{\text{\text{\text{\texi{\text{\texi{\texi{\texi{\text{\texitex{\text{\tex{

### Steel Code Check

Report: Concise

Members: Hot-rolled Design code: AISC 360-2010 ASD

1 (column) OK Member Design status

Section information

Section name: W 18X76 (US) Dimensions

Depth
Distance k
Distance k1
Flange thickness
Web thickness EEEEE 11.000 18.200 1.080 1.063 0.680 0.425

### Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	22.300	
Moment of Inertia (local axes) (I)	[in4]	1330.000	152.000
Moment of Inertia (principal axes) (I')	[in4]	1330,000	152.000
Bending constant for moments (principal axis) (J')	[ij]	0.000	0.000
Radius of gyration (local axes) (r)	Ξ	7.723	2.611
Radius of gyration (principal axes) (r')	Ξ	7.723	2.611
Saint-Venant torsion constant. (J)	[in4]	2.830	
Section warping constant. (Cw)	[jue]	11700.000	
Distance from centroid to shear center (principal axis) (xo,yo)	Ξ	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	146.000	27.600
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	146.000	27.600
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	146.000	27.600
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	146.000	27,600
Plastic section modulus (local axis) (Z)	[in3]	163.000	42.200
Plastic section modulus (principal axis) (Z')	[in3]	163.000	42.200
Polar radius of gyration. (ro)	[u]	8.152	
Area for shear (Aw)	[in2]	14,960	7.740
Torsional constant. (C)	[in3]	4.050	

Material: A992 Gr50

Properties	Unit	Value	
Yield stress (Fy):	[Kip/in2]	50.00	
Tensile strength (Fu):	[Kip/in2]	65.00	
Elasticity Modulus (E):	[Kip/in2]	29000.00	
Shear modulus for steel (G):	[Kip/in2]	11153.85	

DESIGN CRITERIA

Value 17.00 Unit Œ Length for tension slenderness ratio (L) Description

Distance between member lateral bracing points

Length (Lb) [ft] Top Bottom

rally u	ed length				
Major axis(L33)	Length [ft] Minor axis(L22)	Langth (f) Effective length factor Effective length factor L33) Minor axis(K22) Minor axis(K22)	Major axis(K33)	Effective length factor Minor axis(K22)	Torsional axis(Kt)
17.00	17.00	17.00 1.0 17.00 1.0 1.0 1.0	1.0	1.0	1.0

Additional assumptions
Continuous lateral into account of the state of

No No None Sway Sway

DESIGN CHECKS

AXIAL TENSION DESIGN

Axial tension

Ratio Capacity Demand

Reference Ctrl Eq. 0.00 667.66 [Kip] 0.00 [Kip]

: Eq. Sec. D2 : LC1 at 0.00%

99'299 Value Unit [Kip] Factored axial tension capacity(Pn/Ω) Intermediate results

Eq. Sec. D2 Reference

AXIAL COMPRESSION DESIGN

Compression in the major axis 33

Page2

Capacity 630.24 (Kip) Demand 55.85 (Kip)	Reference Ctrl Eq.	: Sec. E1	0.00%	Intermediate results		Unit
				Factored shear capacity(Vn/ $\Omega$ )		[Kip]
Intermediate results	Unit	Value	Reference			
Section classification Factored flexural buckling strength(Pn33/ $\Omega$ )	[Kip]	630.24	Sec. E1	ixis 22		
Compression in the minor axis 22				Capacity : 154.80 [Kip] Demand : -20.83 [Kip]		Reference Ctrl Eq.
Ratio : 0.13 Canacity : 427.25 (Kin)	Reference	ш.		Intermediate results		Unit
Demand : 55.85 [Kip]	Ctrl Eq.	: LC1 at 0.00%	0.00%	Factored shear capacity( $V_n/\Omega$ )		[Kip]
Intermediate results	Unit	Value	Reference	COMBINED ACTIONS DESIGN	>	
Section classification Factored flexural buckling strength(Pnzz/ $\Omega$ )	[Kip]	427.25	Sec. E1	Combined flexure and axial compression		
Factored torsional or flexural-torsional buckling strength(Pn11/Ω)	[Kip]	506.21	Sec. E4	Ratio : 0.91 Ctrl Eq. : LC1 at 100.00%	% Reference	
FLEXURAL DESIGN						
Bending about major axis, M33				Intermediate results		Unit
				Interaction for doubly symmetric members for in-plane bending		/
Capacity : 406.69 [Kip*ft]	Reference	Sec. F	. Sec. F1	Interaction for doubly symmetric members for out-of-plane bending		_
	Ctrl Eq.	: LC1 at	100.00%			/
Intermediate results	Unit	Value	Reference	xure and axial ten		
Section classification				Ctrl Eq. ( : LC1 at 100.00%	Reference	ш
Eactored vielding strength(Mn/Ω) Factored lateral-torsional buckling strength(Mn/Ω)	[Kip*ft]	406.69	Sec. F1			
701			***************************************	Intermediate results	_	Unit
perionic about milled axis, with						
Ratio : 0.00 Capacity : 105.29 [Kip*ft]	Reference	Sec. F		Combined flexure and axial compression about local axis	rt local axis	
Demand : 0.00 [Kip*ft]	Ctrl Eq.	: LC1 at 0.00%	%00.0	Ratio : N/A Crif Fo	Reference	
Intermediate results	Unit	Value	Reference			
Section classification Fartness viability eterosth MA-(O)	[5]	20	Ti de de la companya	Combined flexure and axial tension about local axis		
- CANDON TORRING STRUCKET (ALL)	Pr distri	67.001		Ratio : N/A Ctrl Eq. :	Reference	
DESIGN FOR SHEAR   ✓						
Shear in major axis 33					Î	

Reference

: Eq. H1-1b

Eq. H1-1b Eq. H1-2

Value 0.91 0.46 Reference

Value

: Eq. H1-1b

Sec. G2.1(a)

Value 154.80

Reference

: Sec. G2.1(a) : LC1 at 0.00%

Reference

Value 268.74

Sec. G1

Page4

: 2 (column) : OK

Member Design status

> : Sec. G1 : LC1 at 0.00%

> Reference Ctrl Eq.

: 0.00 : 268.74 [Kip] : 0.00 [Kip]

Ratio Capacity Demand

### Section information

Section name: W 18X76 (US)

Dimensions

	Width	Depth	Distance k	Distance k1	Flange thickness	Web thickness
	Ξ	[i]	[ii]	Ξ	Ξ	<u>[ii]</u>
	11.000	18.200	1.080	1.063	0.680	0.425
† † † † † † † † † † † † † † † † † † †	2 11	ш	П	u	11	11
11- 2	þ	D	×	Z	Ħ	2

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	22.300	
Moment of Inertia (local axes) (I)	[in4]	1330.000	152.000
Moment of Inertia (principal axes) (I')	[in4]	1330.000	152.000
Bending constant for moments (principal axis) (J')	[ii]	0.000	0.000
Radius of gyration (local axes) (r)	[i]	7.723	2.611
Radius of gyration (principal axes) (r')	Ξ	7.723	2.611
Saint-Venant torsion constant. (J)	[in4]	2.830	
Section warping constant. (Cw)	[in6]	11700.000	
Distance from centroid to shear center (principal axis) (xo,yo)	<u>E</u>	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	146.000	27.600
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	146.000	27.600
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	146.000	27.600
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	146.000	27.600
Plastic section modulus (local axis) (Z)	[ju3]	163.000	42.200
Plastic section modulus (principal axis) (Z')	[in3]	163.000	42.200
Polar radius of gyration. (ro)	Ξ	8.152	
Area for shear (Aw)	[in2]	14.960	7.740
Torsional constant. (C)	[in3]	4.050	
Material : A992 Gr50			
Properties	Unit	Value	
Yield stress (Fy):	[Kip/in2]	50.00	
Tensile strength (Fu):	[Kip/in2]	65.00	
Elasticity Modulus (E):	[Kip/in2]	29000.00	
Shoot modulus for along (O).			

DESIGN CRITERIA
Description

Value	17.00	
Onit	[4]	
Description	Length for tension slenderness ratio (L)	

Page5

Distance between member lateral bracing points

	Bottom				
17.00	17.00				
Laterally unbraced length	d length				
Major axis(L33)	Length [ft] Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Effective length factor Minor axis(K22)	actor (x) Torsional axis(Kt)
17.00	17.00	17.00	1.0	1.0	1.0
Additional assumptions	ptions			į	
Tension field action	ioisional resulatin			8 8 8	
Continuous flexural torsional restraint Effective length factor value type	I torsional restraint			ON ON	
Major axis frame type Minor axis frame type	ed o			Sway	
DESIGN CHECKS					
AXIAL TENSION DESIGN	ESIGN	>			
Axial tension					
Ratio Capacity Demand	0.00 : 667.66 [Kip] : 0.00 [Kip]	(lp) (lp)	Reference Ctrl Eq.	: Eq. Sec. D2 : LC1 at 0.00%	. D2 0.00%
Intermediate results	ts		Unit	Value	Reference
Factored axial tension capacity(Pn/ $\Omega$ )	ion capacity(Pn/Ω)		[Kip]	99'.299	Eq. Sec. D2
AXIAL COMPRESSION DESIGN	SION DESIGN	>			
Compression in the major axis 33	ne major axis 33				
Ratio	60.00				
Capacity Demand	: 630.24 [Kip] : 55.85 [Kip]		Reference Ctrl Eq.	: Sec. E1 : LC1 at 0.00%	%00°
Intermediate results	ts		Unit	Value	Reference
Section classification Factored flexural buck	Section classification Factored flexural buckling strength(Pn33/ $\Omega$ )	D)	[Kip]	630.24	Sec. E1

Page6

: Sec. E1 : LC1 at 0.00%

Reference Ctrl Eq.

: 0.13 : 427.25 [Kip] : 55.85 [Kip]

Ratio Capacity Demand

		Tiu0	Value	Reference
Section classification Factored flexural buckling strength(Pnzz/ $\Omega_{ m j}$ Factored flexural buckling strength(Pnzz/ $\Omega_{ m j}$	Section classification Factored flexural buckling strength(Pnzs/Ω) Factored torsional or flexural-torsional buckling strength(Pn11/Ω)	Z Z Z	427.25 506.21	Sec. E1
FLEXURAL DESIGN	>			
Bending about major axis, M33	81			
Ratio Capacity : A Demand : 3	0.87 406.69 [Kip*ti] 354.03 [Kip*fi]	Reference Ctrl Eq.	Sec. F	: Sec. F1 : LC1 at 100.00%
Intermediate results		Unit	Value	Reference
Section classification Factored vielding strength(Mn/S2) Factored lateral-torsional buckling strength(Mn/S2)	) <u>og strength(</u> Mn/Ω)	[Kip*ft] [Kip*ft]	406.69	Sec. F1
Bending about minor axis, M22	2			
	0.00 105.29 [Kip*t]	Reference	Sec. F1	-
	o.co (kip rij	Ctrr Eq.	[C1	00.00 1
Intermediate results		Unit	Value	Reference
Section classification Factored yielding strength(Mn/ $\Omega$ )		[Kip*ft]	105.29	Sec. F1
DESIGN FOR SHEAR	>			
Shear in major axis 33				
Ratio : Capacity : 2 Demand :	0.00 268.74 [Kip] 0.00 [Kip]	Reference Ctrl Eq.	: Sec. G1 : LC1 at 0.00%	0.00%
Intermediate results		Unit	Value	Reference
Factored shear capacity(Vn/Ω)		[Kip]	268.74	Sec. G1
Shear in minor axis 22				
Ratio Capacity 1	0.13 154.80 [K[p] 20.83 [K[p]	Reference Ctrl Eq.	: Sec. G2.1(a) : LC1 at 0.00%	2.1(a) 0.00%
Intermediate results		Unit	Value	Reference
Factored shear capacity(Vn/Ω)		[Kip]	154.80	Sec. G2.1(a)

COMBINED ACTIONS DESIGN

Combined flexure and axial compression

	1001	,000		L	
Cull Eq. : LC1 at 100,00% Reference : Eq. H1-1b	LC1 at 100.00%	, JU%o	Keterence	: Eq. H1-1b	
Intermediate results			Unit	Value	Reference
Interaction for doubly symmetric members for in-plane bending	c members for i	n-plane bending	,	0.91	Eq. H1-1b
Interaction for doubly symmetric members for out-of-plane bending	c members for o	out-of-plane bending	1	0.46	Eq. H1-2
Ratio : 0.87	0.87				
Otrl Eq.	LC1 at 100.00%	%00	Reference	: Eq. H1-1b	

Combined flexure and axial compression about local axis

Ctrl Eq.	***		Reference	
Combined flexure	nd axial	Combined flexure and axial tension about local axis		
***************************************				
Ratio		N/A		
Ctrl Eq.		1	Reference	

: 3 (beam) : OK Member Design status Section information

Width Depth Distance k Distance k1 Flange thickness Web thickness Section name: W 27X94 (US) Dimensions

Page8

Initial State	oss area of the section. Iment of Inertia (local axi Iment of Inertia (principa Inding constant for momi riding of gyration (local axi	(Ag)	[Cuil	111111	
Inition   3270 000	ument of Inertia (local axe ument of Inertia (principa ending constant for mom- idius of gyration (local axidius of gyration)		ZIII.	27.700	
Initial   3270 000	ument of Inertia (principal anding constant for mome idius of gyration (local ax	()) (sa)	[in4]	3270.000	124.000
Transport (principal axe) (J)   (In)   10,805	nding constant for mome	axes) (l)	lin41	3270.000	124.000
10   10   10   10   10   10   10   10	idius of gyration (local ax	onts (oringinal axis) (.l.)	E	000 0	0000
Initial continuity of the content (principal axis) (γ)   Initial content (γ)   Initial	an incol Houself & con	(a) (b)	Ξ	10.865	2 116
International Content	dine of aveation (princips	(1) (2)	[ ]	10.865	2110
Trick   Action	ands or gyration (principle	il aves) (i )	E	10.800	2.110
Implement	IIII-verialit totsion constr	(a)	£ 3	4.030	
In   00000   00000   00000   00000   0000   0000   0000   0000   0000   0000   0000   0000   0000	ction warping constant.	(Cw)	[gu]	21300.000	
In 3	stance from centroid to s	hear center (principal axis) (xo,yo)	Ξ	0.000	0.000
on modulus of the section (local axis) (Sinf) [in3] 243 000  modulus of the section (principal axis) (Sinf) [in3] 243 000  modulus of the section (principal axis) (Sinf) [in3] 243 000  modulus (or axis) (2) [in3] 243 000  mults (or axis) (2) [in3] 278 000  mults (or axis) (2) (2) (2) (2) (2) (2) (2) (2) (2) (2	p elastic section modulu	s of the section (local axis) (Ssup)	[in3]	243.000	24.800
The section (principal axis) (S'sup)   [in3] 243.000   [in3] 243.000   [in3] 278.000   [in3]	ttom elastic section mod	ulus of the section (local axis) (Sinf)	[in3]	243.000	24.800
10   10   10   10   10   10   10   10	p elastic section modulu:	s of the section (principal axis) (S'sup)	[in3]	243.000	24.800
1   1   1   1   1   1   1   1   1   1	ttom elastic section mod	ulus of the section (principal axis) (S'in		243 000	24 800
(c)	etic section modulus (lo	cal axis) (7)		278 000	000
(C)	Charles and the second	(Z) (Z)		20000	00000
(C)   (III)   11.089   (III)   11.089   (III)   14.900   (III)   14.900   (III)   14.900   (III)   14.900   (III)   14.900   (III)   14.900   (III)   (III)   14.900   (III)   (III)   14.900   (III)   (III)   14.900   (III)   (II	istic section modulus (pi	incipal axis) (Z)	[[2]	278.000	38.800
(C) [in2] 14,900 (C) [in3] 5,077 (D) [in3] 5,077 (D) [in3] 14,900 (D) [in3] 5,077 (E) [in3] 5,077 (E) [in3] 5,077 (E) [in4] 50,000 (E) [in5] 11153.85 (E) [in4] 11153.85 (E) [in5] 11153	lar radius of gyration. (n	6	E	11.069	
(5) [in3] 5.077 (6) Unit Value  Unit Value    Kip/in2  55.00   Kip/in2  55.00   Kip/in2  11153.85   Kip/in	ea for shear (Aw)		[in2]	14.900	13.180
Unit   Value	rsional constant. (C)		[in3]	5.077	
Unit   Value	terial : A992 Gr50				
Unit   Value					
	operties		Unit	Value	
	ild etrace (Ev):		[Kin/in]	50.00	
	notice of the county (Fig.):		[Zilliqivi]	00.00	
	issie stierigui (Fu).		[Nipriiz]	99.00	
Helpin	isticity modulus (E):		[Kip/in2]	29000.00	
Unit   Value	ear modulus for steel (G		[Kip/in2]	11153.85	
(ft) 31.62	scription		Unit	Value	
racing points  racing points  racing points  racing points  racing points    In					
Torsional axis(Lt) Major axis(K33) Minor axis(K22)  31.62 1.0 1.0  No No None Sway Sway Sway Sway Sway Sway Sway	igtn for tension siendern	ess ratio (L)	Ē	31.62	
Torsional axis(Lt) Major axis(K33) Minor axis(K22) 31.62 1.0 1.0 No	tance between membe	r lateral bracing points			
Torsional axis(Lt) Major axis(K33) Minor axis(K22)  31.62 1.0 1.0  No	l enoth (I b) (#1				
Torsional axis(Lt) Major axis(K33) Minor axis(K22) 31.62 1.0 1.0 No	וואווו (רים) וו	E			
Torsional axis(Lt) Major axis(K33) Minor axis(K22) 31.62 1.0 1.0 No No None Sway		2			
Torsional axis(Lt) Major axis(K33) Minor axis(K22) 31.62 1.0 1.0 No No None Sway Sway					
Effective length factor Torsional axis(L1) Major axis(K33) Minor axis(K22) 31.62 1.0 1.0 No No No None Sway	terally unbraced length				
Torsional axis(Lt) Major axis(K23) Minor axis(K22) 31.62 1.0 1.0 No	Len	oth (fil)		Effective length factor	
3162 1.0 1.0 No			Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
2 6 6			1.0	1.0	1.0
Z Ø Ø	ditional assumptions	restrain		Š	
Z Ø Ø	usion field action			2	
200	ationic flexinal torsions	Inestraint		S N	
	minuous nexural rol sions	il resugnit		ON	
	ective length factor value	adái		None	
	or axis frame type			Oway	
	Minor axis frame type			Sway	

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Properties

>		00'0 :	: 829.34 [Kip] Reference : Eq. Sec. D2	Ctrl Eq.	Intermediate results Seference
Z			: 82		
AXIAL TENSION DESIGN	Axial tension	Ratio	Capacity	Demand	Intermediate results

Eq. Sec. D2

829.34

[Kip]

Factored axial tension capacity(Pn/Ω)

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(Pn33/Ω)	[Kip]	701.52	Sec. E1

: Sec. E1 : LC1 at 0.00%

Reference Ctrl Eq.

. 0.05 : 701.52 [Kip] : 37.42 [Kip]

Ratio Capacity Demand

Compression in the major axis 33 AXIAL COMPRESSION DESIGN

Section classification Factored flexural buckling strength (Pn33/)	$\Omega_{ m Ming}  { m strength}({ m Pn}_33/\Omega)$	[Kip]	701.52	Sec. E1
Compression in the minor axis 22	or axis 22			
Ratio	0.29			
Capacity	: 129.43 [Kip]	Reference	Sec. E	E1
Demand	: 37.42 [Kip]	Ctrl Eq.	: LC1 a	LC1 at 0.00%

ntermediate results	Unit	Value	Reference
Section classification			
actored flexural buckling strength(Pn22 $\Omega$ )	[Kip]	129.43	Sec. E1
Factored torsional or flexural-torsional buckling strength(Pn111 $\Omega$ )	[Kip]	367.56	Sec. E4

Factored flexural buckling strength (Pn22/ $\Omega$ ) Factored torsional or flexural-torsional buckling strength (Pn11/ $\Omega$ )	ling strer	orsional b	γΩ) uckling str	ength(Pn11/Ω)	[Kip]	129.43	129.43 Sec. E1 367.56 Sec. E4
FLEXURAL DESIGN			,				
Bending about major axis, M33	r axis, M	33					
Ratio		0.87					
Capacity	- 12	408.44 [Kip*ft]	Kip*ft]		Reference	-69	Sec. F1
Domand	8	254 N3 [Kin*#1	Kin*#1		Ota Fo		101 at 0 00%

termediate results	Unit	Value	Reference
Section classification			
ictored yielding strength(Mn/ $\Omega$ )	[Kip*ft]	693.61	Sec. F1
actored lateral-torsional buckling strength(Mn/ $\Omega$ )	[Kip*ft]	408.44	Sec. F1

## Bending about minor axis, M22

Page 10

c. F1 1 at 0.0 1 at 0.0		267,66 Sec. G1	: Sec. G2.1(a) : LC1 at 0.00%	Value Reference	263.60 Sec. G2.1(a)	: Eq. H1-1b	Value Reference	0.89 Eq. H1-1b
Reference Cirl Eq.  Unit [Kip*ti] Reference Cirl Eq.	Unit	[Kip]	Reference Cirl Eq.	Unit	[Kip]	Reference : Eq. H1-1b	Unit	Interaction for doubly symmetric members for in-plane bending

Reference tension about local axis Α, I

Reference

Section information

					S	
	Width	Depth	Distance k	Distance k1	Flange thickness	Web thickness
	[l]	Ξ	Ē	Ξ	Ξ	Ξ
	10.000	26.900	1.340	1.063	0.745	0.490
֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓	п	Ħ	ш	11	II	п
11+	þĮ	ъ	×	¥	Ħ	¥

		101111111	
Section properties	5	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	27.700	
Moment of Inertia (local axes) (I)	[in4]	3270.000	124.000
Moment of Inertia (principal axes) (I')	[in4]	3270.000	124.000
Bending constant for moments (principal axis) (J')	[i]	0.000	0.000
Radius of gyration (local axes) (r)	[ij]	10.865	2.116
Radius of gyration (principal axes) (r)	[ii]	10.865	2.116
Saint-Venant torsion constant. (J)	[in4]	4.030	
Section warping constant. (Cw)	[in6]	21300.000	
Distance from centroid to shear center (principal axis) (xo,yo)	Ē	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	243.000	24.800
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	243.000	24.800
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	243.000	24.800
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	243.000	24.800
Plastic section modulus (local axis) (Z)	[in3]	278.000	38.800
Plastic section modulus (principal axis) (Z')	[in3]	278.000	38.800
Polar radius of gyration. (ro)	亘	11.069	
Area for shear (Aw)	[in2]	14.900	13.180
Torsional constant. (C)	[in3]	5.077	

Material : A992 Gr50

Reference

Value

Unit

Intermediate results

: Eq. H1-1b

Reference

: 0.87 : LC1 at 0.00%

Ratio Ctrl Eq.

Page12

Properties		Unit	Value			
Yield stress (Fy):		[Kip/in2]	50.00		Intermediate results	Cuit
Tensile strength (Fu): Elasticity Modulus (E): Shear modulus for steel (G):		[Kip/in2] [Kip/in2] [Kip/in2]	65.00 29000.00 11153.85		Section classification Factored flexural buckling strength(Ph33/Ω)	[Kip]
DESIGN CRITERIA					the minor ax	
Description		Unit	Value		Capacity : 129.43 [Kip] Demand : 37.42 [Kip]	Reference Ctrl Eq.
Length for tension slenderness ratio (L)	9	Œ	31.62		Intermediate results	Unit
Length (Lb) (ft)  Top Bottom	suind fi				Section classification Factored flexural buckling strength(Pnzz/Ω)	[Kip]
					Factored torsional or flexural-torsional buckling strength(Pn11/4.2)	[Kip]
Laterally unbraced length					FLEXURAL DESIGN  Bending about major axis. M33	
Length [ft] Major axis(L23) Minor axis(L22) To	Torsional axis(Lt)	Major axis(K33)	Effective length factor Minor axis(K22)	tor Torsional axis(Kt)	Ratio : 0.37 Capacity : 408.44 Kfc/*II	Reference
31.62 31.62	31.62	1.0	1.0	1.0		Ctrl Eq.
Additional assumptions					Intermediate results	Unit
Continuous lateral torsional restraint Tension field action Continuous flexural torsional restraint Effective length factor value type			No N		Section classification Eactored vielding strength ( $M_n(\Omega)$ ) Eactored tateral-torsional buckling strength ( $M_n(\Omega)$ )	[Kip*ft] [Kip*ft]
Major axis frame type Minor axis frame type			Sway Sway		Bending about minor axis, M22	
DESIGN CHECKS AXIAL TENSION DESIGN	>				Ratio : 0.00 Capacity : 96.81 Kip*ft] Demand : 0.00 Kip*ft]	Reference Ctrl Eq.
					Intermediate results	Unit
Ratio : 0.00 Capacity : 829.34 [Kip] Demand : 0.00 [Kip]		Reference Ctrl Eq.	: Eq. Sec. D2 : LC1 at 0.00%	22 00%	Section classification Factored yielding strength(Μκ/Ω)	[Kip*ft]
Intermediate results		Unit	Value	Reference	DESIGN FOR SHEAR	
Factored axial tension capacity(Pn/ $\Omega$ )		[Kip]	829.34	Eq. Sec. D2	Shear in major axis 33	
AXIAL COMPRESSION DESIGN	>				Ratio : 0.00 Capacity : 267.66 [Klp] Demand : 0.00 [Klp]	Reference Ctrl Eq.
in the major axis					Intermediate results	Unit
Katio 0.05 Capacity 701.52 [Kip] Demand 37.42 [Kip]		Reference Ctrl Eq.	: Sec. E1 : LC1 at 0.00%	%01	Factored sitear capacity(יהו'ג)	[Kip]

Reference

Value

Sec. E1

701.52

Reference

Value

: Sec. E1 : LC1 at 0.00%

Sec. E1 Sec. E4

129.43 367.56

Page14

Page13

Reference

Value 267.66

: Sec. G1 : LC1 at 0.00%

Sec. G1

Reference

Value

: Sec. F1 : LC1 at 0.00%

Sec. F1

96.81

Sec. F1 Sec. F1

693.61

Value

: Sec. F1 : LC1 at 0.00%

S	
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near	
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0.18 Ratio

Demand	46.39 [Kip] Ctrl	Ctrl Eq.	I C1 a	LC1 at 0.00%
Intermediate results		Ę	Value	Value Reference
-actored shear capacity(Vn/Ω)		[Kip]	263.60	263.60 Sec. G2.1(a)

## COMBINED ACTIONS DESIGN

>

## Combined flexure and axial compression

ratio	 0.09		
Ctrl Eq.	 LC1 at 0.00%	Reference	: Eq. H1-1b

termediate results	Unit	Value	Reference
Interaction for doubly symmetric members for in-plane bending 0.89 Eq. H1-1b	0.89 Ea.	0.89	Ea. H1-1b
iteraction for doubly symmetric members for out-of-plane bending	1	0.63	Eq. H1-2

## Combined flexure and axial tension

: Eq. H1-1b Reference : 0.87 : LC1 at 0.00% Ratio Ctrl Eq.

Intermediate results	Unit	Value	Reference
		TO CONTROL OF THE PARTY OF THE	

# Combined flexure and axial compression about local axis

עפופופ	

• •	N/A	
	1	Reference



Tx=0.594648[in] Ty=-0.017616[in]

Current Date: 8/2/2018-9:54 AM
Units system: English
File name: Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\RAM\18121 - Activity Room Frame.etz\
Load condition: LC1=DL+SL

4/391 Ty=-1.84426[in] Tx=-0.594648[in] Ty=-0.017616[in]





Current Date: 8/2/2018 9:54-AM
Units system: English
File name: Y:\Projects 2018\18121 -\NAC Recreation Center\Engineering\Calculations\RAM\18121 - Activity Room Frame.etz\
Load condition: SL=Snow Load

450

Ty=-1.5978[in]

Tx=-0.515182[in] Ty=-0.015262[in]

Tx=0.515182[in] Ty=-0.015262[in]



Current Date: 8/2/2018 9:54 AM

Units system: English
File name: Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\RAM\18121 - Activity Room Frame.etz\
Load condition: LC1=DL+SL

Internal forces

Bending moment

M33=275.4[Kip\*ft] M33=275.4[Kip\*ft] M33=288.34[Kip\*ft] M33=288.34[Kip\*ft]

M33=-354.03[Kip\*ft]

M33=-354.03[Kip\*ft]

M33=354.03[Kip\*ft]

M33=-354.03[Kip\*ft]



Current Date: 8/2/2018 9:57 AM

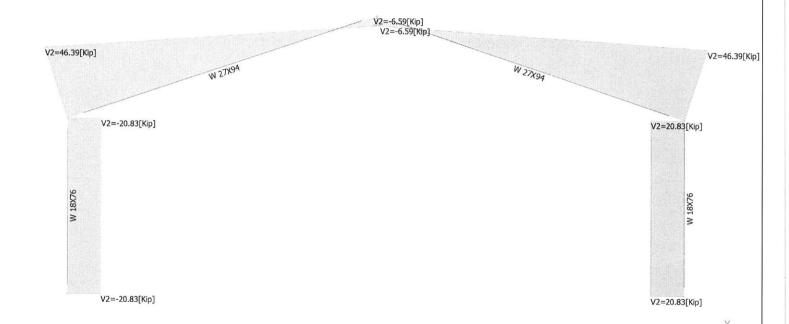
Units system: English

File name: Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\RAM\18121 - Activity Room Frame.etz\

Load condition: LC1=DL+SL



Shear force

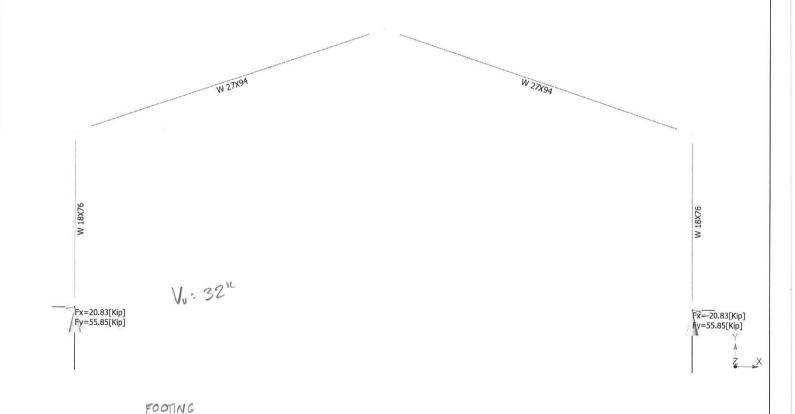


Current Date: 8/3/2018 1:25 PM

Units system: English

File name: Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\RAM\18121 - Activity Room Frame.etz\

Load condition: LC1=DL+SL



0.3 (56K) + 0,3 KSF (3.5') \( 2 \) (3.5') \( 15' \) = 44.4K > 20.8K OX

FRICTION

PASSIVE SOIL

W = \\ \frac{56^k}{1.8 \text{ KSF}} = 5.6' \quad \text{USE F6}

LATERAL

### IBC 2015 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

03-Aug-18

1:46 PM

ENGINEERS

Version Date: December 5, 2017

JOB TITLE: NAC Recreation Center DESCRIPTION: Typical W columns

JOB #: DESIGNER: 18121 TMD

Shear Calculations **Bolts Resisting Shear** 

Steel Shear Capacity (Section 17.5.1)

16.86 kips / AB

Bolts resist tension and shear 67.5 kips

В

Ultimate Steel Shear Capacity based on

Assumed Breakout Surface

Specify which system resists shear

Number of anchors resisting shear based on assumed concrete breakout surface

Grout height below base plate, h2: 1.5 inches

Concrete Shear Breakout Capacity (Section 17.5.2)

$\psi_{ec,V}$ :	1.00	$\psi_{\sf ed,V}$ :	0.84	₩c.\	1.2	$\psi_{h,N}$	; 1.00
Top Edge B	olts				Bottom Edg	e Bolts	
Edge Distance:	10.0	inches		Edg	ge Distance :	10.0	inches
A <sub>Vc</sub> =	270.0	in <sup>2</sup>			A <sub>Vc</sub> =	270.0	in <sup>2</sup>
A <sub>Vco</sub> =	450.0	in <sup>2</sup>			A <sub>Vco</sub> =	450.0	in <sup>2</sup>
$V_b =$	18.4	kips			$V_b =$	18.4	kips
$V_{cb} =$	11.1	kips			V <sub>cb</sub> =	11.1	kips
Concrete SI	hear Pry	out Capacity	(Section 17.	5.3)			
V <sub>cp</sub> =	97.2	kips			$N_b =$	55.1	kips
r				2		11070100	

Group Capacity V<sub>n</sub>: 97.2 kips Group Capacity V<sub>n</sub>: 97.2 kips

Ultimate Concrete Capacity Based on Pryout Strength Only

Combined Tension and Shear

Shear Fo	rce Acting	Towards Top Pier Edge, V <sub>top</sub>	,5	Shear For	ce Acting Tov	vards Bottom Pier Edge, V <sub>bottom</sub>
				0.000	< 1 - OK	Concrete Tension (P <sub>u</sub> / $\phi$ P <sub>n</sub> )
0.439	< 1 - OK	Concrete Shear (V <sub>u</sub> /φV <sub>n</sub> )		0.439	< 1 - OK	Concrete Shear (V <sub>u</sub> / $\phi$ V <sub>n</sub> )
			/	0.439	< 1 - OK	Concrete Combined (Section D.7)
			/	0.000	< 1 - OK	Steel Tension (P <sub>u</sub> / $\phi$ P <sub>ss</sub> )
0.730	< 1 - OK	Steel Shear (V <sub>u</sub> / $\phi$ V <sub>ss</sub> )	/	0.730	< 1 - OK	Steel Shear (V <sub>u</sub> / $\phi$ V <sub>ss</sub> )
				0.730	< 1 - OK	Steel Combined (Section D.7)

Anchor Bolts O.K. Checks Shear Only Concrete O.K.

Anchor Bolts O.K. Checks Shear and Tension Concrete O.K.

ACI 318 SECTION 17.2.3.4.3 DOES NOT APPLY

### **DESIGN SUMMARY**

**Anchor Bolts** 

(4) 1" diameter headed anchor bolts w/ 12"minimum embedment

Designed for combined tension and shear

**Tension Confined Pier** 

18" wide x 24" long w/ min (1) #5 vertical bars

Designed to transfer anchor bolt tension into reinforcement

**Shear Confined Pier** 

18" wide x 24" long w/ min (2) #4 hairpin

Designed to transfer anchor bolt shear into reinforcement



### IBC 2015 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

03-Aug-18 1:46 PM

Version Date: December 5, 2017

JOB TITLE: NAC Recreation Center

DESCRIPTION: Typical W columns

JOB #: DESIGNER: 18121 TMD

### **Anchor Reinforcing**

### Reinforcing Data

Ψt:	1	
<b>∀е</b> :	1	
₩s:	1	
21.	0.0	

Assumes that vertical reinforcement layout is symmetrical around anchor

bolt pattern

NOTE: The calculation for the concrete anchor capacities are based on 1) Tension pullout, 2) Shear pryout. The breakout strength in tension and shear and side face blowout are omitted because the vertical reinforcement is used to confine this failure cone.

### Pier Reinforcement to Resist Tension Breakout (17.4.2.9)

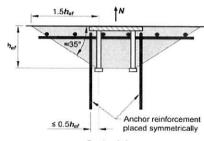
Vert. Pier Reinforcing Size : 5 bar Distance from A.B. to Rebar (B) : in Cover above vert. reinf (A): 2 in 3 in Rebar Area: 0.31  $in^2$ 

Rebar Diameter: 0.625 in 21.36 in 6.00 in

 $0.75^*F_y$  of Rebar @  $I_{de}$  -  $f_s$  : 12.64 ksi 0.00 in<sup>2</sup> A<sub>st</sub>:

Total # of vertical bars required : Quantity of reinforcement placed symmetrically around anchor bolts

Embedment of standard hook: 6.00



Section A-A

### Pier Reinforcement to Resist Shear Breakout (17.5.2.9)

Hairpin/stirrup reinforcing size : bar

Rebar area : 0.4  $in^2$ 

0.71 in<sup>2</sup> Total # of hairpins/stirrups

Quantity of hairpins or stirrups 2 required wrapped around anchor bolts

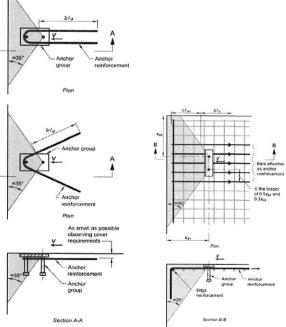


Fig. RD.6.2.9(a)—Hairpin anchor reinforcement for shear.

Title Block Line 1 You can change this area using the "Settings" menu item and then using the "Printing & Title Block" selection.

Project Title: Engineer: Project ID: Project Descr:

Printed: 2 AUG 2018, 8:33AM

### Wood Beam

Title Block Line 6

File = Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\Other\18121.ec6

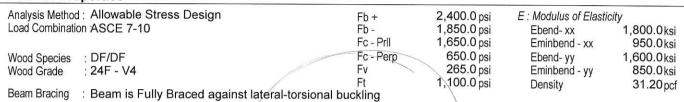
Lic. #: KW-06002489 Description: Vestibule Glulam Beam Licensee: ARW ENGINEERS

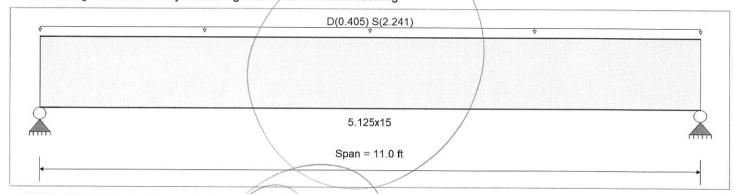
### **CODE REFERENCES**

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

Load Combination Set : ASCE 7-10

### **Material Properties**





### **Applied Loads**

Max Upward Total Deflection

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

Uniform Load: D = 0.0150, S = 0.0830 ksf, Tributary Width = 27.0 ft

DESIGN SUMMARY				Design OK
Maximum Bending Stress Ratio Section used for this span	= 0.905 1 5.125x15	Maximum Shear Stress Ratio Section used for this span	=	0.721 : 1 5.125x15
fb : Actual	= 2,498.86psi	fv : Actual	=	219.71 psi
FB : Allowable	= 2,760.00psi	/ Fv : Allowable	=	304.75 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	H E	+D+S 9.755 ft Span # 1
Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection	0.200	atio = 0 <240		

0<180

0.000 in Ratio =

**Maximum Forces & Stresses for Load Combinations** 

Load Combination		Max Stres	s Ratios								Mor	ment Values			Shear Va	lues
Segment Length	Span#	M	V	$C^{d}$	C F/V	$C_i$ $C_r$ $C_m$	$c_t$ $c$	C <sub>L</sub>	М	fb	F'b	V	fv	F'v		
D Only													0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.177	0.141	0.90	1.000	1.00	1.00	1.00	1.00	1.00	6.13	382.48	2160.00	1.72	33.63	238.50
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.905	0.721	1.15	1.000	1.00	1.00	1.00	1.00	1.00	40.02	2,498.86	2760.00	11.26	219.71	304.75
+D+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.714	0.568	1.15	1.000	1.00	1.00	1.00	1.00	1.00	31.55	1,969.76	2760.00	8.88	173.19	304.75
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.060	0.048	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.68	229.49	3840.00	1.03	20.18	424.00

Title Block Line 1
You can change this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.

Project Title: Engineer: Project ID: Project Descr:

Title Block Line 6

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**Wood Beam** 

File = Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\Other\18121.ec6

Lic. # : KW-06002489

Description: Vestibule Glulam Beam

Licensee : ARW ENGINEERS

Overell	Maximo	Deflections
Overall	waxiiiiuiii	Deflections

Load Combination	Span	Max. "-" Defl L	ocation in Span	Load Combination	Max. "+" Defl	Location in Spar
+D+S	1	0.3379	5.540		0.0000	0.000
Vertical Reactions			Suppo	rt notation : Far left is #1	Values in KIPS	
Load Combination		Support 1	Support 2			
Overall MAXimum		14.55	3 14.553			
Overall MINimum		12.32	6 12.326			
D Only		2.22	3 2.228			
+D+S		14.55	3 14.553			
+D+0.750S		11.47	2 11.472			
+0.60D		1.33	7 1.337			
S Only		12.32	6 12.326			

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Project Title: Engineer: Project ID: Project Descr:

Title Block Line 6

Printed: 2 AUG 2018, 10:48AM

#### Steel Column

Lic. #: KW-06002489

Description \

File = Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\Other\18121.ec6

#### Vestibule Column under GLB

#### 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:

Steel Stress Grade Fy: Steel Yield E: Elastic Bending Modulus

HSS4x4x1/4 Allowable Strength

, A500, Grade B, Fy = 46 ksi, Carbon

46.0 ksi 29000 ksi

Overall Column Height

12.0 ft Top & Bottom Fixity Top & Bottom Pinned

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 = 12.0 ft, K = 1.0

Y-Y (depth) axis

Unbraced Length for Y-Y Axis buckling = 12.0 ft, K = 1.0

#### **Applied Loads**

Column self weight included: 146.170 lbs \* Dead Load Factor AXIAL LOADS.

Axial Load at 12.0 ft, Yecc = 1.0 in, D = 2.20, S = 12.40 k

#### **DESIGN SUMMARY**

Bending & Shear Check Results	
PASS Max. Axial+Bending Stress Ratio =	0.3903 : 1
Load Combination	/ +D+S
Location of max.above base	11.919 ft
At maximum location values are	
Pa : Axial	14.746 k
Pn / Omega : Allowable	50.754 k
Ma-x : Applied	-1.209 k-ft
Mn-x / Omega : Allowable	10.765 k-ft
Ma-y : Applied	0.0 k-ft
Mn-y / Omega : Allowable	10.765 k-ft
PASS Maximum Shear Stress Ratio =	<b>0.003988</b> : 1

0.003988:1 +D+S 0.0 ft 0.1014 k 25.423 k

Maximum Load Reactions . .

Top along X-X 0.0 kBottom along X-X 0.0 kTop along Y-Y 0.1014 k Bottom along Y-Y 0.1014 k

#### Maximum Load Deflections . . .

Along Y-Y -0.08660 in at 7.007ft above base for load combination :+D+S Along X-X 0.0 in at 0.0ft above base for load combination:

#### **Load Combination Results**

Load Combination

Location of max.above base

At maximum location values are . . . Va : Applied

Vn / Omega: Allowable

Extreme Value

Maximum

Minimum

@ Base

14.746

1.408

	Maximum Axial +	Bending S	tress Ratios	Maximu	m Shear R	atios	
Load Combination	Stress Ratio	Status	Location	Stress Ratio	Status	Location	
D Only	0.046	PASS	0.00 ft	0.001	PASS	0.00 ft	10
+D+S	0.390	PASS	11.92 ft	0.004	PASS	0.00 ft	
+D+0.750S	0.308	PASS	11.92 ft	0.003	PASS	0.00 ft	
+0.60D	0.028	PASS	0.00 ft	0.000	PASS	0.00 ft	

Axial @ Base

Maximum Reactions Note: Only non-zero reactions are listed. **Axial Reaction** X-X Axis Reaction Y-Y Axis Reaction Mx - End Moments k-ft My - End Moments Load Combination @ Base @ Base @ Base @ Top @ Base @ Top @ Base @ Top D Only 2.346 -0.015 0.015 +D+S 14.746 -0.1010.101 = 8.2 FT 2 +D+0.750S 1.8 KSF 11.646 -0.080 0.080 +0.60D 1.408 -0.009 0.009 S Only 12.400 -0.0860.086 Extreme Reactions **Axial Reaction** X-X Axis Reaction Y-Y Axis Reaction Mx - End Moments k-ft My - End Moments Item

OK

@ Top

@ Base

-0.101

-0.009

@ Top

0.101

0.009

@ Base

@ Top

@ Base

@ Top

@ Base

Project Title: Engineer: Project ID: Project Descr:

**Steel Column** 

Title Block Line 6

Ycg

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Lic. #: KW-06002489

Description: Vestibule Column under GLB

0.000 in

Licensee : ARW ENGINEERS

		Axial Read	tion	X-X Axis Re	action	k	Y-Y Axis	Reaction	Mx - En	d Mom	nents	k-ft	My - End	Moments
Item	Extreme Valu	ue @ Base	Э	@ Base	@ To	р	@ Base	@ Top	@ Bas	Э	@ Top		@ Base	@ Top
Reaction, X-X Axis Base	Maximum	2.3	346				-0.015	0.015						
П.	Minimum	2.3	346				-0.015	0.015						
Reaction, Y-Y Axis Base	Maximum	1.4	804				-0.009	0.009						
	Minimum	14.7	<b>'</b> 46				-0.101	0.101						
Reaction, X-X Axis Top	Maximum	2.3	346				-0.015	0.015						
н	Minimum	2.3	346				-0.015	0.015						
Reaction, Y-Y Axis Top	Maximum	2.3	346				-0.015	0.015						
1	Minimum	12.4	00				-0.086	0.086						
Moment, X-X Axis Base	Maximum	2.3	346				-0.015	0.015						
н	Minimum	2.3	346				-0.015	0.015						
Moment, Y-Y Axis Base	Maximum	2.3	346				-0.015	0.015						
п	Minimum	2.3	346				-0.015	0.015						
Moment, X-X Axis Top	Maximum	2.3	346				-0.015	0.015						
л	Minimum	2.3	346				-0.015	0.015						
Moment, Y-Y Axis Top	Maximum	2.3	346				-0.015	0.015						
	Minimum	2.3	46				-0.015	0.015						
Maximum Deflection	ns for Load Co	mbination	8											
Load Combination		Max. X-X [	eflection	Distan	ce		Max. Y-Y D	eflection	Distanc	e				
D Only		0.000	0 in	0.00	0 ft		-0.013		7.007	ft				
+D+S		0.000	0 in	0.00			-0.087		7.007	ft				
+D+0.750S		0.000	0 in	0.00			-0.068		7.007	ft				
+0.60D		0.000	0 in	0.00			-0.008		7.007	ft				
S Only		0.000	0 in	0.00	0 ft		-0.074		7.007	ft				
Steel Section Prop	erties :	HSS4x4x1	14											
Depth	= 4.000		Lxx	=		7.80	) in^4		J		=		12.800 in^4	1
Design Thick	= 0.233		Sxx	=			) in^3		-					5
Width	= 4.000		Rxx	=		1.520								
Wall Thick	= 0.250		Zx	=			) in^3							
Area		) in^2	l yy	_			) in^4		С		=		6 E60 :- A	2
/ 11 UU				_					U		-		6.560 in^3	)
	- 10 104	nlf	CIM	_			1 :- 12							
Weight	= 12.181	plf	S yy R yy	=		3.900 1.520	) in^3							

Title Block Line 6

Project Title: Engineer: Project ID: Project Descr:

A36

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#### **Steel Column**

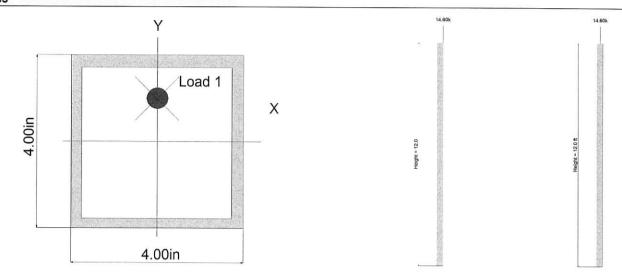
File = Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\Other\18121.ec6

Lic. # : KW-06002489

Description : Vestibule Column under GLB

Licensee : ARW ENGINEERS

#### **Sketches**



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#### **Wood Beam**

Lic. #: KW-06002489

Licensee: ARW ENGINEERS

Description: Outlookers

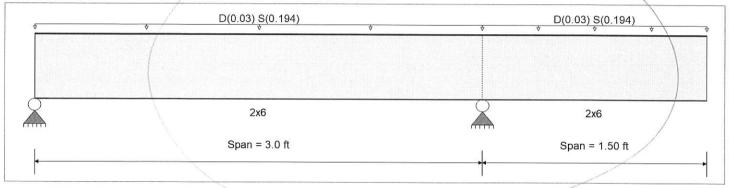
#### CODE REFERENCES

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+	900.0 psi	E : Modulus of Elastic	city	•
Load Combination ASCE 7-10	Fb-	900.0 psi	Ebend- xx	1,600.0ksi	
	Fc - Prll	1,350.0 psi	Eminbend - xx	580.0ksi	
Wood Species : Douglas Fir - Larch	Fc - Perp	625.0 psi			
Wood Grade : No.2	Fv	180.0 psi			
	Ft	575.0 psi	Density	31.20 pcf	
Beam Bracing : Beam is Fully Braced against lateral-torsiona	l buckling		Repetitive Membe	er Stress Increase	



#### **Applied Loads**

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

File = Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\Other\18121.ec6

Load for Span Number 1

Uniform Load: D = 0.0150, S = 0.0970 ksf, Tributary Width = 2.0 ft

Load for Span Number 2

Uniform Load: D = 0.0150, S = 0.0970 ksf, Tributary Width = 2.0 ft

Maximum Bending Stress Ratio	= /	0.258 1	Maximum Shear Stress Ratio	2047	0.000 1
Section used for this span	-/	2x6	Section used for this span	=	0.280 : 1
fb : Actual	=	399.87 psi	fv : Actual	_	<b>2x6</b> 57.93 psi
FB : Allowable	= \	1,547.33 psi	Fv : Allowable	_	207.00 psi
	- \	Water State of		_	Annual Control of the
Load Combination		+D+S	Load/Combination		+D+S
Location of maximum on span	= \	3.000ft	Location of maximum on span	=	2.547 ft
Span # where maximum occurs	= \	Span # 1	Span # where maximum occurs	=	Span #1
Maximum Deflection					
Max Downward Transient Deflecti	on	0.006 in Ratio	<del>=</del> 5666 >=240		
Max Upward Transient Deflection		0.000 in Ratio	= 0<240		
Max Downward Total Deflection		0.007 in Ratio	= 4908 >=180		
Max Upward Total Deflection		0.000 in Ratio			

Maximum	Forces	2	Stroseos for	l oad	Combinations
IVIAXIIIIIIII	LOICES	Ot.	onesses for	I Oaci	Combinations

Load Combination		Max Stres	s Ratios								Morr	ent Values			Shear Va	lues
Segment Length	Span #	M	V	$C_d$	C FN	Ci	$C_{r}$	$C_{m}$	$C_t$	C <sub>L</sub>	М	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.044	0.048	0.90	1.300	1.00	1.15	1.00	1.00	1.00	0.03	53.55	1210.95	0.04	7.76	162.00
Length = 1.50 ft	2	0.044	0.048	0.90	1.300	1.00	1.15	1.00	1.00	1.00	0.03	53.55	1210.95	0.03	7.76	162.00
+D+S					1.300	1.00	1.15	1.00	1.00	1.00	0.00	00.00	0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.258	0.280	1.15	1.300	1.00	1.15	1.00	1.00	1.00	0.25	399.87	1547.33	0.32	57.93	207.00

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S Only

Project Title: Engineer: Project ID: Project Descr:

Title Block Line 6													Prir	nted: 2 A	UG 2018.	9:50AM
Wood Bean	n							File	= Y:\Proj	ects 2018\	18121 - NAC	Recreation Ce	enter\Engineerin			
Lic. #: KW-06002	2489												Licens	see : AF	RW ENGI	NEERS
Description :	Outlookers														Company of the Company	
Load Combination		Max Stres	s Ratios								Mom	nent Values			Shear Va	alues
Segment Length	Span #	М	٧	$C^{d}$	C FN	Ci	$c_r$	$C_{m}$	$c_t$	C <sub>L</sub>	М	fb	F'b	V	fv	F'v
Length = 1.50 ft +D+0.750S	2	0.258	0.280	1.15	1.300	1.00	1.15	1.00	1.00	1.00	0.25	399.87	1547.33	0.23	57.93	207.00
Length = 3.0 ft	1	0.202	0.219	1.15	1.300 1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.50 ft	2	0.202	0.219	1.15	1.300	1.00	1.15 1.15	1.00	1.00	1.00	0.20	313.29	1547.33	0.25	45.39	207.00
+0.60D	2	0.202	0.219	1.15	1.300	1.00	1.15	1.00	1.00 1.00	1.00 1.00	0.20	313.29	1547.33	0.18	45.39	207.00
Length = 3.0 ft	1	0.015	0.016	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.00	20.42	0.00	0.00		0.00
Length = 1.50 ft	2	0.015	0.016	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.02	32.13 32.13	2152.80 2152.80	0.03	4.66 4.66	288.00 288.00
Overall Maxis	mum D	eflectio	ns											-		200.00
Load Combination		S	Span	Max. "-	Defl	Locatio	n in Span		Load Co	mbinatio	n		Max. "+"	Defl	Location in	Span
+D+S +D+S			1 2		0052 0073		1.274 1.500							000		000
Vertical Read	ctions						Sup	port not	ation : F	ar left is #	<b>#</b> 1		Values in K			
Load Combination					Support	1 Su	pport 2	Suppo	ort 3							
Overall MAXimum					0.25	52	0.756									
Overall MINimum					0.2	18	0.655									
D Only					0.03	34	0.101									
+D+S					0.25	52	0.756									
+D+0.750S					0.19	97	0.592									
+0.60D					0.02	20	0.061									
S Only					0.2	10	O CEE									

0.218

0.655

# LATERAL ANALYSIS



#### PROJECT DESIGN CRITERIA

Governing Building Code :

IBC2015

#### WIND DESIGN

Basic Wind Speed, V<sub>3s</sub>:

115

Wind Importance Factor,  $I_{\rm w}$ :

Wind Exposure :

#### **SEISMIC DESIGN**

Seismic Importance Factor, I<sub>e</sub>: 1

Street: 1000 Ability Way

USGS Design Code: ASCE 7-10

D

City: Park City State: UT

Site Class :

11

40.6808539

Seismic Risk Category :

Latitude : Longitude :

-111.475762

Design Category : D

Basic Seismic Force Resisting System : Light-Frame (Wood) walls sheathed with wood structural panels

Response Modification Factor, R:

6.5

Type of Analysis : STATIC



USGS-Provided Output

 $S_s = 0.624 g$ 

 $F_a = 1.301$ 

 $S_{MS} = 0.812 g$ 

 $S_{DS} = 0.541 g$ 

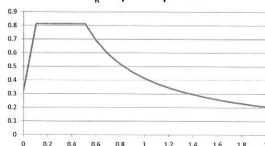
 $s_1 = 0.209 g$ 

 $F_v = 1.982$ 

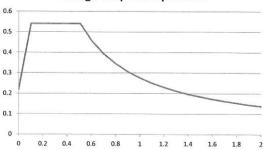
 $S_{M1} = 0.414 g$ 

 $S_{D1} = 0.276 g$ 

MCE<sub>R</sub> Response Spectrum



**Design Response Spectrum** 



82



**ASCE 7 Windspeed** 

ASCE 7 Ground Snow Load

**Related Resources** 

Sponsors

About ATC

Contact

This site will be taken offline on June 30th 2018. Please start using the new site at https://hazards.atcouncil.org.

#### Search Results

L 1

Query Date: Thu Jun 21 2018

Latitude: 40.6809 Longitude: -111.4758

ASCE 7-10 Windspeeds (3-sec peak gust in mph\*):

Use ctrl + scroll to zoom the map

Risk Category I: 105 Risk Category II: 115 Risk Category III-IV: 120 MRI\*\* 10-Year: 76 MRI\*\* 25-Year: 84 MRI\*\* 50-Year: 90

ASCE 7-05 Windspeed: 90 (3-sec peak gust in mph) ASCE 7-93 Windspeed: 70 (fastest mile in mph)

Google

Map data @2018 Google, INEGI

MRI\*\* 100-Year: 96

Users should consult with local building officials to determine if there are community-specific wind speed requirements that govern.

Print your results

#### WINDSPEED WEBSITE DISCLAIMER

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<sup>\*</sup>Miles per hour \*\*Mean Recurrence Interval

Tekla Tedds	Project				Job Ref.	B3
ARW Engineers	Section				Sheet no./rev.	
	Calc. by	Date 7/2/2018	Chk'd by	Date	App'd by	Date

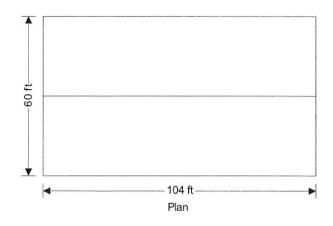
#### WIND LOADING (ASCE7-10)

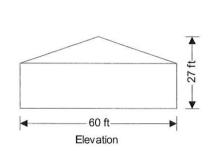
In accordance with ASCE7-10 incorporating Errata No. 1 and Errata No. 2

Using the directional design method

HIGH ROOF

Tedds calculation version 2.0.20





#### **Building data**

Type of roof	Gable
Length of building	b = <b>104.00</b> ft
Width of building	d = 60.00  ft
Height to eaves	H = 17.00 ft
Pitch of roof	$\alpha_0 = 18.5 \text{ deg}$
Mean height	h = 22.02 ft

#### General wind load requirements

Basic wind speed	V = <b>115.0</b> mph
Risk category	II
Velocity pressure exponent coeff (Table 26.6-1)	$K_d = 0.85$
Exposure category (cl.26.7.3)	С
Enclosure classification (cl.26.10)	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1)	$GC_{pi_p} = 0.18$
Internal pressure coef -ve (Table 26.11-1)	$GC_{pi_n} = -0.18$
Gust effect factor	$G_f = 0.85$

#### **Topography**

Topography factor not significant	$K_{zt} = 1.0$
Velocity pressure equation	$q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1 psf/mph^2$

#### Velocity pressures table

z (ft)	K <sub>z</sub> (Table 27.3-1)	q <sub>z</sub> (psf)
15.00	0.85	24.46
17.00	0.87	25.04
20.00	0.90	25.90
22.02	0.92	26.36
27.04	0.96	27.52



Project		Job Ref.			
Section		TOCH TOWNSHIP	3819a ikse. 22	Sheet no./rev	·.
Calc. by	Date 7/2/2018	Chk'd by	Date	App'd by	Date

#### Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q<sub>i</sub> = 26.36 psf

Pressures and forces

Net pressure

 $p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$ 

Net force

 $F_w = p \times A_{ref}$ 

#### Roof load case 1 - Wind 0, GCpi 0.18, -cpe

Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
A (-ve)	22.02	-0.42	26.36	-14.18	3290.02	-46.64
B (-ve)	22.02	-0.57	26.36	-17.52	3290.02	-57.64

Total vertical net force

 $F_{w,v} = -98.89 \text{ kips}$ 

Total horizontal net force

 $F_{w,h} = 3.49 \text{ kips}$ 

N-S

#### Walls load case 1 - Wind 0, GCpi 0.18, -cpe

Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
A <sub>1</sub>	15.00	0.80	24.46	11.89	1560.00	18.54
A <sub>2</sub>	17.00	0.80	25.04	12.28	208.00	2.55
В	22.02	-0.50	26.36	-15.95	1768.00	-28.20
С	22.02	-0.70	26.36	-20.43	1321.14	-26.99
D	22.02	-0.70	26.36	-20.43	1321.14	-26.99

#### **Overall loading**

Projected vertical plan area of wall

 $A_{\text{vert\_w\_0}} = b \times H = 1768.00 \text{ ft}^2$ 

Projected vertical area of roof

 $A_{\text{vert}_{r_0}} = b \times d/2 \times \tan(\alpha_0) = 1043.94 \text{ ft}^2$ 

Minimum overall horizontal loading

 $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = 36.64 \text{ kips}$ 

Leeward net force

 $F_1 = F_{w,wB} = -28.2 \text{ kips}$ 

Windward net force

 $F_w = F_{w,wA_1} + F_{w,wA_2} = 21.1 \text{ kips}$ 

Overall horizontal loading

 $F_{w,total} = max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = 52.8 \text{ kips}$ 

#### Roof load case 2 - Wind 0, GCpi -0.18, -0cpe

Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
A (+ve)	22.02	0.05	26.36	5.85	3290.02	19.24
B (+ve)	22.02	-0.57	26.36	-8.03	3290.02	-26.41

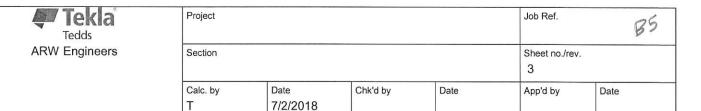
Total vertical net force

 $F_{w,v} = -6.80 \text{ kips}$ 

Total horizontal net force

 $F_{w,h} = 14.48 \text{ kips}$ 

Walls load case 2 - Wind 0, GCpi -0.18, -0cpe



Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
A <sub>1</sub>	15.00	0.80	24.46	21.38	1560.00	33.35
A <sub>2</sub>	17.00	0.80	25.04	21.77	208.00	4.53
В	22.02	-0.50	26.36	-6.46	1768.00	-11.42
С	22.02	-0.70	26.36	-10.94	1321.14	-14.45
D	22.02	-0.70	26.36	-10.94	1321.14	-14.45

#### Overall loading

Projected vertical plan area of wall

Projected vertical area of roof

Minimum overall horizontal loading

Leeward net force

Windward net force

Overall horizontal loading

 $A_{\text{vert\_w\_0}} = b \times H = 1768.00 \text{ ft}^2$ 

 $A_{\text{vert_r}_0} = b \times d/2 \times \tan(\alpha_0) = 1043.94 \text{ ft}^2$ 

 $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = 36.64 \text{ kips}$ 

E-W

 $F_1 = F_{w,wB} = -11.4 \text{ kips}$ 

 $F_w = F_{w,wA_1} + F_{w,wA_2} = 37.9 \text{ kips}$ 

 $F_{w,total} = max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = 63.8 \text{ kips}$ 

#### Roof load case 3 - Wind 90, GCpi 0.18, -cpe

Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
A (-ve)	22.02	-0.90	26.36	-24.91	696.56	-17.35
B (-ve)	22.02	-0.90	26.36	-24.91	696.56	-17.35
C (-ve)	22.02	-0.50	26.36	-15.95	1393.13	-22.22
D (-ve)	22.02	-0.30	26.36	-11.47	3793.78	-43.51

Total vertical net force

F<sub>w,v</sub> = **-95.25** kips

Total horizontal net force

 $F_{w,h} = 0.00 \text{ kips}$ 

Walls load case 3 - Wind 90, GCpi 0.18, -cpe

Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
A <sub>1</sub>	15.00	0.80	24.46	11.89	900.00	10.70
A <sub>2</sub>	20.00	0.80	25.90	12.87	273.10	3.51
A <sub>3</sub>	27.04	0.80	27.52	13.97	148.08	2.07
В	22.02	-0.35	26.36	-12.66	1321.14	-16.73
С	22.02	-0.70	26.36	-20.43	1768.00	-36.12
D	22.02	-0.70	26.36	-20.43	1768.00	-36.12

#### Overall loading

Projected vertical plan area of wall

Projected vertical area of roof

Minimum overall horizontal loading

Leeward net force Windward net force  $A_{\text{vert}_{-}\text{w}_{-}90} = d \times H + d^2 \times \tan(\alpha_0) / 4 = 1321.14 \text{ ft}^2$ 

 $A_{\text{vert r 90}} = 0.00 \text{ ft}^2$ 

 $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = 21.14 \text{ kips}$ 

 $F_1 = F_{w,wB} = -16.7 \text{ kips}$ 

 $F_w = F_{w,wA_1} + F_{w,wA_2} + F_{w,wA_3} = 16.3 \text{ kips}$ 

Tekla Tekla	Project				Job Ref.	B6
ARW Engineers	Section				Sheet no./rev.	
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Overall horizontal loading

 $F_{w,total} = max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = 33.0 \text{ kips}$ 

Roof load case 4 - Wind 90, GCpi -0.18, +cpe

Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
A (+ve)	22.02	-0.18	26.36	0.71	696.56	0.50
B (+ve)	22.02	-0.18	26.36	0.71	696.56	0.50
C (+ve)	22.02	-0.18	26.36	0.71	1393.13	0.99
D (+ve)	22.02	-0.18	26.36	0.71	3793.78	2.70

Total vertical net force

 $F_{w,v} = 4.44 \text{ kips}$ 

Total horizontal net force

 $F_{w,h} = 0.00 \text{ kips}$ 

#### Walls load case 4 - Wind 90, GCpi -0.18, +cpe

Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
A <sub>1</sub>	15.00	0.80	24.46	21.38	900.00	19.24
A <sub>2</sub>	20.00	0.80	25.90	22.36	273.10	6.11
A <sub>3</sub>	27.04	0.80	27.52	23.46	148.08	3.47
В	22.02	-0.35	26.36	-3.17	1321.14	-4.19
С	22.02	-0.70	26.36	-10.94	1768.00	-19.34
D	22.02	-0.70	26.36	-10.94	1768.00	-19.34

#### **Overall loading**

Projected vertical plan area of wall

Projected vertical area of roof

Minimum overall horizontal loading

Leeward net force

Windward net force

Overall horizontal loading

 $A_{\text{vert\_w\_90}} = d \times H + d^2 \times \tan(\alpha_0) / 4 = 1321.14 \text{ ft}^2$ 

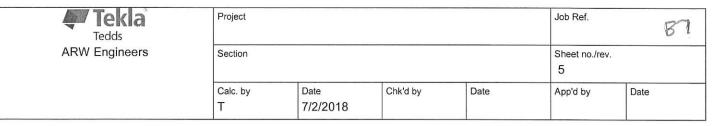
 $A_{vert r 90} = 0.00 ft^2$ 

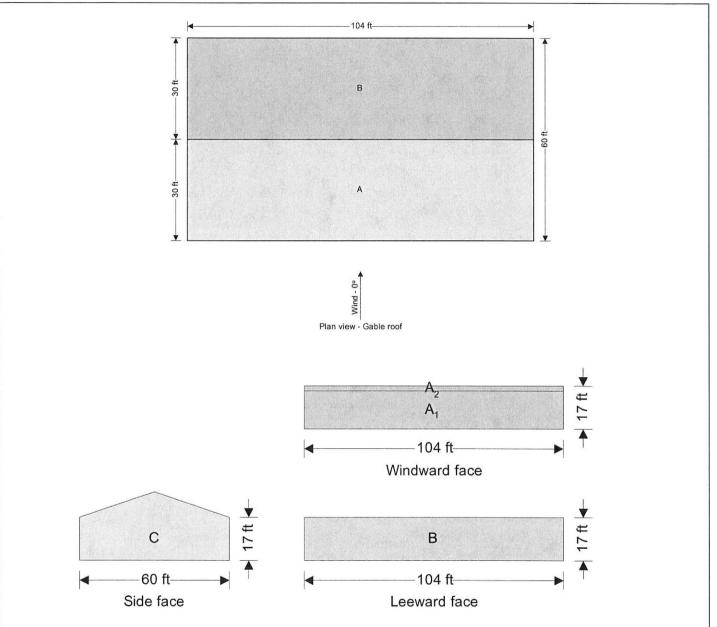
 $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = 21.14 \text{ kips}$ 

 $F_1 = F_{w,wB} = -4.2 \text{ kips}$ 

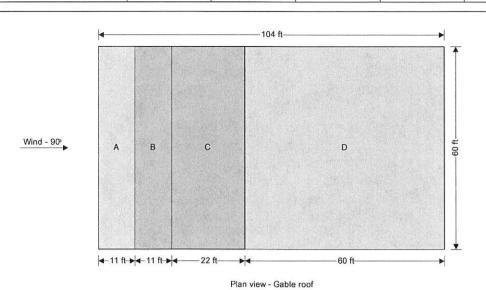
 $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} = 28.8 \text{ kips}$ 

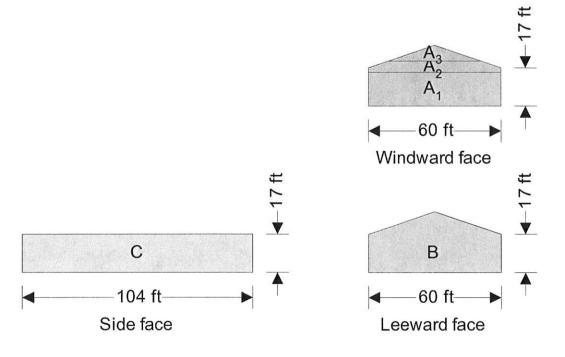
 $F_{w,total} = max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = 33.0 \text{ kips}$ 





Tekla Tedds	Project				Job Ref.	B8
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Tekla Tedds	Project				Job Ref.	89
ARW Engineers	Section				Sheet no./rev.	
	Calc. by	Date 7/2/2018	Chk'd by	Date	App'd by	Date

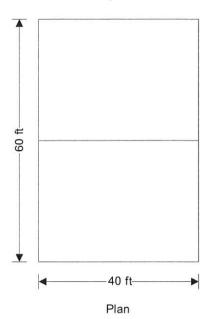
#### WIND LOADING (ASCE7-10)

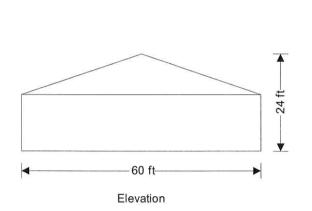
In accordance with ASCE7-10 incorporating Errata No. 1 and Errata No. 2

Using the directional design method

LOW

Tedds calculation version 2.0.20





#### **Building data**

Type of roof Gable

Length of building b = 40.00 ftWidth of building d = 60.00 ftHeight to eaves H = 14.00 ftPitch of roof  $\alpha_0 = 18.5 \text{ deg}$ Mean height h = 19.02 ft

#### General wind load requirements

Basic wind speed V = 115.0 mph

Risk category II

Velocity pressure exponent coeff (Table 26.6-1) K<sub>d</sub> = **0.85** 

Exposure category (cl.26.7.3)

Enclosure classification (cl.26.10) Enclosed buildings

Internal pressure coef +ve (Table 26.11-1)  $GC_{pi\_p} = 0.18$ Internal pressure coef -ve (Table 26.11-1)  $GC_{pi\_n} = -0.18$ 

Gust effect factor  $G_f = 0.85$ 

#### Topography

Topography factor not significant  $K_{zt} = 1.0$ 

Velocity pressure equation  $q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1 psf/mph^2$ 

#### Velocity pressures table

z (ft)	K <sub>z</sub> (Table 27.3-1)	q <sub>z</sub> (psf)
14.00	0.85	24.46



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z (ft)	K <sub>z</sub> (Table 27.3-1)	q <sub>z</sub> (psf)
15.00	0.85	24.46
15.00	0.85	24.46
19.02	0.89	25.62
24.04	0.93	26.83

#### Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.)  $q_i = 25.62 \text{ psf}$ 

#### Pressures and forces

Net pressure

 $p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$ 

Net force

 $F_w = p \times A_{ref}$ 

#### Roof load case 1 - Wind 0, GCpi 0.18, -cpe

Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
A (-ve)	19.02	-0.39	25.62	-13.21	1265.39	-16.71
B (-ve)	19.02	-0.57	25.62	-17.02	1265.39	-21.54

Total vertical net force

 $F_{w,v} = -36.28 \text{ kips}$ 

Total horizontal net force

 $F_{w,h} = 1.53 \text{ kips}$ 

#### Walls load case 1 - Wind 0, GCpi 0.18, -cpe

Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
Α	14.00	0.80	24.46	12.02	560.00	6.73
В	19.02	-0.40	25.62	-13.32	560.00	-7.46
С	19.02	-0.70	25.62	-19.85	1141.14	-22.66
D	19.02	-0.70	25.62	-19.85	1141.14	-22.66

#### **Overall loading**

Projected vertical plan area of wall

 $A_{vert\_w\_0} = b \times H = 560.00 \text{ ft}^2$ 

Projected vertical area of roof

 $A_{\text{vert}_r_0} = b \times d/2 \times \tan(\alpha_0) = 401.51 \text{ ft}^2$ 

Minimum overall horizontal loading

 $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = \textbf{12.17 kips}$ 

N-S

Leeward net force

 $F_1 = F_{w,wB} = -7.5 \text{ kips}$ 

Windward net force

 $F_w = F_{w,wA} = 6.7 \text{ kips}$ 

Overall horizontal loading

 $F_{w,total} = max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = 15.7 \text{ kips}$ 

#### Roof load case 2 - Wind 0, GCpi -0.18, -0cpe

Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
A (+ve)	19.02	0.09	25.62	6.53	1265.39	8.26
B (+ve)	19.02	-0.57	25.62	-7.80	1265.39	-9.87

Total vertical net force

 $F_{w,v} = -1.53 \text{ kips}$ 

Total horizontal net force

 $F_{w,h} = 5.75 \text{ kips}$ 



Project				Job Ref.	BII
Section				Sheet no./rev.	
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#### Walls load case 2 - Wind 0, GCpi -0.18, -0cpe

Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
А	14.00	0.80	24.46	21.24	560.00	11.90
В	19.02	-0.40	25.62	-4.10	560.00	-2.30
С	19.02	-0.70	25.62	-10.63	1141.14	-12.13
D	19.02	-0.70	25.62	-10.63	1141.14	-12.13

#### Overall loading

Projected vertical plan area of wall

Projected vertical area of roof

Minimum overall horizontal loading

Leeward net force Windward net force

Overall horizontal loading

 $A_{vert\_w\_0} = b \times H = 560.00 \text{ ft}^2$ 

 $A_{\text{vert}_{r_0}} = b \times d/2 \times tan(\alpha_0) = 401.51 \text{ ft}^2$ 

 $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = 12.17 \text{ kips}$ 

E-W

 $F_1 = F_{w,wB} = -2.3 \text{ kips}$  $F_w = F_{w,wA} = 11.9 \text{ kips}$ 

 $F_{w,total} = max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = 19.9 \text{ kips}$ 

#### Roof load case 3 - Wind 90, GCpi 0.18, -cpe

Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
A (-ve)	19.02	-0.90	25.62	-24.21	601.66	-14.57
B (-ve)	19.02	-0.90	25.62	-24.21	601.66	-14.57
C (-ve)	19.02	-0.50	25.62	-15.50	1203.32	-18.65
D (-ve)	19.02	-0.30	25.62	-11.14	124.14	-1.38

Total vertical net force

 $F_{w,v} = -46.62 \text{ kips}$ 

Total horizontal net force

 $F_{w,h} = 0.00 \text{ kips}$ 

#### Walls load case 3 - Wind 90, GCpi 0.18, -cpe

Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
A <sub>1</sub>	15.00	0.80	24.46	12.02	897.01	10.78
A <sub>2</sub>	15.00	0.80	24.46	12.02	0.00	0.00
A <sub>3</sub>	24.04	0.80	26.83	13.63	244.18	3.33
В	19.02	-0.50	25.62	-15.50	1141.14	-17.69
С	19.02	-0.70	25.62	-19.85	560.00	-11.12
D	19.02	-0.70	25.62	-19.85	560.00	-11.12

#### Overall loading

Projected vertical plan area of wall

0.00.62

Projected vertical area of roof

 $A_{vert_r_{90}} = 0.00 \text{ ft}^2$ 

Minimum overall horizontal loading

 $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = 18.26 \text{ kips}$ 

Leeward net force

 $F_1 = F_{w,wB} = -17.7 \text{ kips}$ 

Windward net force

 $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} = 14.1 \text{ kips}$ 

 $A_{\text{vert\_w\_90}} = d \times H + d^2 \times \tan(\alpha_0) / 4 = 1141.14 \text{ ft}^2$ 



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Overall horizontal loading

 $F_{w,total} = max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = 31.8 \text{ kips}$ 

Roof load case 4 - Wind 90, GCpi -0.18, +cpe

Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
A (+ve)	19.02	-0.18	25.62	0.69	601.66	0.42
B (+ve)	19.02	-0.18	25.62	0.69	601.66	0.42
C (+ve)	19.02	-0.18	25.62	0.69	1203.32	0.83
D (+ve)	19.02	-0.18	25.62	0.69	124.14	0.09

Total vertical net force

 $F_{w,v} = 1.66 \text{ kips}$ 

Total horizontal net force

 $F_{w,h} = 0.00 \text{ kips}$ 

#### Walls load case 4 - Wind 90, GCpi -0.18, +cpe

Zone	Ref. height (ft)	Ext pressure coefficient cpe	Peak velocity pressure qp (psf)	Net pressure p (psf)	Area A <sub>ref</sub> (ft²)	Net force F <sub>w</sub> (kips)
A <sub>1</sub>	15.00	0.80	24.46	21.24	897.01	19.06
A <sub>2</sub>	15.00	0.80	24.46	21.24	0.00	0.00
A <sub>3</sub>	24.04	0.80	26.83	22.86	244.18	5.58
В	19.02	-0.50	25.62	-6.28	1141.14	-7.16
С	19.02	-0.70	25.62	-10.63	560.00	-5.95
D	19.02	-0.70	25.62	-10.63	560.00	-5.95

#### **Overall loading**

Projected vertical plan area of wall

Projected vertical area of roof

Minimum overall horizontal loading

Leeward net force

Windward net force

Overall horizontal loading

 $A_{\text{vert\_w}} = 0 = d \times H + d^2 \times \tan(\alpha_0) / 4 = 1141.14 \text{ ft}^2$ 

 $A_{vert\ r\ 90} = 0.00\ ft^2$ 

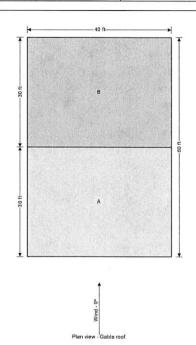
 $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = 18.26 \text{ kips}$ 

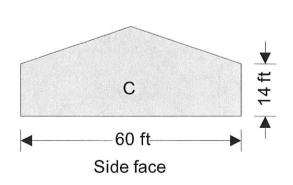
 $F_1 = F_{w,wB} = -7.2 \text{ kips}$ 

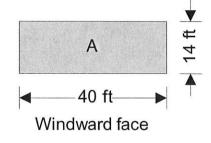
 $F_w = F_{w,wA_1} + F_{w,wA_2} + F_{w,wA_3} = 24.6 \text{ kips}$ 

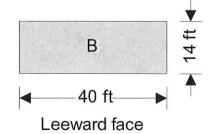
 $F_{w,total} = max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = 31.8 \text{ kips}$ 

Tekla Tedds ARW Engineers	Project				Job Ref.	B13
	Section				Sheet no./rev.	
	Calc. by	Date 7/2/2018	Chk'd by	Date	App'd by	Date

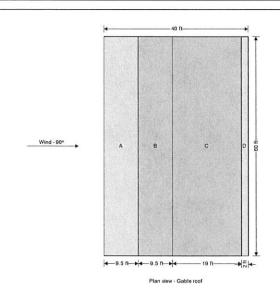


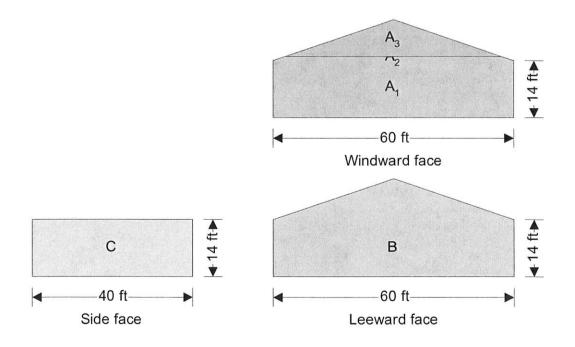










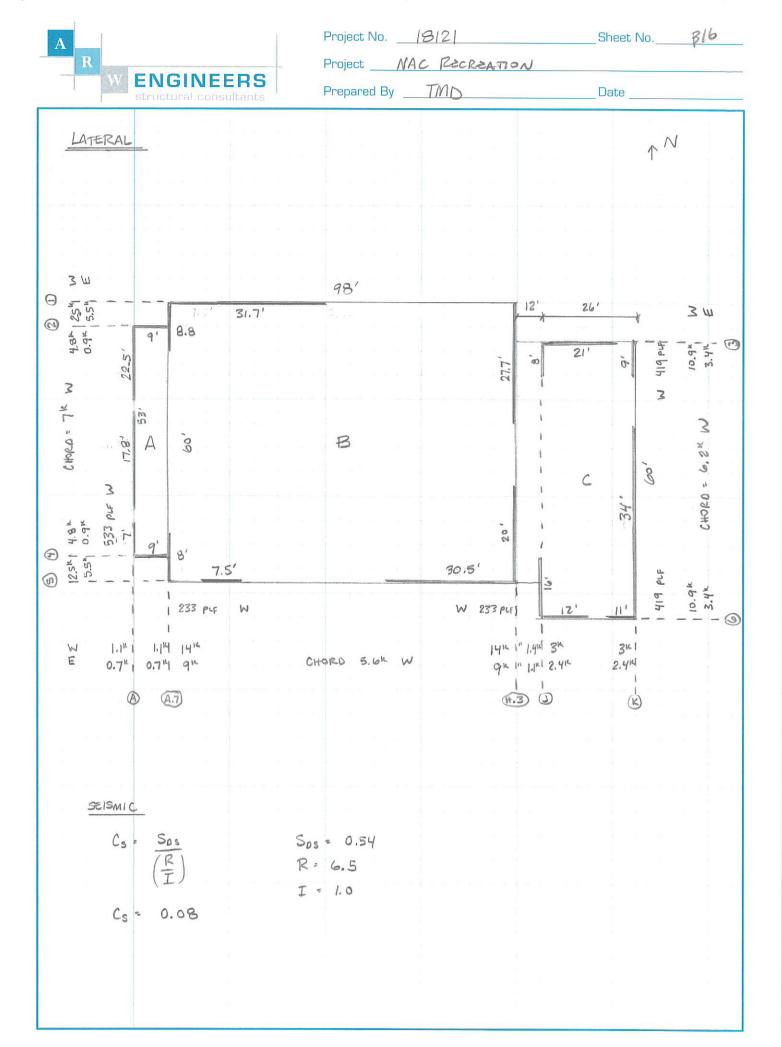




Project NAC RECREATION

Prepared By \_\_\_\_\_\_\_ Date \_\_\_\_\_

	structural consultants — — — — — — — — — — — — — — — — — — —		11 (17) (17) (17)
je (# 1	EA A		
E	$W_{NS} = 0.08 \left[ 34.5  psf \left( 53' \right) + 2 \left( 11  psf \right) \left( \frac{17}{2} \right) \right]$	5	161 PLF
E	WEW = 0.08 [ 34.5 PSF (9') + 11 PSF (17/2)]		33 PLF
W	$\omega_{NS} = 29  psf \left(\frac{17}{2}\right)$	8d	247 PLF - CONTROLS
	WEW = 27 PSF ( 17 + 3.5 )	=	180 PLF
AR	EA B		
E	WAS = 0.08 [ 34.5 PSF (60') + 2(11 PSF) (19/2)]	11	183 PLF
	Wev = 0.08 [34.5 PSF (98) + 2(11PSF) (19)]	8.3	288 PLF CONTROLS
W	UNS = 29 PSF (19/2)	हमी ए	278 PLF CONTROLS
-	Wen = 27 PSF (27,5-24,5)  DIFFERENCE BETWEEN HIGH : LOW ROOF	92	81 PLF
AR	EA C		
E	WNS = 0.08 [34.5 psf (60') + 2(11 psf)(17/2)]	ů te	181 PLF
	WEW = 0.03 [ 34.5 PSF (38) + 11 PSF (17)	g.	113 PUF
W	WAS = 26 PSF (17/2)	5h	221 PLF ] - CONTROLS 360 PLF
- 4 4	WEN = 30 PSF (17 + 3.5)	11	360 PLI= ]





Project No. 18121 Sheet No. 817

Project NAC RECREATION

Prepared By \_\_\_\_\_TMD

Date

### DIAPHRAGM

WORST CASE

AREA A

V = 0.6 (533 PLF) = 320 PLF

UNBLOCKED 19/32" SHEATHING 10° @ 6" OC V. = 800 = 400 PLF (WIND)

Ve = 570 = 285 Pur (ER)

#### CHORDS

AREA A

T= 0.6 (7") = 4.2"

10d NAIL Z= 101" Z' = 162 \*/NAIL

Cd = 1.6

4200 = 26 NAILS

". USZ (28) 10° COMMON NAILS @ TOP PLATE SPLICES

2.6 OF-L No 2 Ft = 575 PSE Ft = 920 PSI

Cd = 1.6

920 (1.5)(5.5) = 7.6k > 4.2k OK

AREA C

TASD = 0.6 (6.2) = 3.7k

CMSTC16 STRAP G.F 4.6K



#### ARW ENGINEERS

ASD SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015
Version: April 17, 2017 Author: Wayne Young, E.I.T.

07-Aug-18 12:19 PM Reviewed By: Troy M. Dye, S.E. JOB #: 18121

JOB TITLE: NAC Recreation ENGINEERS DESCRIPTION: Grid 1

JOB #: DESIGNER:

TMD

		ight of wall =	10.0	psf				Wind (W)	Seismic (E)		
Weight of roof =			15.0	psf	Shea	r at wall li	ne $(V_u) =$	2500	5500	lbs	Strength level
oof Tributary length (bearing & uplift) = Height of wall =		1.0	ft	Shear a	at wall line	$=(V_{ASD})=$	1500	3850	lbs	ASD level	
		ight of wall =	17.0	ft							
W	ind roo	f uplift (W) =	27	YES Shear wall capacity penalized if unblocked							
В	locked	shear wall?	YES								
		d nailing (in)	12	in							
	S	tud spacing	16	in							
							E & W	0.6W	0.7E		
Shear Panel	L		d		H/L		Red.	v (plf)	v (plf)		Shear wall Type
#1	31.7	ft	30.7	ft	0.5	oK	1.00	47	122		Type 'A'
#2		ft		ft							
#3		ft		ft							
#4		ft		ft							
#5		ft		ft							
#6		ft		ft							
#7		ft		ft							

d = 30.7 ft

L = 31.7 ft Mr = 55.7kips

W EQ Mo= 33.6 65.5

kips kips kips (2) 2x6 POST ..OK HDF = -0.7 0.3

Rmax = 0.8 2.1 kips (2) 2x6 POST USE HDU2-SDS2.5 w/ (2) 2x and 5/8" SSIB16 anchor (foundation wall height min 14")

PAB 5



#### **ARW ENGINEERS** SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

07-Aug-18 12:19 PM

ENGINEERS

JOB TITLE: NAC Recreation

DESCRIPTION: Grid 1

JOB#: #REF! DESIGNER: #REF!

Nominal		Allowable	2		Shear wall types
W	E	W	E		
870	620	435	310	Type 'A'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 6 in. o.c. 2 x members @ panel edges
1290	920	645	460		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 4 in. o.c. 2 x members @ panel edges
1680	1200	840	600		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 3 in. o.c. 3 x members @ panel edges
2155	1540	1078	770		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 2 in. o.c. 3 x members @ panel edges
		Holdown	types		
Holdown	EQ A.B.	WA.B.	EQ Cap.	W Cap.	
3.08	2.55	3.61	2.55	3.08	HDU2-SDS2.5 w/ (2) 2x and 5/8" SSTB16 anchor (foundation wall height min 14")
4.57	2.96	4.04	2.96	4.04	HDU4-SDS2.5 w/ (2) 2x and 5/8" SSTB20 anchor (foundation wall height min 18")
5.65	3.33	4.47	3.33	4.47	HDU5-SDS2.5 w/ (2) 2x and 5/8" SSTB24 anchor (foundation wall height min 22")
7.87	6.4	7.62	6.4	7.62	HDU8-SDS2.5 w/ (3) 2x and 7/8" SSTB28 anchor (foundation wall height min 26")
9.54	6.07	7.22	6.07	7.22	HDU11-SDS2.5 w/ (4) 2x and 1" SB1x30 anchor (foundation wall height min 30")

\*NOTE: A.B. capacities are based on worst case scenario of A.B. at end wall. If holdown is located at corner or midwall revise A.B. capacities based on Simpson Manual page 33 or 35.

#### Sill Plate Properties

PT Hem Fir

No. 2

ts = 1.5 Fc perpendicular = 405 psi Max Reaction = 2.1

Automatically calculates maximum compression force at end of shear wall

#### Compression Member Size

Cm = Ct = 1 Ci = Cb= 1.13 Eqn 3.10-2 2015 NDS Fc' = 456 psi (2) 2x6 width = 3 depth = 5.5 in Max load = 7.52 kips

kips

#### **Anchor Bolt Properties**

bolt diameter = 0.625 in. spacing = 32 in. Z parallel = 930 lbs Table 12E 2015 NDS Cd = 1.6 Cm = 1 Ct = Cg = 1 Cdelta = Z' = 1488 v allowable = 558 plf oĸ

:: USE 0.625 in dia. anchor bolt @ 32 in o.c.

:: Use 0.229in x 3in x 3in plate washers See AWC 4.3.6.4.3

#### **Unblocked Shear Wall Sheathing Reduction**

Cub =	1								
Cub	Stud Spacing (in)								
Field nailing (in)	12	16	20	24					
6	1	0.8	0.6	0.5					
12	0.8	0.6	0.5	0.4					

02-Jul-18

10:42 AM



#### ARW ENGINEERS

ft

ft

ASD SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015 Version: April 17, 2017

Author: Wayne Young, E.I.T. Reviewed By: Troy M. Dye, S.E.

JOB TITLE: NAC Recreation ENGINEERS DESCRIPTION: Grid 2

JOB #: 18121 DESIGNER: TMD

INPUT	

#5

#6

#7

INFOI.											
	We	eight of wall =	10.0	psf				Wind (W)	Seismic (E)		
	We	eight of roof =	15.0	psf	Shea	r at wall li	ne (V <sub>u</sub> ) =	4800	900 `	lbs	Strength level
Roof Tributary length	(bear	ing & uplift) =	1.0	ft	Shear a	at wall line	$= (V_{ASD}) =$	2880	630	lbs	ASD level
	He	eight of wall =	14.0	ft							
W	ind roo	of uplift (W) =	27	psf							
Blocked shear wall?				Shear v	vall capacity pe	enalized i	f unblocke	ed			
	Field nailing (in)										
	5	Stud spacing	16	in							
							E&W	0.6W	0.7E		
Shear Panel	L		d		H/L		Red.	v (plf)	v (plf)		Shear wall Type
#1	9.0	ft	8.0	ft	1.6	OK	1.00	320	70		Type 'A'
#2		ft		ft							
#3		ft		ft							
#4		ft		ft							

LOAD COMBINATIONS AUTOMATICALLY USED IN Mr CALCULATION OUTPUT: 0.6D + 0.7E 0.6D + 0.6W

ft

ft

L= 9.0 ft 3.8 kips EQ 8.0 ft 8.8 Mo = 41.0 kips HDF = 4.7 0.6 kips

Rmax = 4.5 1.0 kips (2) 2x6 POST ..OK

USE HDU8-SDS2.5 w/ (3) 2x and 7/8" SSTB28 anchor (foundation wall height min 26")

PAB 7

FTG UPLIFT FC2.5

= (150 pcF)(2.5')(8') 12 (150 pcF)(2.5')(10') = 5.8k > 4.7k



#### ARW ENGINEERS SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

02-Jul-18 10:42 AM

ENGINEERS

JOB TITLE: NAC Recreation DESCRIPTION: Grid 2

JOB #: #REF! DESIGNER: #REF!

Nominal		Allowable	<u>e</u>		Shear wall types
W	Ε	W	E		
870	620	435	310	Type 'A'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 6 in. o.c. 2 x members @ panel edges
1290	920	645	460	Type 'B'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 4 in. o.c. 2 x members @ panel edges
1680	1200	840	600	Type 'C'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 3 in. o.c. 3 x members @ panel edges
2155	1540	1078	770	Type 'D'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 2 in. o.c. 3 x members @ panel edges
		Holdowr	1 types		
Holdown	EQ A.B.	WA.B.	EQ Cap.	W Cap.	
3.08	2.55	3.61	2.55	3.08	HDU2-SDS2.5 w/ (2) 2x and 5/8" SSTB16 anchor (foundation wall height min 14")
4.57	2.96	4.04	2.96	4.04	HDU4-SDS2.5 w/ (2) 2x and 5/8" SSTB20 anchor (foundation wall height min 18")
5.65	3.33	4.47	3.33	4.47	HDU5-SDS2.5 w/ (2) 2x and 5/8" SSTB24 anchor (foundation wall height min 22")
7.87	6.4	7.62	6.4	7.62	HDU8-SDS2.5 w/ (3) 2x and 7/8" SSTB28 anchor (foundation wall height min 26")
9.54	6.07	7.22	6.07	7.22	HDU11-SDS2.5 w/ (4) 2x and 1" SB1x30 anchor (foundation wall height min 30")

\*NOTE: A.B. capacities are based on worst case scenario of A.B. at end wall. If holdown is located at corner or midwall revise A.B. capacities based on Simpson Manual page 33 or 35.

#### Sill Plate Properties

PT Hem Fir

No. 2

ts = Fc perpendicular = 405 psi

Max Reaction =

Automatically calculates maximum compression force at end of shear wall

#### 4.5 Compression Member Size

Cm = Ct = Ci = 1 Cb= Eqn 3.10-2 2015 NDS 1.13 Fc' = 456 width = 3 (2) 2x6in depth = 5.5

kips

#### **Anchor Bolt Properties**

Max load = 7.52

bolt diameter = 0.625 in. spacing = 32 Z parallel = 930 lbs Table 12E 2015 NDS Cd = 1.6 Cm= Ct = Cg = Cdelta = Z' = 1488 v allowable = 558 plf OK

- :: USE 0.625 in dia. anchor bolt @ 32 in o.c.
- :: Use 0.229in x 3in x 3in plate washers See AWC 4.3.6.4.3

#### **Unblocked Shear Wall Sheathing Reduction**

Cub =	1			
Cub	S	tud Spacin	ng (in)	
Field nailing (in)	12	16	20	24
6	1	0.8	0.6	0.5
12	0.8	0.6	0.5	0.4



## ARW ENGINEERS ASD SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015 Version: April 17, 2017 Author: Wayne Young, E.I.T. JOB TITLE: NAC Recreation DESCRIPTION: Grid 4

02-Jul-18 10:42 AM

ENGINEERS

Reviewed By: Troy M. Dye, S.E. JOB #: 18121 DESIGNER: TMD

#6 #7 OUTPUT :		ft ft			NS AUTOMATI 0.6D + 0.6W	CALLY US	ED IN Mr CA	LCULATION			
#4 #5		ft ft		ft ft							
#3		ft		ft							
Shear Panel #1 #2	9.0	ft ft		ft ft	H/L 1.6	oĸ	Red. 1.00	v (plf) 320	v (plf) 70		Shear wall Type  Type 'A'
				33.50			E & W	0.6W	0.7E		
		ld nailing (in) Stud spacing		in in							
	Blocked	of uplift (W) = shear wall?	YES		all capacity p	enalized	if unblocke	d			
\$	He	eight of wall =	14.0	ft			- ( · A3D)	2000	500	100	710D ICVCI
f Tributary length		eight of roof = ina & uplift) =		psf ft			ine (V <sub>u</sub> ) = e (V <sub>ASD</sub> ) =	4800 2880	900 630	lbs lbs	Strength level ASD level
		eight of wall =		psf				Wind (W)	Seismic (E)	V.	

Rmax = 4.5 1.0 kips (2) 2x6 POST ..OK USE HDU8-SDS2.5 w/ (3) 2x and 7/8" SSTB28 anchor (foundation wall height min 26")

PAB7



#### ARW ENGINEERS SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

02-Jul-18 10:42 AM

ENGINEERS

JOB TITLE: NAC Recreation DESCRIPTION: Grid 4

JOB #: #REF! DESIGNER: #REF!

Nominal		Allowable	€		Shear wall types
W	E	W	E		***
870	620	435	310	Type 'A'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 6 in. o.c. 2 x members @ panel edges
1290	920	645	460	Type 'B'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 4 in. o.c. 2 x members @ panel edges
1680	1200	840	600	Type 'C'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 3 in. o.c. 3 x members @ panel edges
2155	1540	1078	770	Type 'D'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 2 in. o.c. 3 x members @ panel edges
		Holdown	types		
Holdown	EQ A.B.	WA.B.	EQ Cap.	W Cap.	
3.08	2.55	3.61	2.55	3.08	HDU2-SDS2.5 w/ (2) 2x and 5/8" SSTB16 anchor (foundation wall height min 14")
4.57	2.96	4.04	2.96	4.04	HDU4-SDS2.5 w/ (2) 2x and 5/8" SSTB20 anchor (foundation wall height min 18")
5.65	3.33	4.47	3.33	4.47	HDU5-SDS2.5 w/ (2) 2x and 5/8" SSTB24 anchor (foundation wall height min 22")
7.87	6.4	7.62	6.4	7.62	HDU8-SDS2.5 w/ (3) 2x and 7/8" SSTB28 anchor (foundation wall height min 26")
9.54	6.07	7.22	6.07	7.22	HDU11-SDS2.5 w/ (4) 2x and 1" SB1x30 anchor (foundation wall height min 30")

\*NOTE: A.B. capacities are based on worst case scenario of A.B. at end wall. If holdown is located at corner or midwall revise A.B. capacities based on Simpson Manual page 33 or 35.

#### Sill Plate Properties

PT Hem Fir

No. 2

ts = 1.5 in. Fc perpendicular = 405 psi Max Reaction = 4.5 kips

Automatically calculates maximum compression force at end of shear wall

#### **Compression Member Size**

Cm = Ct = 1 Ci = Cb = 1.13 Eqn 3.10-2 2015 NDS Fc' = 456 psi width = (2) 2x63 in depth = Max load = 7.52

#### **Anchor Bolt Properties**

bolt diameter = 0.625 in. spacing = 32 in. Z parallel = 930 lbs Table 12E 2015 NDS Cd= 1.6 Cm = 1 Ct = 1 Cg = Cdelta = 1 Z' = 1488 lbs v allowable = 558 plf OK

kips

:: USE 0.625 in dia. anchor bolt @ 32 in o.c.

:: Use 0.229in x 3in x 3in plate washers See AWC 4.3.6.4.3

#### **Unblocked Shear Wall Sheathing Reduction**

Cub =	1			
Cub	S	tud Spacin	ıg (in)	
Field nailing (in)	12	16	20	24
6	1	0.8	0.6	0.5
12	0.8	0.6	0.5	0.4

07-Aug-18

12:26 PM



#### **ARW ENGINEERS** ASD SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

Author: Wayne Young, E.I.T.

Reviewed By: Troy M. Dye, S.E. JOB #: 18121 DESIGNER: TMD

JOB #: DESIGNER:

ENGINEERS

Version: April 17, 2017 JOB TITLE: NAC Recreation DESCRIPTION: Grid 5

INPUT:											
	We	eight of wall =	10.0	psf				Wind (W)	Seismic (E)		
	We	eight of roof =	15.0	psf	Shea	ar at wall li	ne (V <sub>u</sub> ) =	2500	5500	lbs	Strength level
oof Tributary lengt	h (beari	ing & uplift) =	1.0	ft	Shear	at wall line	e (V <sub>ASD</sub> ) =	1500	3850	lbs	ASD level
	He	eight of wall =	17.0	ft							
V	Vind roc	of uplift (W) =	27	psf							
I	Blocked	shear wall?	YES	Shear w	all capacity p	enalized i	f unblocke	ed			
	Fie	ld nailing (in)	12	in							
	S	Stud spacing	16	in							
							E&W	0.6W	0.7E		
Shear Panel	L		d		H/L		Red.	v (plf)	v (plf)		Shear wall Type
#1	7.5	ft	6.5	ft	2.3	ok	0.97	41	105		Type 'A'
#2	30.5	ft	29.5	ft	0.6	ok	1.00	39	101		Type 'A'
#3		ft		ft							(1 <del>5</del> 35)(11 30)(3
#4		ft		ft							
#5		ft		ft							
#6		ft		ft							
#7		ft		ft							

			LOAD COM	BINATIO	NS AUTOMATICA	LLY USED IN Mr CAL	CULATION			
OUTPUT :			0.6D + 0.7E		0.6D + 0.6W					
L =	7.5	ft	Mr =	3.1	kips		W	EQ		
d =	6.5	ft				Mo =	5.5	12.9	kips	
						HDF =	0.4	1.5	kips	
						Rmax =	0.7	1.7	kips	(2) 2x6 POSTOK
			USE H	IDU2-S	DS2.5 w/ (2) 2x	and 5/8" SSTB16	anchor (f	oundation	wall he	eight min 14")
						PAB5	,			
L =	30.5	ft	Mr =	51.6	kips	11100	W	EQ		
d =	29.5	ft				Mo =	28.0	52.5	kips	
						HDF =	-0.8	0.0	kips	
						Rmax =	0.7	1.7	kips	(2) 2x6 POSTOK

USE HDU2-SDS2.5 w/ (2) 2x and 5/8" SSTB16 anchor (foundation wall height min 14")

PAB 5



#### ARW ENGINEERS SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

07-Aug-18 12:26 PM

ENGINEERS

JOB TITLE: NAC Recreation DESCRIPTION: Grid 5

JOB #: #REF! DESIGNER: #REF!

Nominal		Allowable	<u>e</u>		Shear wall types
W	E	W	E		
870	620	435	310	Type 'A'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 6 in. o.c. 2 x members @ panel edges
1290	920	645	460	Type 'B'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 4 in. o.c. 2 x members @ panel edges
1680	1200	840	600	Type 'C'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 3 in. o.c. 3 x members @ panel edges
2155	1540	1078	770	Type 'D'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 2 in. o.c. 3 x members @ panel edges
		Holdowr	1 types		
Holdown	EQ A.B.	WA.B.	EQ Cap.	W Cap.	
3.08	2.55	3.61	2.55	3.08	HDU2-SDS2.5 w/ (2) 2x and 5/8" SSTB16 anchor (foundation wall height min 14")
4.57	2.96	4.04	2.96	4.04	HDU4-SDS2.5 w/ (2) 2x and 5/8" SSTB20 anchor (foundation wall height min 18")
5.65	3.33	4.47	3.33	4.47	HDU5-SDS2.5 w/ (2) 2x and 5/8" SSTB24 anchor (foundation wall height min 22")
7.87	6.4	7.62	6.4	7.62	HDU8-SDS2.5 w/ (3) 2x and 7/8" SSTB28 anchor (foundation wall height min 26")
9.54	6.07	7.22	6.07	7.22	HDU11-SDS2.5 w/ (4) 2x and 1" SB1x30 anchor (foundation wall height min 30")

\*NOTE: A.B. capacities are based on worst case scenario of A.B. at end wall. If holdown is located at corner or midwall revise A.B. capacities based on Simpson Manual page 33 or 35.

#### Sill Plate Properties

PT Hem Fir

No. 2

ts = 1.5 in.

Fc perpendicular = 405 psi Max Reaction = 1.7 kips

Automatically calculates maximum compression force at end of shear wall

#### Compression Member Size

Cm = Ct = 1 Ci = 1 Cb= Eqn 3.10-2 2015 NDS 1.13 Fc' = 456 psi width = (2) 2x6 3 in depth = 5.5

in

#### **Anchor Bolt Properties**

Max load = 7.52

bolt diameter = 0.625 in. spacing = 32 Z parallel = 930 Table 12E 2015 NDS lbs Cd = 1.6 Cm = 1 Ct = Cg = 1 Cdelta = Z' = 1488 lbs v allowable = 558 plf OK

- :: USE 0.625 in dia. anchor bolt @ 32 in o.c.
- :: Use 0.229in x 3in x 3in plate washers See AWC 4.3.6.4.3

#### **Unblocked Shear Wall Sheathing Reduction**

Cub =	1		•	
Cub	S	tud Spacir	ng (in)	
Field nailing (in)	12	16	20	24
6	1	0.8	0.6	0.5
12	0.8	0.6	0.5	0.4



## ARW ENGINEERS ASD SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015 Version: April 17, 2017 Author: Wayne Young, E.I.T. JOB TITLE: NAC Recreation DESCRIPTION: Grid A

03-Aug-18 1:08 PM Reviewed By: Troy M. Dye, S.E. JOB #: 18121 DESIGNER: TMD

INPUT:											
INFOT.	We	ight of wall =	10.0	psf				Wind (W)	Seismic (E)		
		ight of roof =	15.0	psf	Shea	r at wall	line (V <sub>u</sub> ) =	1100	700	lhs	Strength level
of Tributary leng		•	2.0	ft			ie (V <sub>ASD</sub> ) =	660	490		ASD level
or rinbutary forig		ight of wall =	19.0	ft	Oricar	at wan mi	C (VASD)	000	430	IDS	AGD level
		f uplift (W) =	27	psf							
		shear wall?	YES		wall capacity p	enalized	if unblocks	ad			
		d nailing (in)	12	in	wan capacity p	CHAILECU	ii diibiocke	Ju			
		tud spacing	16	in							
		taa opaanig	10				E&W	0.6W	0.7E		
Shear Panel	l L		d		H/L		Red.	v (plf)	v (plf)		Shear wall Type
#1	22.5	ft	21.5	ft	0.8	ok	1.00	14	10		Type 'A'
#2	17.8	ft	16.8	ft	1.1	ok	1.00	14	10		Type 'A'
#3	7.0	ft	6.0	ft	2.7	ok	0.91	15	11		Type 'A'
#4		ft		ft	77000		'	207			. 160
#5		ft		ft							
#6		ft		ft							
#7		ft		ft							
				11							
		*****		10							
				MBINATI	ONS AUTOMATION	CALLY US	ED IN Mr CA	ALCULATION			
OUTPUT :			LOAD CC 0.6D + 0.	MBINATI	ONS AUTOMATION O.6D + 0.6W	CALLY US	ED IN Mr CA	ALCULATION			
	- 22.5		0.6D + 0.	MBINATI 7E	0.6D + 0.6W	CALLY US	ED IN Mr CA				
L=		ft		MBINATI 7E	0.6D + 0.6W	CALLY US		W	EQ	kino	
			0.6D + 0.	MBINATI 7E	0.6D + 0.6W	CALLY US	Mo =	W 14.2	4.4	kips	
L=		ft	0.6D + 0.	MBINATI 7E	0.6D + 0.6W	CALLY US	Mo = HDF =	W 14.2 -0.9	4.4 -1.3	kips	
L=		ft	0.6D + 0.°	MBINATI 7E 33.4	0.6D + 0.6W kips		Mo =	W 14.2	4.4		
L=		ft	0.6D + 0.	MBINATI 7E 33.4	0.6D + 0.6W		Mo = HDF =	W 14.2 -0.9	4.4 -1.3	kips	
L=	= 21.5	ft	0.6D + 0.°	MBINATI 7E 33.4 NO H	0.6D + 0.6W kips OLDOWN REG		Mo = HDF =	W 14.2 -0.9 0.3	4.4 -1.3 0.2	kips	
L = d =	= 21.5	ft ft	0.6D + 0.1 Mr =	MBINATI 7E 33.4 NO H	0.6D + 0.6W kips OLDOWN REG		Mo = HDF =	W 14.2 -0.9 0.3	4.4 -1.3 0.2	kips kips	(2) 2x6 POSTOK
L = d = L =	= 21.5	ft ft	0.6D + 0.1 Mr =	MBINATI 7E 33.4 NO H	0.6D + 0.6W kips OLDOWN REG		Mo = HDF = Rmax =	W 14.2 -0.9 0.3	4.4 -1.3 0.2	kips kips	(2) 2x6 POSTOK
L = d = L =	= 21.5	ft ft	0.6D + 0.1 Mr =	MBINATI 7E 33.4 NO H	0.6D + 0.6W kips OLDOWN REG		Mo = HDF = Rmax =	W 14.2 -0.9 0.3 W 9.9	4.4 -1.3 0.2 EQ 3.5	kips kips kips kips	(2) 2x6 POSTOK
L = d = L =	= 21.5	ft ft	0.6D + 0.1 Mr =	33.4 NO H	0.6D + 0.6W kips OLDOWN REG	QUIRED	Mo = HDF = Rmax = Mo = HDF =	W 14.2 -0.9 0.3 W 9.9 -0.7	4.4 -1.3 0.2 EQ 3.5 -1.0	kips kips	(2) 2x6 POSTOK
L = d = d =	= 21.5 = 17.8 = 16.8	ft ft	USE  USE	MBINATI 7E 33.4 NO H 20.9	0.6D + 0.6W kips  OLDOWN REC kips  OLDOWN REC	QUIRED	Mo = HDF = Rmax = Mo = HDF =	W 14.2 -0.9 0.3 W 9.9 -0.7 0.3	4.4 -1.3 0.2 EQ 3.5 -1.0 0.2	kips kips kips kips	(2) 2x6 POSTOK
L = d = d = L =	= 21.5 = 17.8 = 16.8 = 7.0	ft ft ft	0.6D + 0. Mr = USE Mr =	MBINATI 7E 33.4 NO H 20.9	0.6D + 0.6W kips OLDOWN REG kips	QUIRED	Mo = HDF = Rmax = Mo = HDF = Rmax =	W 14.2 -0.9 0.3 W 9.9 -0.7 0.3	4.4 -1.3 0.2 EQ 3.5 -1.0 0.2	kips kips kips kips	(2) 2x6 POSTOK
L = d = d =	= 21.5 = 17.8 = 16.8 = 7.0	ft ft	USE  USE	MBINATI 7E 33.4 NO H 20.9	0.6D + 0.6W kips  OLDOWN REC kips  OLDOWN REC	QUIRED	Mo = HDF = Rmax = Mo = HDF = Rmax =	W 14.2 -0.9 0.3 W 9.9 -0.7 0.3	4.4 -1.3 0.2 EQ 3.5 -1.0 0.2	kips kips kips kips	(2) 2x6 POSTOK
L = d = d = L =	= 21.5 = 17.8 = 16.8 = 7.0	ft ft ft	USE  USE	MBINATI 7E 33.4 NO H 20.9	0.6D + 0.6W kips  OLDOWN REC kips  OLDOWN REC	QUIRED	Mo = HDF = Rmax = Mo = HDF = Rmax =	W 14.2 -0.9 0.3 W 9.9 -0.7 0.3 W 2.6 -0.1	4.4 -1.3 0.2 EQ 3.5 -1.0 0.2 EQ 1.4 -0.3	kips kips kips kips kips	(2) 2x6 POSTOK
L = d = d = L =	= 21.5 = 17.8 = 16.8 = 7.0	ft ft ft	USE  USE	MBINATI 7E 33.4 NO H 20.9 NO H 3.2	0.6D + 0.6W kips  OLDOWN REC kips  OLDOWN REC	QUIRED	Mo = HDF = Rmax = Mo = HDF = Rmax =	W 14.2 -0.9 0.3 W 9.9 -0.7 0.3	4.4 -1.3 0.2 EQ 3.5 -1.0 0.2	kips kips kips kips	(2) 2x6 POSTOK



## ARW ENGINEERS SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

03-Aug-18 1:08 PM

ENGINEERS

JOB TITLE: NAC Recreation DESCRIPTION: Grid A

JOB #: #REF! DESIGNER: #REF!

Nominal		Allowable	е		Shear wall types
W	Ε	W	E		Committee Visitation (Committee Committee Comm
870	620	435	310	Type 'A'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 6 in. o.c. 2 x members @ panel edges
1290	920	645	460		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 4 in. o.c. 2 x members @ panel edges
1680	1200	840	600		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 3 in. o.c. 3 x members @ panel edges
2155	1540	1078	770		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 2 in. o.c. 3 x members @ panel edges
Halalarina		Holdowr		W 0	
Holdown		WA.B.	EQ Cap.	W Cap.	
3.08	2.55	3.61	2.55	3.08	HDU2-SDS2.5 w/ (2) 2x and 5/8" SSTB16 anchor (foundation wall height min 14")
4.57	2.96	4.04	2.96	4.04	HDU4-SDS2.5 w/ (2) 2x and 5/8" SSTB20 anchor (foundation wall height min 18")
5.65	3.33	4.47	3.33	4.47	HDU5-SDS2.5 w/ (2) 2x and 5/8" SSTB24 anchor (foundation wall height min 22")
7.87	6.4	7.62	6.4	7.62	HDU8-SDS2.5 w/ (3) 2x and 7/8" SSTB28 anchor (foundation wall height min 26")
9.54	6.07	7.22	6.07	7.22	HDU11-SDS2.5 w/ (4) 2x and 1" SB1x30 anchor (foundation wall height min 30")

\*NOTE: A.B. capacities are based on worst case scenario of A.B. at end wall. If holdown is located at corner or midwall revise A.B. capacities based on Simpson Manual page 33 or 35.

#### Sill Plate Properties

PT Hem Fir

No. 2

 $\begin{array}{cccc} ts = & 1.5 & \text{in.} \\ \text{Fc perpendicular} = & 405 & \text{psi} \\ \text{Max Reaction} = & 0.3 & \text{kips} \end{array}$ 

Automatically calculates maximum compression force at end of shear wall

#### **Compression Member Size**

Cm = Ct = 1 Ci = 1 Cb= 1.13 Eqn 3.10-2 2015 NDS Fc' = 456 psi width = 3 (2) 2x6 in 5.5 depth = Max load = 7.52

#### **Anchor Bolt Properties**

bolt diameter = 0.625 in. spacing = 32 in. Z parallel = 930 Table 12E 2015 NDS lbs Cd = 1.6 Cm = 1 Ct = 1 Cg = Cdelta = Z' = 1488 lbs v allowable = 558 plf

:: USE 0.625 in dia. anchor bolt @ 32 in o.c.

:: Use 0.229in x 3in x 3in plate washers See AWC 4.3.6.4.3

#### Unblocked Shear Wall Sheathing Reduction

Cub =	1			
Cub	8	tud Spacir	ıg (in)	
Field nailing (in)	12	16	20	24
6	1	0.8	0.6	0.5
12	0.8	0.6	0.5	0.4



## ARW ENGINEERS ASD SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

Author: Wayne Young, E.I.T.

10:42 AM

kips (4) 2x6 POST ..OK

02-Jul-18

ENGINEERS

Version: April 17, 2017 JOB TITLE: NAC Recreation DESCRIPTION: Grid A.7

8.0 ft

7.2 ft

Reviewed By: Troy M. Dye, S.E. JOB #: 18121 DESIGNER: TMD

INPUT:	3,200										
		eight of wall =		psf				Wind (W)	Seismic (E)		
		eight of roof =		psf	Shea	ar at wall I	ne $(V_u) =$	15100	9700	lbs	Strength level
of Tributary length	(bear	ing & uplift) =	2.0	ft	Shear	at wall line	$= (V_{ASD}) =$	9060	6790	lbs	ASD level
	He	eight of wall =	17.0	ft							
V	ind roo	of uplift (W) =	27	psf							
В	locked	shear wall?	YES	Shear w	all capacity p	enalized i	f unblocke	ed .			
	Fie	ld nailing (in)	12	in							
	5	Stud spacing	16	in							
							E&W	0.6W	0.7E		
Shear Panel	L		d		H/L		Red.	v (plf)	v (plf)		Shear wall Type
#1	8.8	ft	8.0	ft	1.9	OK	1.00	539	404		Type 'B'
#2	8.0	ft	7.2	ft	2.1	ok	0.98	548	411		Type 'B'
#3		ft		ft							<i>(*.</i> *)
#4		ft		ft							
#5		ft		ft							
#6		ft		ft							
#7		ft		ft							
			LOAD CO	MBINATIC	NS AUTOMATI	CALLY USE	D IN Mr CA	LCULATION			
OUTPUT:			0.6D + 0.		0.6D + 0.6W						
L=	8.8	ft	Mr=	4.6	kips			W	EQ		
d =	8.0	ft					Mo =	81.9	60.5	kips	3

HDF = 9.8 7.1 kips
Rmax = 9.3 6.9 kips (4) 2x6 POST ..OK
USE HDU11-SDS2.5 w/ (4) 2x and 1" \$B4x30 anchor (foundation wall height min 30")

9.2

74.4

6.9

EQ

55.0

kips

Rmax =

Mo =

USE HDU11-SDS2.5 w/ (4) 2x and 1" SB1x30 anchor (foundation wall height min 30")

FTG UPLIFT

Mr =

3.8

kips

== (150 por)(2.5')(3') + == (150 por)(5')(5') = 9.5" ≈ 9.7" ON

USE F5 × 24' THICK



## ARW ENGINEERS SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

02-Jul-18 10:42 AM

ENGINEERS

JOB TITLE: NAC Recreation DESCRIPTION: Grid A.7

JOB #: #REF! DESIGNER: #REF!

Nominal	Allowable				Shear wall types
W	E	W	E		
870	620	435	310	Type 'A'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 6 in. o.c. 2 x members @ panel edges
1290	920	645	460	Type 'B'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 4 in. o.c. 2 x members @ panel edges
1680	1200	840	600		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 3 in. o.c. 3 x members @ panel edges
2155	1540	1078	770	Type 'D'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 2 in. o.c. 3 x members @ panel edges
Holdown types					
Holdown	EQ A.B.	WA.B.	EQ Cap.	W Cap.	
3.08	2.55	3.61	2.55	3.08	HDU2-SDS2.5 w/ (2) 2x and 5/8" SSTB16 anchor (foundation wall height min 14")
4.57	2.96	4.04	2.96	4.04	HDU4-SDS2.5 w/ (2) 2x and 5/8" SSTB20 anchor (foundation wall height min 18")
5.65	3.33	4.47	3.33	4.47	HDU5-SDS2.5 w/ (2) 2x and 5/8" SSTB24 anchor (foundation wall height min 22")
7.87	6.4	7.62	6.4	7.62	HDU8-SDS2.5 w/ (3) 2x and 7/8" SSTB28 anchor (foundation wall height min 26")
11.2	8.3	9.9	8.3	9.9	HDU11-SDS2.5 w/ (4) 2x and 1" SB1x30 anchor (foundation wall height min 30")

\*NOTE: A.B. capacities are based on worst case scenario of A.B. at end wall. If holdown is located at corner or midwall revise A.B. capacities based on Simpson Manual page 33 or 35.

#### Sill Plate Properties

PT Hem Fir

No. 2

 $\begin{array}{cccc} ts = & 1.5 & \text{in.} \\ \text{Fc perpendicular} = & 405 & \text{psi} \\ \text{Max Reaction} = & 9.3 & \text{kips} \end{array}$ 

Automatically calculates maximum compression force at end of shear wall

#### Compression Member Size

Cm = Ct = 1 Ci = 1 Cb= 1.06 Eqn 3.10-2 2015 NDS Fc' = 430 (4) 2x6 width = 6 depth = 5.5 Max load = 14.20

#### **Anchor Bolt Properties**

boit diameter =	0.625	ın.	
spacing =	24	in.	
Z parallel =	930	lbs	Table 12E 2015 NDS
Cd =	1.6		
Cm =	1		
Ct =	1		
Cg =	1		
Cdelta =	1		
Z' =	1488	lbs	
v allowable =	744	plf	ок

- :: USE 0.625 in dia. anchor bolt @ 24 in o.c.
- :: Use 0.229in x 3in x 3in plate washers See AWC 4.3.6.4.3

#### Unblocked Shear Wall Sheathing Reduction

Cub =	1						
Cub	Stud Spacing (in)						
Field nailing (in)	12	16	20	24			
6	1	0.8	0.6	0.5			
12	0.8	0.6	0.5	0.4			



## ARW ENGINEERS ASD SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015 Version: April 17, 2017 Author: Wayne Young JOB TITLE: NAC Recreation DESCRIPTION: Grid H.3

03-Aug-18 1:08 PM

ENGINEERS

Reviewed By: Troy M. Dye JOB #: 18121 DESIGNER: TMD

INPUT:											
	Wei	ight of wall =	10.0	psf				Wind (W)	Seismic (E)		
	Wei	ight of roof =	15.0	psf	Shear	at wall l	ine (V <sub>u</sub> ) =	15400	10100		Strength level
of Tributary length	ı (bearii	ng & uplift) =	2.0	ft	Shear a	t wall lin	e (V <sub>ASD</sub> ) =	9240	7070		ASD level
	Hei	ight of wall =	19.0	ft							
V	lind roo	f uplift (W) =	27	psf							
E		shear wall?	YES	Shear w	all capacity pe	nalized	f unblocke	ed			
		d nailing (in)	12	in							
	S	tud spacing	16	in							
VENTO CONTROL CONTROL							E & W	0.6W	0.7E		
Shear Panel	L		d		H/L		Red.	v (plf)	v (plf)		Shear wall Type
#1	20.0	ft		ft	1.0	OK	1.00	194	148		Type 'A'
#2	27.7	ft	26.7	ft	0.7	OK	1.00	194	148		Type 'A'
#3		ft		ft							
#4		ft		ft							
#5		ft		ft							
#6		ft		ft							
#7		ft		ft							
OUTDUT :					NS AUTOMATIC	ALLY US	ED IN Mr CA	LCULATION			
OUTPUT:			0.6D + 0.7	'E	0.6D + 0.6W						
L =	20.0	ft	Mr =	26.4	kips			W	EQ		
d =	19.0	ft			•)		Mo =	80.1	56.3	kips	
							HDF =	2.8	1.6	kips	
							Rmax =	3.7	2.8		(4) 2x6 POSTOK
			USE	HDU2-S	DS2.5 w/ (2) 2	2x and 5	/8" SSTB1	Γ6 anchor (f			eight min 14")
y <b>≢</b> 992 ags	07.7						PABS				rena <del>-</del> predu-renamento - Fili d <b>e</b>
L=	27.7		Mr=	50.6	kips			VV	EQ		
d =	26.7	π					Mo =	114.4	78.0	kips	
							HDF =	2.4	1.0	kips	
				LIBLIA	D00 F / /5: 5		Rmax =	3.7	2.8		(4) 2x6 POSTOK
			USE	HDU2-S	DS2.5 W/ (2) 2	x and 5	8" SSTB	f6 anchor (f	oundation w	all h	eight min 14")
							DAR	-			

PABS



### ARW ENGINEERS SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

03-Aug-18 1:08 PM

ENGINEERS

JOB TITLE: NAC Recreation DESCRIPTION: Grid H.3

JOB #: #REF! DESIGNER: #REF!

Nominal		Allowable	<u>e</u>		Shear wall types
W	E	W	E		
870	620	435	310	Type 'A'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 6 in. o.c. 2 x members @ panel edges
1290	920	645	460		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 4 in. o.c. 2 x members @ panel edges
1680	1200	840	600		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 3 in. o.c. 3 x members @ panel edges
2155	1540	1078	770	Type 'D'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 2 in. o.c. 3 x members @ panel edges
		Holdowr	types		
Holdown	EQ A.B.	WA.B.	EQ Cap.	W Cap.	
3.08	2.55	3.61	2.55	3.08	HDU2-SDS2.5 w/ (2) 2x and 5/8" SSTB16 anchor (foundation wall height min 14")
4.57	2.96	4.04	2.96	4.04	HDU4-SDS2.5 w/ (2) 2x and 5/8" SSTB20 anchor (foundation wall height min 18")
5.65	3.33	4.47	3.33	4.47	HDU5-SDS2.5 w/ (2) 2x and 5/8" SSTB24 anchor (foundation wall height min 22")
7.87	6.4	7.62	6.4	7.62	HDU8-SDS2.5 w/ (3) 2x and 7/8" SSTB28 anchor (foundation wall height min 26")
11.2	8.3	8.3	8.3	8.3	HDU11-SDS2.5 w/ (4) 2x and 1" SB1x30 anchor (foundation wall height min 30")

\*NOTE: A.B. capacities are based on worst case scenario of A.B. at end wall. If holdown is located at corner or midwall revise A.B. capacities based on Simpson Manual page 33 or 35.

#### Sill Plate Properties

PT Hem Fir

No. 2

ts = 1.5 in. Fc perpendicular = 405 psi

Max Reaction = 3.7 kips Automatically calculates maximum compression force at end of shear wall

#### **Compression Member Size**

Cm = Ct= 1 Ci = Cb = 1.06 Eqn 3.10-2 2015 NDS Fc' = 430 psi width = 6 (4) 2x6 in depth = 5.5 Max load = 14.20

#### **Anchor Bolt Properties**

bolt diameter = 0.625 in. spacing = 32 in. Z parallel = 930 Table 12E 2015 NDS lbs Cd= 1.6 Cm = 1 Ct = Cg = Cdelta = Z' = 1488 lbs v allowable = 558 plf OK

:: USE 0.625 in dia. anchor bolt @ 32 in o.c.

:: Use 0.229in x 3in x 3in plate washers See AWC 4.3.6.4.3

#### Unblocked Shear Wall Sheathing Reduction

Cub =	1			
Cub	S	tud Spacin	ıg (in)	
Field nailing (in)	12	16	20	24
6	1	0.8	0.6	0.5
12	0.8	0.6	0.5	0.4

02-Jul-18

10:42 AM



#### ARW ENGINEERS ASD SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

Version: April 17, 2017 Author: Wayne Young, E.I.T. Reviewed By: Troy M. Dye, S.E. JOB #: 18121 DESIGNER: TMD

JOB TITLE: NAC Recreation
DESCRIPTION: Grid 3 ENGINEERS

		ight of wall = ight of roof =	10.0 15.0	psf psf	Sher	ar at wall li	ne (\/ \ -	Wind (W) 10900	Seismic (E) 3400	llaa	Chan ath lavel
f Tributary len		· · · · · · · · · · · · · · · · · · ·	30.0	ft						lbs	
or inbutary len				LOGO IDO NOD IEVEL						ASD level	
		ight of wall =	14.0	ft							
		f uplift (W) =	27	psf							
		shear wall?	YES		all capacity p	enalized if	unblocke	ed			
		d nailing (in)	12	in							
	S	tud spacing	16	in							
01					177.27		E&W	0.6W	0.7E		
Shear Pane		120	d	2	H/L	27.5	Red.	v (plf)	v (plf)		Shear wall Type
#1	21.0	ft	20.0	ft	0.7	ok	1.00	311	113		Type 'A'
#2		ft		ft							
#3		ft		ft							
#4		ft		ft							
#5		ft		ft							
#6		ft		ft							
#7		ft		ft							

OUTPUT: 0.6D + 0.7E 0.6D + 0.6W L = 21.0 ft Mr = 78.1 kips W EQ d = 20.0 ft Mo= 198.7 33.3 kips HDF = -2.2 6.0 kips Rmax = 4.4 1.6 kips (2) 2x6 POST ..OK

USE HDU8-SDS2.5 w/ (3) 2x and 7/8" SSTB28 anchor (foundation wall height min 26")

PAB 7



#### ARW ENGINEERS SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

02-Jul-18 10:42 AM

ENGINEERS

JOB TITLE: NAC Recreation DESCRIPTION: Grid 3

JOB #: #REF! DESIGNER: #REF!

Nominal		Allowable	<u>e</u>		Shear wall types
W	E	W	E		
870	620	435	310	Type 'A'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 6 in. o.c. 2 x members @ panel edges
1290	920	645	460	Type 'B'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 4 in. o.c. 2 x members @ panel edges
1680	1200	840	600		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 3 in. o.c. 3 x members @ panel edges
2155	1540	1078	770		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 2 in. o.c. 3 x members @ panel edges
		Holdowr	types		
Holdown	EQ A.B.	WA.B.	EQ Cap.	W Cap.	
3.08	2.55	3.61	2.55	3.08	HDU2-SDS2.5 w/ (2) 2x and 5/8" SSTB16 anchor (foundation wall height min 14")
4.57	2.96	4.04	2.96	4.04	HDU4-SDS2.5 w/ (2) 2x and 5/8" SSTB20 anchor (foundation wall height min 18")
5.65	3.33	4.47	3.33	4.47	HDU5-SDS2.5 w/ (2) 2x and 5/8" SSTB24 anchor (foundation wall height min 22")
7.87	6.4	7.62	6.4	7.62	HDU8-SDS2.5 w/ (3) 2x and 7/8" SSTB28 anchor (foundation wall height min 26")
9.54	6.07	7.22	6.07	7.22	HDU11-SDS2.5 w/ (4) 2x and 1" SB1x30 anchor (foundation wall height min 30")

\*NOTE: A.B. capacities are based on worst case scenario of A.B. at end wall. If holdown is located at corner or midwall revise A.B. capacities based on Simpson Manual page 33 or 35.

#### Sill Plate Properties

PT Hem Fir

No. 2

ts = in.

Fc perpendicular = 405 psi Max Reaction = 4.4 kips

Automatically calculates maximum compression force at end of shear wall

#### Compression Member Size

Cm = Ct = Ci = 1

Cb= Eqn 3.10-2 2015 NDS 1.13

(2) 2x6

Fc' = 456 psi width = 3 in

depth = 5.5 in Max load = 7.52

#### **Anchor Bolt Properties**

bolt diameter = 0.625 in. spacing = 32 Z parallel = 930 Table 12E 2015 NDS lbs Cd = 1.6 Cm = Ct = Cg= 1 Cdelta = Z' = 1488 lbs v allowable = 558 plf OK

:: USE 0.625 in dia. anchor bolt @ 32 in o.c.

:: Use 0.229in x 3in x 3in plate washers See AWC 4.3.6.4.3

#### **Unblocked Shear Wall Sheathing Reduction**

Cub =	1			
Cub	S	tud Spacir	ng (in)	
Field nailing (in)	12	16	20	24
6	1	0.8	0.6	0.5
12	0.8	0.6	0.5	0.4

02-Jul-18

10:42 AM



## **ARW ENGINEERS** ASD SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015 Version: April 17, 2017 Author: Wayne Young, E.I.T. JOB TITLE: NAC Recreation DESCRIPTION: Grid 6

Reviewed By: Troy M. Dye, S.E. JOB #: 18121 DESIGNER: TMD

JOB #: DESIGNER:

ENGINEERS

INPUT:	9999										
	Wei	ight of wall =	10.0	psf				Wind (W)	Seismic (E)		
	Wei	ight of roof =	15.0	psf	Shea	ar at wall li	ne (V <sub>u</sub> ) =	10900	3400	lbs	Strength level
oof Tributary length	n (bearii	ng & uplift) =	30.0	ft	Shear	at wall line	$= (V_{ASD}) =$	6540	2380	lbs	ASD level
	Hei	ight of wall =	14.0	ft							
V	lind roo	f uplift (W) =	27	psf							
E	Blocked	shear wall?	YES	Shear v	vall capacity p	enalized if	f unblocke	ed			
	Fiel	d nailing (in)	12	in							
	S	tud spacing	16	in							
							E&W	0.6W	0.7E		
Shear Panel	L		d		H/L		Red.	v (plf)	v (plf)		Shear wall Type
#1	12.0	ft	11.0	ft	1.2	oK	1.00	284	103		Type 'A'
#2	11.0	ft	10.0	ft	1.3	oK	1.00	284	103		Type 'A'
#3		ft		ft							
#4		ft		ft							
#5		ft		ft							
#6		ft		ft							
#7		ft		ft							

			LOAD COM	BINATIO	NS AUTOMATICALL	Y USED IN Mr CAL	CULATION					
OUTPUT:		0.6D + 0.7E		0.6D + 0.6W	0.6D + 0.6W							
L =	12.0	ft	Mr =	25.5	kips		W	EQ				
d =	11.0	ft				Mo =	82.8	17.4	kips			
						HDF =	5.2	-0.7	kips			
						Rmax =	4.0	1.4	kips	(2) 2x6 POSTOK		
			USE F	IDU8-S	DS2.5 w/ (3) 2x a	nd 7/8" SSTB28	anchor (f	oundation	wall he	eight min 26")		
¥						PAB 7				State → Visit Administration in Section 11.0 Project 1.0 Project		
L =		ft	Mr =	21.4	kips	17.65	W	EQ				
d =	10.0	ft				Mo =	73.2	15.9	kips			
						HDF =	5.2	-0.5	kips			
						Rmax =	4.0	14	kips	(2) 2x6 POST OK		

 $Rmax = 4.0 \\ 1.4 \\ kips (2) 2x6 POST ..OK \\ USE HDU8-SDS2.5 w/ (3) 2x and 7/8" $SSTB28 anchor (foundation wall height min 26")$ 

PAB 7



#### ARW ENGINEERS SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

02-Jul-18 10:42 AM

ENGINEERS

JOB TITLE: NAC Recreation DESCRIPTION: Grid 6

JOB#: #REF! DESIGNER: #REF!

					v
<u>Nominal</u>		Allowable	<u>e</u>		Shear wall types
W	E	W	E		
870	620	435	310	Type 'A'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 6 in. o.c. 2 x members @ panel edges
1290	920	645	460		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 4 in. o.c. 2 x members @ panel edges
1680	1200	840	600		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 3 in. o.c. 3 x members @ panel edges
2155	1540	1078	770	Type 'D'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 2 in. o.c. 3 x members @ panel edges
		Holdowr	ı types		
Holdown	EQ A.B.	WA.B.	EQ Cap.	W Cap.	
3.08	2.55	3.61	2.55	3.08	HDU2-SDS2.5 w/ (2) 2x and 5/8" SSTB16 anchor (foundation wall height min 14")
4.57	2.96	4.04	2.96	4.04	HDU4-SDS2.5 w/ (2) 2x and 5/8" SSTB20 anchor (foundation wall height min 18")
5.65	3.33	4.47	3.33	4.47	HDU5-SDS2.5 w/ (2) 2x and 5/8" SSTB24 anchor (foundation wall height min 22")
7.87	6.4	7.62	6.4	7.62	HDU8-SDS2.5 w/ (3) 2x and 7/8" SSTB28 anchor (foundation wall height min 26")
9.54	6.07	7.22	6.07	7.22	HDU11-SDS2.5 w/ (4) 2x and 1" SB1x30 anchor (foundation wall height min 30")

\*NOTE: A.B. capacities are based on worst case scenario of A.B. at end wall. If holdown is located at corner or midwall revise A.B. capacities based on Simpson Manual page 33 or 35.

#### Sill Plate Properties

PT Hem Fir

No. 2

ts = 1.5 in.

Fc perpendicular = 405

psi Max Reaction = 4.0

Automatically calculates maximum compression force at end of shear wall

#### Compression Member Size

Cm = Ct = 1 Ci= 1

Cb= Eqn 3.10-2 2015 NDS 1.13

Fc' = 456 psi width = 3 in

(2) 2x6 depth = 5.5

Max load = 7.52

#### **Anchor Bolt Properties**

bolt diameter = 0.625 in. spacing = 32 in.

Z parallel = 930 lbs Table 12E 2015 NDS

Cd = 1.6 Cm =

Ct =

Cg = Cdelta =

Z' = 1488 lbs

v allowable = 558 plf OK

:: USE 0.625 in dia. anchor bolt @ 32 in o.c.

:: Use 0.229in x 3in x 3in plate washers

See AWC 4.3.6.4.3

#### Unblocked Shear Wall Sheathing Reduction

Cub =	1		J				
Cub	·s	Stud Spacing (in)					
Field nailing (in)	12	16	20	24			
6	1	0.8	0.6	0.5			
12	0.8	0.6	0.5	0.4			



## ARW ENGINEERS ASD SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015 Version: April 17, 2017 Author: Wayne Young JOB TITLE: NAC Recreation DESCRIPTION: Grid J

02-Jul-18 10:42 AM

ENGINEERS

Reviewed By: Troy M. Dye JOB #: 18121 DESIGNER: TMD

INPUT:											-
	We	eight of wall =	10.0	psf				Wind (W)	Seismic (E)		
		eight of roof =	15.0	psf	Shea	r at wall l	ine (V <sub>u</sub> ) =	3400	3500	lbs	Strength level
oof Tributary leng	th (beari	ing & uplift) =	2.0	ft	Shear	at wall lin	e (V <sub>ASD</sub> ) =	2040	2450	lbs	ASD level
	He	eight of wall =	16.0	ft			a Accorded				
	Wind roc	of uplift (W) =	27	psf							
		shear wall?	YES	Shear w	vall capacity p	enalized i	f unblocke	ed			
	Fie	ld nailing (in)	12	in							
	S	Stud spacing	16	in							
							E & W	0.6W	0.7E		
Shear Pane			d		H/L		Red.	v (plf)	v (plf)		Shear wall Type
#1	16.0	ft	15.0	ft	1.0	ok	1.00	85	102		Type 'A'
#2	8.0	ft	7.0	ft	2.0	OK	1.00	85	102		Type 'A'
#3		ft		ft							
#4		ft		ft							
#5		ft		ft							
#6		ft		ft							
#7		ft		ft							
-											
OUTPUT:			0.6D + 0.1		ONS AUTOMATIC	CALLY USE	ED IN Mr CA	LCULATION			
OUIFUI.			0.60 + 0.	<i>/</i> E	0.6D + 0.6W						
L:	= 16.0	ft	Mr =	14.6	kips			W	EQ		
d =	15.0	ft					Mo =	25.9	26.1	kips	
							HDF =	0.8	0.8	kips	
							Rmax =	1.4	1.6		(2) 2x6 POSTOI
			USE	HDU2-S	SDS2.5 w/ (2)	2x and 5			foundation w	all h	eight min 14")
							PABS	ò			
L:		ft	Mr =	3.6	kips			W	EQ		
d =	7.0	ft					Mo =	11.9	13.1	kips	
							HDF =	1.2	1.3	kips	
						_	Rmax =	1.4	1.6		(2) 2x6 POSTOI
			USE	HDU2-S	SDS2.5 w/ (2)	2x and 5	/8" SSTB	f6 anchor (	foundation w	all h	eight min 14")
							DA-2 G	5			

PABS



## ARW ENGINEERS SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

02-Jul-18 10:42 AM

ENGINEERS

JOB TITLE: NAC Recreation DESCRIPTION: Grid J

JOB #: #REF! DESIGNER: #REF!

Nominal		Allowable	<u>e</u>		Shear wall types
W	E	W	E		14/09/2016/09/2016/09/2016/09/30 • • • • • • • • • • • • • • • • • • •
870	620	435	310	Type 'A'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 6 in. o.c. 2 x members @ panel edges
1290	920	645	460		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 4 in. o.c. 2 x members @ panel edges
1680	1200	840	600		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 3 in. o.c. 3 x members @ panel edges
2155	1540	1078	770		shear wall 15/32 in. plywood sheathing w/ 10d nails @ 2 in. o.c. 3 x members @ panel edges
850 VS 00		Holdowr			
Holdown	EQ A.B.	W A.B.	EQ Cap.	W Cap.	
3.08	2.55	3.61	2.55	3.08	HDU2-SDS2.5 w/ (2) 2x and 5/8" SSTB16 anchor (foundation wall height min 14")
4.57	2.96	4.04	2.96	4.04	HDU4-SDS2.5 w/ (2) 2x and 5/8" SSTB20 anchor (foundation wall height min 18")
5.65	3.33	4.47	3.33	4.47	HDU5-SDS2.5 w/ (2) 2x and 5/8" SSTB24 anchor (foundation wall height min 22")
7.87	6.4	7.62	6.4	7.62	HDU8-SDS2.5 w/ (3) 2x and 7/8" SSTB28 anchor (foundation wall height min 26")
11.2	6.07	7.22	6.07		HDU11-SDS2.5 w/ (4) 2x and 1" SB1x30 anchor (foundation wall height min 30")

\*NOTE: A.B. capacities are based on worst case scenario of A.B. at end wall. If holdown is located at corner or midwall revise A.B. capacities based on Simpson Manual page 33 or 35.

#### Sill Plate Properties

PT Hem Fir

No. 2

ts = 1.5 in.
Fc perpendicular = 405 psi
Max Reaction = 1.6 kips

Automatically calculates maximum compression force at end of shear wall

#### Compression Member Size

Cm = Ct = 1 Ci = 1 Cb = Eqn 3.10-2 2015 NDS 1.13 Fc' = 456 psi width = (2) 2x6 3 in depth = 5.5 Max load = 7.52

#### **Anchor Bolt Properties**

bolt diameter = 0.625 in. spacing = 32 in. Z parallel = 930 Ibs Table 12E 2015 NDS Cd= 1.6 Cm = Ct = Cg= Cdelta = Z' = 1488 lbs v allowable = 558 OK

:: USE 0.625 in dia. anchor bolt @ 32 in o.c.

:: Use 0.229in x 3in x 3in plate washers See AWC 4.3.6.4.3

#### Unblocked Shear Wall Sheathing Reduction

Cub =	1			
Cub	Stud Spacing (in)			
Field nailing (in)	12	16	20	24
6	1	0.8	0.6	0.5
12	8.0	0.6	0.5	0.4

03-Aug-18 1:10 PM



## **ARW ENGINEERS** ASD SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015 Version: April 17, 2017 Author: Wayne Young JOB TITLE: NAC Recreation DESCRIPTION: Grid K

ENGINEERS

Reviewed By: Troy M. Dye JOB #: 18121 DESIGNER: TMD JOB #: DESIGNER:

INPUT:				10000 <b>-</b>							
		ight of wall =		psf				Wind (W)	Seismic (E)		
Weight of roof =			psf			ine $(V_u) =$	3000	2400		Strength level	
of Tributary len				ft	Shear a	t wall line	$e(V_{ASD}) =$	1800	1680	lbs	ASD level
		ight of wall =		ft							
		of uplift (W) =		psf							
		shear wall?	YES		all capacity pe	enalized i	f unblocke	ed			
		ld nailing (in)		in							
	5	Stud spacing	16	in			72 V 1988				
O. D.							E&W	0.6W	0.7E		
Shear Pane			d	20	H/L	1220	Red.	v (plf)	v (plf)		Shear wall Type
#1	34.0	ft		ft	0.5	oK	1.00	42	39		Type 'A'
#2	9.0	ft	8.0	ft	1.8	ok	1.00	42	39		Type 'A'
#3		ft		ft							
#4		ft		ft							
#5		ft		ft							
#6		ft		ft							
#7		ft		ft							
					NS AUTOMATIC	ALLY USE	ED IN Mr CA	ALCULATION			
OUTPUT :			0.6D + 0.7	Έ	0.6D + 0.6W						
L		ft	Mr =	65.9	kips			W	EQ		
d	= 33.0	ft					Mo =	41.5	21.3	kips	
							HDF =	-0.7	-1.4	kips	
							Rmax =	0.7	0.6	kips	(2) 2x6 POSTOK
			USE	NO HO	LDOWN REG	UIRED					
E	= 9.0	ft	Mr =	4.6	kips			W	EQ		
_	= 8.0	ft					Mo =	7.3	5.6	kips	
d									1200	TOTAL STREET	
_							HDF =	0.3	0.1	kips	

PAB 5



## ARW ENGINEERS SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

03-Aug-18 1:10 PM

ENGINEERS

JOB TITLE: NAC Recreation DESCRIPTION: Grid K

JOB #: #REF! DESIGNER: #REF!

<u>Nominal</u>		Allowable	<u>e</u>		Shear wall types
W	E	W	E		
870	620	435	310	Type 'A'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 6 in. o.c. 2 x members @ panel edges
1290	920	645	460	Type 'B'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 4 in. o.c. 2 x members @ panel edges
1680	1200	840	600	Type 'C'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 3 in. o.c. 3 x members @ panel edges
2155	1540	1078	770	Type 'D'	shear wall 15/32 in. plywood sheathing w/ 10d nails @ 2 in. o.c. 3 x members @ panel edges
		Holdowr	types		
Holdown	EQ A.B.	WA.B.	EQ Cap.	W Cap.	
3.08	2.55	3.61	2.55	3.08	HDU2-SDS2.5 w/ (2) 2x and 5/8" SSTB16 anchor (foundation wall height min 14")
4.57	2.96	4.04	2.96	4.04	HDU4-SDS2.5 w/ (2) 2x and 5/8" SSTB20 anchor (foundation wall height min 18")
5.65	3.33	4.47	3.33	4.47	HDU5-SDS2.5 w/ (2) 2x and 5/8" SSTB24 anchor (foundation wall height min 22")
7.87	6.4	7.62	6.4	7.62	HDU8-SDS2.5 w/ (3) 2x and 7/8" SSTB28 anchor (foundation wall height min 26")
11.2	6.07	7.22	6.07	7.22	HDU11-SDS2.5 w/ (4) 2x and 1" SB1x30 anchor (foundation wall height min 30")

\*NOTE: A.B. capacities are based on worst case scenario of A.B. at end wall. If holdown is located at corner or midwall revise A.B. capacities based on Simpson Manual page 33 or 35.

#### Sill Plate Properties

PT Hem Fir

No. 2

Max load =

ts = 1.5 in.
Fc perpendicular = 405 psi
Max Reaction = 0.7 kips

Automatically calculates maximum compression force at end of shear wall

#### **Compression Member Size**

Cm = Ct = Ci = 1 Cb= Eqn 3.10-2 2015 NDS 1.13 Fc' = 456 psi width = 3 (2) 2x6depth = 5.5 in

#### Anchor Bolt Properties

7.52

bolt diameter = 0.625 in. spacing = 32 in. Z parallel = 930 lbs Table 12E 2015 NDS Cd = 1.6 Cm = Ct = Cg = Cdelta = 1 Z' = 1488 lbs v allowable = 558 OK plf

kips

:: USE 0.625 in dia. anchor bolt @ 32 in o.c.

:: Use 0.229in x 3in x 3in plate washers See AWC 4.3.6.4.3

#### Unblocked Shear Wall Sheathing Reduction

Cub =	1				
Cub	Cub Stud Spacing (in				
Field nailing (in)	12	16	20	24	
6	1	8.0	0.6	0.5	
12	0.8	0.6	0.5	0.4	

**WALLS** 



#### WALLS

#### WALL A: DEAD LOAD

5	psf
2	psf
2	psf
	psf
	psf
2	psf
	psf
11	psf
11	psf
	2 2 2

Comm	enis			

#### WALL B: DEAD LOAD

Framing:		psf
Batting/Blown Insulation:		psf
Sheathing:		psf
Veneer:		psf
Mechanical Ducts/Misc.:		psf
Gypsum Board:		psf
Collateral:		psf
Total Dead Load:	0	psf
Seismic Mass Dead Load:	0	psf

#### WALL C: DEAD LOAD

psf	Framing:
psf	Batting/Blown Insulation:
psf	Sheathing:
psf	Veneer:
psf	Mechanical Ducts/Misc.:
psf	Gypsum Board:
psf	Collateral:
0 psf	Total Dead Load:
0 psf	Seismic Mass Dead Load:



#### WALL D: DEAD LOAD

psf		Framing:
psf		Batting/Blown Insulation:
psf		Sheathing:
psf		Veneer:
psf		Mechanical Ducts/Misc.:
psf		Gypsum Board:
psf		Collateral:
psf	0	Total Dead Load:
psf	0	Seismic Mass Dead Load:

# Comments

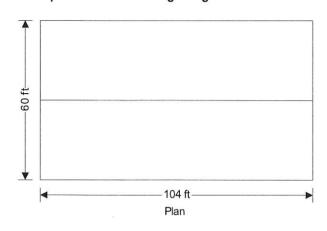
Tekla Tedds	Project				Job Ref.	C2
ARW Engineers	Section		Sheet no./rev.	Sheet no./rev.		
	Calc. by	Date 7/2/2018	Chk'd by	Date	App'd by	Date

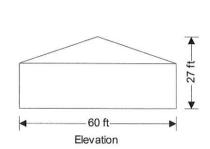
#### WIND LOADING (ASCE7-10)

In accordance with ASCE7-10 incorporating Errata No. 1 and Errata No. 2

Using the components and cladding design method

Tedds calculation version 2.0.20





#### **Building data**

Type of roof	Gable
Length of building	b = <b>104.00</b> ft
Width of building	d = <b>60.00</b> ft
Height to eaves	H = 17.00 ft
Pitch of roof	$\alpha_0$ = <b>18.5</b> deg
Mean height	h = <b>22.02</b> ft

#### General wind load requirements

General wind load requirements	
Basic wind speed	V = <b>115.0</b> mph
Risk category	II
Velocity pressure exponent coeff (Table 26.6-1)	$K_d = 0.85$
Exposure category (cl.26.7.3)	С
Enclosure classification (cl.26.10)	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1)	$GC_{pi_p} = 0.18$
Internal pressure coef –ve (Table 26.11-1)	$GC_{pi_n} = -0.18$
Gust effect factor	$G_f = 0.85$
Topography	

Topography factor not significant	$K_{zt} = 1.0$
-----------------------------------	----------------

#### Velocity pressure

Velocity pressure coefficient (T.30.3-1)	$K_z = 0.92$
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1psf/mph^2 = 26.4 psf$

#### Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q<sub>i</sub> = **26.36** psf

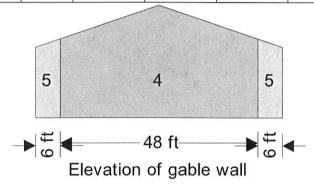
#### Equations used in tables

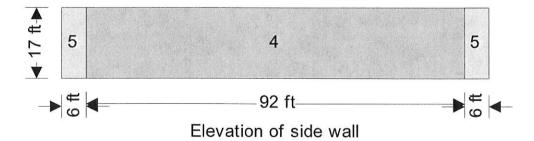
Net pressure  $p = q_h \times [GC_p - GC_{pi}]$ 

Tekla Tekla	Project			лина простинуванского до причина причи	Job Ref.	<b>C</b> 3
ARW Engineers	Section				Sheet no./rev.	
	Calc. by	Date 7/2/2018	Chk'd by	Date	App'd by	Date

#### Components and cladding pressures - Wall (Figure 30.4-1)

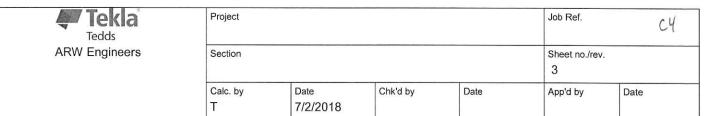
Component	Zone	Length (ft)	Width (ft)	Eff. area (ft²)	+GC <sub>p</sub>	-GC <sub>p</sub>	Pres (+ve) (psf)	Pres (-ve) (psf)
<10sf	4	-	-	10.0	1.00	-1.10	31.1	-33.7
50sf	4	-	-	50.0	0.88	-0.98	27.9	-30.5
200sf	4	-	-	200.0	0.77	-0.87	25.1	-27.7
>500sf	4	-	-	500.0	0.70	-0.80	23.2	-25.8
<10sf	5	_	-	10.0	1.00	-1.40	31.1	-41.7
50sf	5	-	-	50.0	0.88	-1.15	27.9	-35.1
200sf	5	-	-	200.0	0.77	-0.94	25.1	-29.5
>500sf	5	-	-	500.0	0.70	-0.80	23.2	-25.8





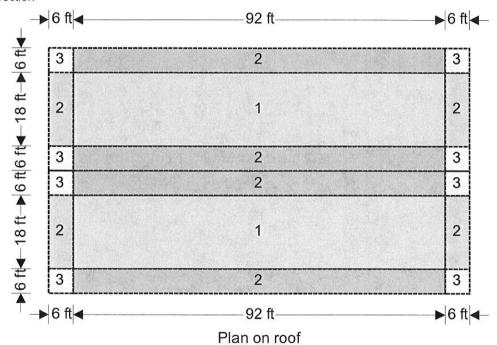
#### Components and cladding pressures - Roof (Figure 30.4-2B)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft²)	+GC <sub>p</sub>	-GC <sub>p</sub>	Pres (+ve) (psf)	Pres (-ve) (psf)
<10sf	1	-	-	10.0	0.50	-0.90	17.9	-28.5
25sf	1	-	-	25.0	0.42	-0.86	15.8 #	-27.4
50sf	1	-	-	50.0	0.36	-0.83	14.2 #	-26.6
>100sf	1	-	-	100.0	0.30	-0.80	12.7 #	-25.8
<10sf	2	-	-	10.0	0.50	-1.70	17.9	-49.6
25sf	2	-	-	25.0	0.42	-1.50	15.8 #	-44.3
50sf	2	-	-	50.0	0.36	-1.35	14.2 #	-40.4
>100sf	2	-	_	100.0	0.30	-1.20	12.7 #	-36.4



Component	Zone	Length (ft)	Width (ft)	Eff. area (ft²)	+GC <sub>p</sub>	-GC <sub>p</sub>	Pres (+ve) (psf)	Pres (-ve) (psf)
<10sf	3	-	-	10.0	0.50	-2.60	17.9	-73.3
25sf	3	-	-	25.0	0.42	-2.36	15.8 #	-67.0
50sf	3	-	-	50.0	0.36	-2.18	14.2 #	-62.2
>100sf	3	-	-	100.0	0.30	-2.00	12.7 #	-57.5

# The final net design wind pressure, including all permitted reductions, used in the design shall not be less than 16psf acting in either direction





Wood Stud Design Based on IBC 2015 & 2015 NDS

Version Date: May 25, 2017

JOB TITLE: NAC Rec Center

WALL LOCATION: Grid 1 & 5

9-Aug-18 9:05 AM

This program will design a wood stud/column based upon the compression and uniaxial bending interaction equation of the 2015 NDS Section 15.4 and the IBC 2015. APPLIED VERTICAL LOADS UNIFORM SNOW; UNIFORM LIVE: UNIFORM DEAD: WALL WEIGHTS
SELF WEIGHT:
FINISHES WEIGHT:

UNIFORM DEA		0 plf	S <sub>ds</sub> :	0.541 g		
DESIGN ROOF SNOW LOA			l <sub>e</sub> :	1		
BUILDING ELEVATION ALLOWABLE SOIL BEARING		0 ft. 0 psf	APPLIED LATE WIND (W):	RAL LOADS 30 psf		
THE OTT THE OOIE BETWEEN	100	Olbai	SEISMIC (E):	3.2 psf		
IF YOU ARE DESIGNING A	NYTHING OTHER T	HAM STUDS IF TE	DIMMEDS KING STUDS	OR COLLIMNS MARK TO	JIC CELL MITH AN "V"	
	and the same of th	HAN STODS, IE. IF			TIS CELL WITH AN X	
STUD CHARACTERISTICS NOMINAL STUD SIZ		7	REQUIRED FOOTING	WIDTH: 2.0 ft	(Footing sized for bearing of	only)
NOWINAL STOD SIZ	b(in.)		d(in.)			
STUD SIZE (actua	1.75	х 🗆		ACED AT 16 in	ches o.c.	
STUD LENGT	17	ft.	ECCE	NTRICITY 0 in	ches (at top of wall)	
MATERIAL PROPERTIES	WALL STUDS		BO	OTTOM PLATE	}	
Materia	: LVL 2.0E		Material:	LVL 2.0E	1	
		1 psi	f <sub>c</sub> =	49 psi		
		0 psi	F <sub>cperp</sub> ' =		.K.	
ANALYSIS E,	1,016,535	psi	E= 2	2,000,000 psi		
bending C		(2x4-1.5, 2x6-1.3	3, 2x8-1.2, 4x4-1.5) verify	with table 4A - Not for eng	ineered lumber	
comp. C		(2x4-1.15, 2x6-1	.1, 2x8-1.05, 4x4-1.15) ve	erify with table 4A - Not for	engineered lumber	
unsupported length,	Bending effective 204		37.09 in.	unsupported length, I <sub>u1</sub>	mpression effective length 204 k <sub>e1</sub> =	1.00
unsupported length,			6.86 in.	unsupported length, l <sub>u2</sub>	204	1.00
unbraced length,				unbraced length, I <sub>e1</sub>	204.0 d <sub>1</sub> =	5.5 in.
unbraced length,				unbraced length, le2	12.0 d <sub>2</sub> =	1.75 in.
bending	C <sub>r</sub> 1.15	(Gravity Load Cor	mbs.)	$l_{e1}/d_1 =$	37.09	Priore
Wind bending	1.35	(Only for Combs.	Including Wind)	$I_{e2}/d_2 =$	6.86	
Load Cambination #1	D. I		(6 1 40 0)			
Load Combination #1 C <sub>1</sub>	D+L = 1.1:	5	(formula 16-9) F <sub>bE</sub> =	1946	f <sub>b</sub> =	0.0 psi
C <sub>D</sub>			C <sub>L</sub> =	1.000	F <sub>c</sub> * =	2259 psi
F <sub>b</sub> *		2 psi	F <sub>b</sub> ' =	2992 psi	F <sub>cE</sub> =	607 psi
R <sub>B</sub>			S <sub>x</sub> =	8.8 in <sup>3</sup>	K <sub>1</sub> =	50. ps.
f <sub>c</sub>	= 2	2 psi	F <sub>c</sub> ' =	569 psi	C <sub>p</sub> =	0.252
Load at Base	= 28	5 plf	C.S.R =	0.001 <u>O.K.</u>		
Load Combination #2	D+S		(formula 16-10)			
C,		5	F <sub>bE</sub> =	1946	f <sub>b</sub> =	0.0 psi
CD	= 1	1	C <sub>L</sub> =	1.000	F <sub>c</sub> * =	2510 psi
F <sub>b</sub> *	= 332	5 psi	F <sub>b</sub> ' =	3325 psi	F <sub>cE</sub> =	607 psi
R <sub>B</sub>		0	S <sub>x</sub> =	8.8 in <sup>3</sup>	K <sub>r</sub> =	
f <sub>c</sub>		9 psi	F <sub>c</sub> ' =	573 psi	C <sub>p</sub> =	0.228
Load at Base	= 479	9 plf	C.S.R =	0.007 <u>O.K.</u>		
Load Combination #3	D + 0.75L + 0.75S		(formula 16-11)			
C,	= 1.15	5	F <sub>bE</sub> =	1946	f <sub>b</sub> =	0.0 psi
CD		1	C <sub>L</sub> =	1.000	F <sub>c</sub> * =	2510 psi
F <sub>b</sub> *		5 psi	F <sub>b</sub> ' =	3325 psi	F <sub>cE</sub> =	607 psi
R <sub>B</sub>			S <sub>x</sub> =	8.8 in <sup>3</sup>	K <sub>f</sub> =	
f <sub>c</sub> Load at Base		2 psi 1 plf	F <sub>c</sub> ' = C.S.R =	573 psi	C <sub>p</sub> =	0.228 in
Load at base	- 45	ı pii	C.S.R =	0.005 <u>O.K.</u>		
Load Combination #4	D + 0.6W		(formula 16-12)			
C,			F <sub>bE</sub> =	1946	f <sub>b</sub> =	1179.2 psi
C <sub>D</sub>			C <sub>L</sub> =	1	F <sub>c</sub> * =	4016 psi
F <sub>b</sub> *		5 psi	F <sub>b</sub> ' =	6245 psi	F <sub>cE</sub> =	607 psi
R <sub>B</sub>		u 2 psi	S <sub>x</sub> = F <sub>c</sub> ' =	8.8 in <sup>3</sup> 587 psi	K <sub>1</sub> =	0.440
Load at Base		D plf	C.S.R =	0.197 O.K.	C <sub>p</sub> =	0.146 in
		er 5/000		-43		
Load Combination #5 C <sub>r</sub>	D + 0.7E	=	(formula 16-12)	4046		7722
C <sub>D</sub>			F <sub>b∈</sub> = C <sub>L</sub> =	1946 1	f <sub>b</sub> = F <sub>c</sub> * =	148.9 psi 4016 psi
F <sub>b</sub> *		9 psi	F <sub>b</sub> ' =	5319 psi	F <sub>cE</sub> =	607 psi
R <sub>B</sub>			S, =	8.8 in <sup>3</sup>	K <sub>r</sub> =	oor pai
f <sub>c</sub>		2 psi	F <sub>c</sub> ' =	587 psi	C <sub>p</sub> =	0.146 in
Load at Base		0 plf	C.S.R =	0.030 <u>O.K.</u>	S.M.	A 50.10.70.000
Load Combination #6	D + 0.75L + 0.75S	± 0.46\M	(formula 16-13)			
C <sub>r</sub>			F <sub>bE</sub> =	1946	f <sub>b</sub> =	884.4 psi
C <sub>D</sub>			C <sub>L</sub> =	1	F <sub>c</sub> * =	4016 psi
F <sub>b</sub> *		5 psi	F <sub>b</sub> ' =	6245 psi	F <sub>cE</sub> =	607 psi
$R_{\theta}$		0	S <sub>x</sub> =	8.8 in <sup>3</sup>	K <sub>1</sub> =	C550 \$400
f <sub>c</sub>		2 psi	F <sub>0</sub> ' =	587 psi	C <sub>p</sub> =	0.146 in
Load at Base	= 43	1 plf	C.S.R =	0.157 <u>O.K.</u>		
Load Combination #7	D + 0.75L + 0.2125	SS + 0.75*0.7E	(formula 16-13)		snow load factor =	0.2 per exception No. 2
C <sub>r</sub>			F <sub>bE</sub> =	1946	f <sub>b</sub> =	111.6 psi
C <sub>D</sub>		5	C <sub>L</sub> =	1	F <sub>c</sub> * =	4016 psi
F₀*			F <sub>b</sub> ' =	5319 psi	F <sub>cE</sub> =	607 psi
R <sub>B</sub>			S <sub>x</sub> =	8,8 in <sup>3</sup>	K <sub>f</sub> =	
f <sub>c</sub>		3 psi	F <sub>c</sub> ' =	587 psi	C <sub>p</sub> =	0.146 in
Load at Base based on N	= 285 DS equation 15.4-1:	5 plf	C.S.R =	0.024 <u>O.K.</u>		



Wood Stud Design Based on IBG-2016 & 2015 NDS

Version Date: May 25, 2017

JOB TITLE: NAC Rec Center

WALL LOCATION: Grid B & H

9-Aug-18 9:05 AM

JOB #: ENGINEER:

APPLIED VERTIC	AL LOADS	970	olplf \	s	WALL WEIGHT:		psf			
	ORM LIVE:		plf		HES WEIGHT:		psi			
	RM DEAD:		plf		S <sub>ds</sub> :	0.541				
ESIGN ROOF SN		9			l <sub>e</sub> :	1	]			
BUILDING EI		5500 1500			APPLIED LAT WIND (W):		psf			
IF YOU ARE DES	IGNING AN	VTHING OTHER TI	JAN STUDS II	E TOIMME	SEISMIC (E):		psf	THIS CELL WITH AN		
		THE CONTENT	IAIT OTODO, II					A STATE OF THE PARTY OF THE PAR		
STUD CHARACTI NOMINAL S		1.75x5.5 b(in.)	]	RE d(in.)	QUIRED FOOTIN	NG WIDTH:	2.0	ft (Footing sized for be	earing only)	
	IZE (actual)	1.75	_ x	5.5		PACED AT		inches o.c.		
STUD MATERIAL PROP	ERTIES	21	_lft.		ECCI	ENTRICITY	0	inches (at top of wall)	)	
	Material:	WALL STUDS LVL 2.0E	1		Material:	BOTTOM P	LATE 2.0E	1		
	Fb		」 ∣psi		f <sub>c</sub> =		psi psi	THE PERSON NAMED IN COLUMN TWO IS NOT THE OWNER.		
	Fc	2510	) psi		F <sub>operp</sub> 's	750	psi	O.K.		
ANALYSIS	Emin	1,016,535	psi		E =	2,000,000	psi			
	bending CF	1						engineered lumber		
	comp. CF	1 Bending effective	(2x4-1.15, 2 e length	x6-1.1, 2x8-	1.05, 4x4-1.15)	verify with ta	able 4A - Not	for engineered lumber Compression effective I	length	
unsupporte		252	l <sub>u1</sub> /d <sub>1</sub> =	45.82	in.	unsuppo	rted length, l <sub>u</sub> .		e1 = 1.00	
	d length, l <sub>u2</sub>	12	$I_{u2}/d_2 =$	6.86	in.		rted length, l <sub>ut</sub>		e2 = 1.00	
	d length, l <sub>et</sub> d length, l <sub>e2</sub>	427.26 24.72					ced length, l <sub>e</sub> ced length, l <sub>e</sub> :		d <sub>1</sub> = 5.5	in.
	bending C,	1.15	(Gravity Load	Combs.)		unbra	cea lengin, l <sub>e:</sub> = l <sub>e1</sub> /d <sub>1</sub>		d <sub>2</sub> = 1.75	in.
Wind	bending C,	1.35	(Only for Con		ng Wind)		l <sub>e2</sub> /d <sub>2</sub> =			
Load Combination	#1	D+L			(formula 16-9)					
	C,=	1.15	i		(formula 16-9) F <sub>bE</sub> =	1590			f <sub>b</sub> = 0	.0 psi
	C <sub>D</sub> =	0.9	)		C <sub>L</sub> =	1.000			c* = 225	9 psi
	F <sub>b</sub> * =	2992	- 6		F <sub>b</sub> ' =	2992				8 psi
	R <sub>B</sub> =	27.7	psi		S <sub>x</sub> = F <sub>c</sub> ' =		in <sup>3</sup>		K, =	
Loa	d at Base=		psi plf		C.S.R =	382 0.012	O.K.		C <sub>p</sub> = 0.16	9
oad Combination	#2	D+S								
Load Combination	#2 C <sub>r</sub> =	1.15			(formula 16-10 F <sub>bE</sub> =	1590			f <sub>b</sub> = 0	.0 psi
	C <sub>D</sub> =	1			C <sub>L</sub> =	1.000			15	0 psi
	F <sub>b</sub> * =	3325			F <sub>b</sub> ' =	3325				8 psi
	$R_B = f_c =$	27.7	psi		S <sub>x</sub> = F <sub>c</sub> ' =		in <sup>3</sup> psi		K <sub>f</sub> =	
Loa	d at Base=	1435			C.S.R =	0.212	O.K.	`	C <sub>p</sub> = 0.15	13
Load Combination		D + 0.75L + 0.75S			(formula 16-11					
	C <sub>r</sub> =	1.15			F <sub>bE</sub> =	1590				.0 psi
	F <sub>b</sub> * =	3325			C <sub>L</sub> = F <sub>b</sub> ' =	1.000 3325				0 psi 8 psi
	R <sub>B</sub> =	27.7			S <sub>x</sub> =		in <sup>3</sup>		K <sub>f</sub> =	o pai
192	f <sub>c</sub> =		psi		F <sub>c</sub> ' =	384	psi		C <sub>p</sub> = 0.15	3 in
Loa	d at Base=	1193	plf		C.S.R =	0.139	O.K.			
oad Combination		D + 0.6W			(formula 16-12					
	C <sub>r</sub> =	1.35			F <sub>bE</sub> =	1590				4 psi
	F <sub>b</sub> * =	1.6 6245			C <sub>L</sub> = F <sub>b</sub> ' =	1 6245	psi			6 psi 8 psi
	R <sub>B</sub> =	27.7			S <sub>x</sub> =	8.8			K <sub>f</sub> =	- 201
	f <sub>c</sub> =		psi		F <sub>c</sub> ' =	390		C	C <sub>p</sub> = 0.09	7 in
	d at Base=	150	plf		C.S.R =	0.335	O.K.			
_oad Combination		D + 0.7E			(formula 16-12	•				
	C <sub>r</sub> =	1.15 1.6			F <sub>bE</sub> = C <sub>L</sub> =	1590 1				1 psi 6 psi
	F <sub>b</sub> * =	5319			F <sub>b</sub> ' =	5319				6 psi 8 psi
	R <sub>B</sub> =	27.7			S <sub>x</sub> =	8.8			K <sub>f</sub> =	
7	f <sub>c</sub> = d at Base=		psi n/f		F <sub>c</sub> ' =	390		c	C <sub>p</sub> = 0.09	7 in
		150			C.S.R =	0.060	U.K.			
oad Combination	#6 C <sub>r</sub> =	D + 0.75L + 0.75S			(formula 16-13)					2000
	C <sub>D</sub> =	1.35 1.6			F <sub>bE</sub> = C <sub>L</sub> =	1590 1			f <sub>b</sub> = 1349. c* = 401	6 psi 6 psi
	F <sub>b</sub> * =	6245			F <sub>b</sub> ' =	6245	psi			8 psi
	R <sub>B</sub> =	27.7			S <sub>x</sub> =	8.8	in³		K <sub>f</sub> =	9.5(//
Loo	f <sub>c</sub> =	143			F <sub>c</sub> ' =	390		C	C <sub>p</sub> = 0.09	7 in
	d at Base=	1193	Talah		C.S.R =	0.473	U.K.			
oad Combination	#7 C <sub>r</sub> =	D + 0.75L + 0.2125			(formula 16-13)			snow load factor	375741 TANDU	per exception No.
	C <sub>D</sub> =	1.15 1.6			F <sub>b∈</sub> = C <sub>L</sub> =	1590 1				4 psi 6 psi
	F <sub>b</sub> * =	5319			F <sub>b</sub> ' =	5319				8 psi
	R <sub>B</sub> =	27.7			S <sub>x</sub> =	8.8	in <sup>a</sup>		K <sub>f</sub> =	(E), "
v	f <sub>c</sub> = d at Base=		psi		F <sub>c</sub> ' =	390		C	C <sub>p</sub> = 0.09	7 in
	u at DaSe=	465	PIII		C.S.R =	0.072	O.K.			

Wood Stud Design Based on IBC 2015 & 2015 NDS

Version Date: May 25, 2017

JOB TITLE

WALL LOCATION: Grid 2 & 4

9-Aug-18 10:12 AM

APPLIED VERTICAL L UNIFORM S UNIFORM UNIFORM	NOW: LIVE: DEAD:	405	plf plf		WALL WEIGHT: ELF WEIGHT: HES WEIGHT: S <sub>ds</sub> :	10	psf psf g				
BUILDING ELEVA ALLOWABLE SOIL BEA	TION:	97 5500 1500			I <sub>e</sub> : APPLIED LATE WIND (W):	30	ADS psf				
IF YOU ARE DESIGNII	NG AN	THING OTHER TH	AN STUDS, IE	. TRIMMER	SEISMIC (E):	e de la companya de l	DMNS-MARK	THIS CELL WITH	I AN "X"		
STUD CHARACTERIS NOMINAL STUD		1.75x5.5 b(in.)	]	REC	QUIRED FOOTING	G WIDTH	: 2.5	ft (Footing sized t	for bearing o	nly)	
STUD SIZE ( STUD LEI MATERIAL PROPERT	NGTH	1.75 11.5	] x [ ]ft.	5.5		PACED AT		inches o.c. inches (at top of	wall)		
		WALL STUDS LVL 2.0E 2891	]		Material:		2.0E	]			
	Fo	2510	psi		f <sub>c</sub> = F <sub>cperp</sub> ' =	750	psi psi	0.K.			
ANALYSIS	E <sub>min</sub>	1,016,535				2,000,000	22				
	ing CF np. CF	1	(2x4-1.5, 2x6 (2x4-1.15, 2x	6-1.3, 2x8-1.	2, 4x4-1.5) verify 1.05, 4x4-1.15) verify	with table	4A - Not for e	engineered lumber for engineered lum	ber		
unsupported len	-	Bending effective 138	length I <sub>u1</sub> /d <sub>1</sub> =	25.09				Compression effect	ctive length	4.00	
unsupported len		12	I <sub>u2</sub> /d <sub>2</sub> =	6.86	in. in.		rted length, l <sub>u1</sub> rted length, l <sub>u2</sub>	138	k <sub>e1</sub> =	1.00	1
unbraced len		241.44	B 75005				iced length, l <sub>e1</sub>		d <sub>1</sub> =	5.5	in.
unbraced len	gth, l <sub>e2</sub> ding C,	1.15	1,0	C \		unbra	iced length, l <sub>e2</sub>		d <sub>2</sub> =	1.75	]in.
Wind bend		1.35	(Gravity Load (Only for Com		g Wind)		$l_{e1}/d_1 = l_{e2}/d_2 =$				
Load Combination #1	X 1	D+L			(formula 16-9)						
Esde Sombiliation	$C_r =$	1.15			F <sub>bE</sub> =	2813	ka		f <sub>b</sub> =	0.0	psi
	C <sub>D</sub> =	0.9	20		C <sub>L</sub> =	1.000			F <sub>c</sub> * =	2259	
	F <sub>b</sub> * = R <sub>B</sub> =	2992 20.8	psi		F <sub>b</sub> ' = S <sub>x</sub> =	2992	psi in³		$F_{cE} = K_f =$	1327	psi
	f <sub>c</sub> =	68	psi		F <sub>c</sub> ' =	1112			C <sub>p</sub> =	0.492	
Load at	Base=	578	plf		C.S.R =	0.004	O.K.				
Load Combination #2		D + S			(formula 16-10)						
	C <sub>r</sub> =	1.15			F <sub>bE</sub> =	2813			f <sub>b</sub> =		psi
	F <sub>b</sub> * =	1 3325	psi		C <sub>L</sub> =	1.000			F <sub>c</sub> * = F <sub>cE</sub> =	2510 1327	•
	R <sub>B</sub> =	20.8			S <sub>x</sub> =		in <sup>3</sup>		K <sub>f</sub> =	1027	pai
Load at	f <sub>c</sub> =	431			F <sub>c</sub> ' =	1138			C <sub>p</sub> =	0.454	
		3197	pit		C.S.R =	0.143	O.K.				
Load Combination #3	C,=	D + 0.75L + 0.75S 1.15			(formula 16-11) F <sub>bE</sub> =	2813			7	0.0	
	C <sub>D</sub> =	1.13			C <sub>L</sub> =	1.000			f <sub>b</sub> = F <sub>c</sub> • =	2510	psi psi
	F <sub>b</sub> * =	3325	psi		F <sub>b</sub> ' =	3325	psi		F <sub>cE</sub> =	1327	S. 2011
	R <sub>B</sub> =	20.8 340			S <sub>x</sub> =		in <sup>3</sup>		K <sub>f</sub> =		
Load at		2542			F <sub>c</sub> ' = C.S.R =	1138 0.089	O.K.		C <sub>p</sub> =	0.454	· in
Load Combination #4		D + 0.6W			(formula 16-12)						
	C <sub>r</sub> =	1.35			F <sub>bE</sub> =	2813			f <sub>b</sub> =	539.6	psi
	C <sub>D</sub> =	1.6			C <sub>L</sub> =	1			F <sub>c</sub> * =	4016	
	$F_b^* = R_B =$	6245 20.8	psi		F <sub>b</sub> ' = S <sub>x</sub> =	6245	psi in <sup>3</sup>		F <sub>cE</sub> = K <sub>f</sub> =	1327	psi
	f <sub>c</sub> =	68	psi		F <sub>c</sub> ' =	1221			C <sub>p</sub> =	0.304	in
Load at	Base=	405	plf		C.S.R =	0.094	O.K.				
Load Combination #5		D + 0.7E			(formula 16-12)						
	C <sub>r</sub> =	1.15			F <sub>bE</sub> =	2813			f <sub>b</sub> =	68.1	- TO
	C <sub>D</sub> =	1.6 5319	psi		C <sub>L</sub> = F <sub>b</sub> ' =	5319			F <sub>c</sub> * = F <sub>cE</sub> =	4016 1327	
	R <sub>B</sub> =	20.8	•		S <sub>x</sub> =		in³		K <sub>f</sub> =	1027	pa
Load at I	f <sub>c</sub> =	68			F <sub>c</sub> ' =	1221			C <sub>p</sub> =	0.304	in
		405			C.S.R =	0.017	O.K.				
Load Combination #6	C <sub>r</sub> =	D + 0.75L + 0.75S + 1.35	0.45W		(formula 16-13) F <sub>bE</sub> =	2813			f <sub>b</sub> =	404.7	nci
	C <sub>D</sub> =	1.6			C <sub>L</sub> =	1			F <sub>c</sub> * =	404.7	
	F <sub>b</sub> * =	6245	psi		F <sub>b</sub> ' =	6245			F <sub>cE</sub> =	1327	
	R <sub>B</sub> =	20.8 340	psi		S <sub>x</sub> = F <sub>c</sub> ' =	8.8 1221			$K_f = C_p =$	0.304	in
Load at I		2542			C.S.R =	0.165			Op -	0.304	
Load Combination #7		D + 0.75L + 0.2125S	+ 0.75*0.7E		(formula 16-13)			snow load	factor =	0.2	per exception No. 2
	C <sub>r</sub> =	1.15			F <sub>bE</sub> =	2813		JIIUW IUdu	f <sub>b</sub> =	51.1	
	$C_D =$ $F_b^* =$	1.6	nei		C <sub>L</sub> =	5240			F <sub>c</sub> * =	4016	
	R <sub>B</sub> =	5319 20.8	hai		F <sub>b</sub> ' = S <sub>x</sub> =	5319 8.8			$F_{cE} = K_f =$	1327	psi
	f <sub>c</sub> =	145			F <sub>c</sub> ' =	1221			C <sub>p</sub> =	0.304	in
Load at I		578 equation 15.4-1:	plf		C.S.R =	0.025	<u>O.K.</u>				
DL3CU .		- ,									

Wood Stud Design Based on IBC 2015 & 2015 NDS

Version Date: May 25, 2017

JOB TITLE: NAC Rec Center

WALL LOCATION: Grid J & K

9-Aug-18

9:05 AM

JOB #: ENGINEER:

APPLIED VERTICAL LOADS UNIFORM SNOW	388	lolf o	WALL WEIGHT		last		
UNIFORM SNOW UNIFORM LIVE			ELF WEIGHT: HES WEIGHT:		psf		
					psf		
UNIFORM DEAD	60		S <sub>ds</sub> :	0.541	g		
ESIGN ROOF SNOW LOAD		psf /	le:	1			
BUILDING ELEVATION	5500		APPLIED LATE				
LLOWABLE SOIL BEARING	1500	psf	WIND (W):		psf		
1			SEISMIC (E):	3.2	*		
F YOU ARE DESIGNING AN	IYTHING OTHER TH	AN STUDS, IE. TRIMMEI	RS, KING STUDS	OR COLL	JMNS MARK	THIS CELL WITH AN "X"	
STUD CHARACTERISTICS NOMINAL STUD SIZE	1.75x5.5	RE	QUIRED FOOTING	3 WIDTH:	2.0	ft (Footing sized for bearing	g only)
	b(in.)	d(in.)	7		- 10		
STUD SIZE (actual	1.75	X <u>5.5</u> ft.		ACED AT	16 0	inches o.c. inches (at top of wall)	
MATERIAL PROPERTIES	WALL STUDS			OTTOM PL		. /	
Material:	LVL 2.0E 2891	nei	Material:f <sub>c</sub> =	LVL 86	2.0E		
F.		•	F <sub>cperp</sub> ' =	750		O.K.	
Emi	1,016,535	psi		000,000		A SHIRITSI	
NALYSIS							
bending CF comp. CF		(2x4-1.5, 2x6-1.3, 2x8-1 (2x4-1.15, 2x6-1.1, 2x8-					
	Bending effective	length	1.00, 121 1.10, 10	any war to		Compression effective length	th
unsupported length, lu-		$I_{u1}/d_1 = 50.18$	in.	unsuppor	ted length, l <sub>u1</sub>	276 k <sub>e1</sub> =	1.00
unsupported length, lui		$l_{u2}/d_2 = 6.86$	in.	unsuppor	ted length, l <sub>u2</sub>	12 k <sub>e2</sub> =	1.00
unbraced length, le	466.38			unbrac	ed length, l <sub>e1</sub>	276.0 d <sub>1</sub> =	5.5 in.
unbraced length, le	24.72				ed length, le2	12.0 d <sub>2</sub> =	1.75 in.
bending C		(Gravity Load Combs.)		- 2013	l <sub>e1</sub> /d <sub>1</sub> =	50.18	
Wind bending C		(Only for Combs. Including	ng Wind)		$l_{e2}/d_2 =$	6.86	
			•				
oad Combination #1 C <sub>r</sub> =	D + L 1.15		(formula 16-9)	1450		£ _	0.0
			F <sub>bE</sub> =	1456		f <sub>b</sub> =	0.0 psi
C <sub>D</sub> =		v.	C <sub>L</sub> =	1.000		F <sub>c</sub> * =	2259 psi
F <sub>b</sub> * =		psi	F <sub>b</sub> ' =	2992		F <sub>cE</sub> =	332 psi
R <sub>B</sub> =			S <sub>x</sub> =	8.8	in³	K <sub>f</sub> =	
f <sub>c</sub> =			F <sub>c</sub> ' =	321		C <sub>p</sub> =	0.142
Load at Base=	405	plf	C.S.R =	0.010	O.K.		
oad Combination #2	D + S		(formula 16-10)				
C <sub>r</sub> =			F <sub>bE</sub> =	1456		f <sub>b</sub> =	0.0 psi
C <sub>D</sub> =	1		C <sub>L</sub> =	1.000		F <sub>c</sub> * =	2510 psi
F <sub>b</sub> * =		psi	F <sub>b</sub> ' =	3325	psi	F <sub>cE</sub> =	332 psi
R <sub>B</sub> =			S <sub>x</sub> =	8.8		K <sub>1</sub> =	oor bo
f <sub>c</sub> =		nei	F <sub>c</sub> ' =	322			0.128
Load at Base=			C.S.R =		O.K.	C <sub>p</sub> =	0.128
		Eq.		-101			
oad Combination #3	D + 0.75L + 0.75S		(formula 16-11)			200	2725 (2)
C <sub>r</sub> =			F <sub>bE</sub> =	1456		f <sub>b</sub> =	0.0 psi
C <sub>D</sub> =			C <sub>L</sub> =	1.000		F <sub>c</sub> * =	2510 psi
F <sub>b</sub> * =		psi	F <sub>b</sub> ' =	3325		F <sub>cE</sub> =	332 psi
R <sub>B</sub> =			S <sub>x</sub> =	8.8	in <sup>3</sup>	K <sub>t</sub> =	
f <sub>c</sub> =			F <sub>c</sub> ' =	322		C <sub>p</sub> =	0.128 in
Load at Base=	696	plf	C.S.R =	0.051	O.K.		
oad Combination #4	D + 0.6W		(formula 16-12)				
C,=			F <sub>b∈</sub> =	1456		f <sub>b</sub> =	2158.5 psi
C <sub>D</sub> =		(C.1014)	C <sub>L</sub> =	1		F <sub>c</sub> * =	4016 psi
F <sub>b</sub> * =		psi	$F_b' =$	6245		F <sub>cE</sub> =	332 psi
R <sub>B</sub> =			S <sub>x</sub> =	8.8		K <sub>t</sub> =	
f <sub>c</sub> =			F <sub>c</sub> ' =	326		C <sub>p</sub> =	0.081 in
Load at Base=	60	plf	C.S.R =	0.393	O.K.		
oad Combination #5	D + 0.7E		(formula 16-12)				
C <sub>r</sub> =	1.15		F <sub>bE</sub> =	1456		f <sub>b</sub> =	272.5 psi
C <sub>D</sub> =			C <sub>L</sub> =	1		F <sub>c</sub> * =	
F <sub>b</sub> * =			F <sub>b</sub> ' =	5319	psi	F <sub>cE</sub> =	332 psi
R <sub>B</sub> =			S, =	8.8		K <sub>1</sub> =	
f <sub>c</sub> =			F <sub>c</sub> ' =	326		C <sub>p</sub> =	
Load at Base=			C.S.R =	0.066			= 10000
oad Combination #6	D + 0.75L + 0.75S +	0.45W	(formula 16-13)				
C <sub>r</sub> =			F <sub>bE</sub> =	1456		f <sub>b</sub> =	1618.9 psi
C <sub>D</sub> =	1.6		C <sub>L</sub> =	1		F <sub>c</sub> * =	4016 psi
F <sub>b</sub> * =		psi	F <sub>b</sub> ' =	6245		F <sub>cE</sub> =	332 psi
R <sub>B</sub> =			S <sub>x</sub> =	8.8		K <sub>f</sub> =	1
f <sub>c</sub> =			F. =	326		C <sub>p</sub> =	0.081 in
Load at Base=			C.S.R =	0.381			
oad Combination #7	D + 0.75L + 0.21258	S + 0.75*0.7E	(formula 16-13)			snow load factor =	0.2 per exception No
C <sub>r</sub> =			F <sub>bE</sub> =	1456		f <sub>b</sub> =	204.4 psi
C <sub>D</sub> =			C <sub>L</sub> =	1		F <sub>c</sub> * =	4016 psi
F <sub>b</sub> * =		psi	F <sub>b</sub> ' =	5319		F <sub>cE</sub> =	332 psi
R <sub>B</sub> =		• 00000	S <sub>x</sub> =	8.8		K <sub>1</sub> =	oor ba
f <sub>c</sub> =		psi	F <sub>c</sub> ' =	326		C <sub>p</sub> =	0.081 in
			- C	JEU		Ο <sub>p</sub> =	0.001 III
Load at Base=	405	plf	C.S.R =	0.062	O.K.		

Wood Stud Design Based on IBC 2015 & 2015 NDS

Version Date: May 25, 2017

JOB TITLE: NAC Rec Center

WALL LOCATION: Grid A

9-Aug-18 9:05 AM

JOB #: ENGINEER:

APPLIED VERTICAL LOAD	is .		WALL WEIGH	rs				
UNIFORM SNOW	V: 388		SELF WEIGHT:		psf			
UNIFORM LIV			HES WEIGHT:		psf			
UNIFORM DEA		plf /	S <sub>ds</sub> :	0.541	g			
DESIGN ROOF SNOW LOA BUILDING ELEVATION			APPLIED LATE	1				
ALLOWABLE SOIL BEARING			WIND (W):		psf )			
1		7	SEISMIC (E):		psf			
IF YOU ARE DESIGNING A	NYTHING OTHER TH	IAN STUDS IE TRIMME	DE KING ETI IDE	OP COLL	IMAIC MADE	THIS CELL WITH AN	L "V"	
	NATIONAL CHIER III	IAN OTODO, IL. TRIVINIE	.NO, KING 31003	, OR COL	NAMA INIMA	INIS CELL WITH AN		
STUD CHARACTERISTICS		RE	QUIRED FOOTIN	G WIDTH:	2.0	ft (Footing sized for b	earing or	nly)
NOMINAL STUD SIZ	E 1.75x5.5 b(in.)	d(in.)						
STUD SIZE (actua	1.75	X 5.5	SF	ACED AT	16	inches o.c.		
STUD LENGT	20	ft.	ECCE	NTRICITY	0	inches (at top of wall)	)	
MATERIAL PROPERTIES	WALL STUDS		B	оттом РІ	ΔTF	/		
Materia		]	Material:		2.0E	and the same of th		
	b 2891	S	f <sub>c</sub> =		psi			
	2510		F <sub>cperp</sub> ' =	750	15002	O.K.		
ANALYSIS E,	1,016,535	psi	E= :	2,000,000	psi			
bending C	F 1	(2x4-1.5, 2x6-1.3, 2x8-	1 2 4x4-1 5) verify	with table	44 - Not for en	nineered lumber		
comp. C	F 1	(2x4-1.15, 2x6-1.1, 2x8			ble 4A - Not fo	r engineered lumber		
unsupported length,	Bending effective					ompression effective		
unsupported length,		$I_{u1}/d_1 = 43.64$ $I_{u2}/d_2 = 6.86$	in.		ted length, lut	240	k <sub>e1</sub> =	1.00
unbraced length,		J 1 <sub>12</sub> 712 - 0.86	in.		ted length, l <sub>u2</sub> ced length, l <sub>e1</sub>	12 240.0	k <sub>e2</sub> =	1.00
unbraced length,					ced length, l <sub>e1</sub>	12.0	d <sub>1</sub> =	5.5 in. 1.75 in.
bending		(Gravity Load Combs.)		und d	l <sub>e1</sub> /d <sub>1</sub> =	43.64	·2	Jill.
Wind bending		(Only for Combs. Includi	ng Wind)		$I_{e2}/d_2 =$	6.86		
(50)					NE E	(1550)		
Load Combination #1	D+L		(formula 16-9)	1922			2	
C, C <sub>D</sub>			F <sub>bE</sub> =	1666			f <sub>b</sub> =	0.0 psi
F <sub>b</sub> *			C <sub>L</sub> = F <sub>b</sub> ' =	1.000	nei		F <sub>c</sub> * =	2259 psi
R <sub>B</sub>			S <sub>x</sub> =	2992 8.8			F <sub>cE</sub> = K <sub>f</sub> =	439 psi
f <sub>c</sub>		psi	F <sub>c</sub> ' =	420			C <sub>p</sub> =	0.186
Load at Base			C.S.R =		O.K.		Op -	0.100
interessionaries		•			and the same of th			
Load Combination #2	D+S		(formula 16-10)				20.00	1990 På
C <sub>D</sub>			F <sub>bE</sub> =	1666			f <sub>b</sub> =	0.0 psi
F <sub>b</sub> *		nei	C <sub>L</sub> = F <sub>b</sub> ' =	1.000 3325			F <sub>c</sub> * =	2510 psi
R <sub>B</sub>		ры	S <sub>x</sub> =	8.8			F <sub>cE</sub> = K <sub>f</sub> =	439 psi
f <sub>c</sub>		psi	F <sub>c</sub> ' =	422			C <sub>p</sub> =	0.168
Load at Base			C.S.R =		O.K.		Op -	0.100
	75000	OF JOSE						
Load Combination #3 C,	D + 0.75L + 0.75S = 1.15		(formula 16-11)	4000				
C <sub>D</sub>			F <sub>bE</sub> = C <sub>L</sub> =	1.000			f <sub>b</sub> = F <sub>c</sub> * =	0.0 psi
F <sub>b</sub> *			F <sub>b</sub> ' =	3325	nei		F <sub>cE</sub> =	2510 psi 439 psi
R <sub>B</sub>		F	S <sub>x</sub> =	8.8			K <sub>f</sub> =	433 psi
f <sub>c</sub>		psi	F <sub>c</sub> ' =	422			C <sub>p</sub> =	0.168 in
Load at Base			C.S.R =		O.K.		P	
Lood Combination #4	D + 0.6W							
Load Combination #4 C <sub>r</sub>			(formula 16-12) $F_{bE} =$	1666			f <sub>b</sub> =	1632.1 psi
C <sub>D</sub>			C <sub>L</sub> =	1			ть — F <sub>c</sub> * =	4016 psi
F <sub>b</sub> *			F <sub>b</sub> ' =	6245	psi		rc = F <sub>cE</sub> =	439 psi
R <sub>B</sub>			S <sub>x</sub> =	8.8		- 1	'c∈ − K <sub>f</sub> =	700 psi
f <sub>c</sub>	= 29	psi	F. =	429			C <sub>p</sub> =	0.107 in
Load at Base		plf	C.S.R =		O.K.		15	
Load Combination #5	D + 0.7E		(formula 16-12)					
C,			(formula 16-12) F <sub>bE</sub> =	1666			f <sub>b</sub> =	206.0 psi
C <sub>D</sub>			C <sub>L</sub> =	1			ть — F <sub>c</sub> * =	4016 psi
F <sub>b</sub> *			F <sub>b</sub> ' =	5319			F <sub>cE</sub> =	439 psi
R <sub>B</sub>			S <sub>x</sub> =	8.8		**	K <sub>f</sub> =	400 psi
f <sub>c</sub>			F <sub>c</sub> ' =	429			C <sub>p</sub> =	0.107 in
Load at Base	= 60	plf	C.S.R =	0.046			8	
Load Combination #6	D + 0.75L + 0.75S +	0.45W	(formula 16 12)					
C <sub>r</sub>			(formula 16-13) F <sub>bE</sub> =	1666			f <sub>b</sub> =	1224.1 psi
C <sub>D</sub>			C <sub>L</sub> =	1		9	'ь - F <sub>c</sub> * =	4016 psi
F <sub>b</sub> *		psi	F <sub>b</sub> ' =	6245			F <sub>cE</sub> =	439 psi
R <sub>B</sub>			S <sub>x</sub> =	8.8			K <sub>f</sub> =	
f <sub>c</sub>			F <sub>c</sub> ' =	429			C <sub>p</sub> =	0.107 in
Load at Base	= 651	plf	C.S.R =	0.259	O.K.		10.0	
Load Combination #7	D + 0.75L + 0.21255	S + 0.75*0.7F	(formula 16-13)			gman, I 1 *	tor -	0.2
C <sub>r</sub>			(formula 16-13) F <sub>bE</sub> =	1666		snow load fac	tor = f <sub>b</sub> =	0.2 per exception No. 154.5 psi
C <sub>D</sub>			C <sub>L</sub> =	1			F <sub>c</sub> * =	4016 psi
F <sub>b</sub> *		psi	F <sub>b</sub> ' =	5319			F <sub>cE</sub> =	439 psi
			S <sub>x</sub> =	8.8			K <sub>f</sub> =	957
R <sub>B</sub>								
f <sub>c</sub>	= 41		F <sub>c</sub> ' =	429	psi		C <sub>p</sub> =	0.107 in
f <sub>c</sub> Load at Base	= 41		F <sub>c</sub> ' = C.S.R =	429 0.041			C <sub>p</sub> =	0.107 in



Wood Stud Design Based on IBC 2015 & 2015 NDS

Version Date: May 25, 2017

JOB TITLE: NAC Rec Center

WALL LOCATION: 5ft opening trimmer

9-Aug-18 9:34 AM

18121 TMD JOB #: ENGINEER:

APPLIED VERTICAL LOADS				WEIGHTS						
UNIFORM SNOW: UNIFORM LIVE:	2910	plf plf	SELF WEI			psf				
UNIFORM DEAD:	450		FINISHES WEI	S <sub>ds</sub> :	0.541	psf				
DESIGN ROOF SNOW LOAD:		psf		I <sub>e</sub> :	1	9				
BUILDING ELEVATION:	5500		APPLI	ED LATER		DS				
ALLOWABLE SOIL BEARING:	1500	psf	WIND			psf psf				
IF YOU ARE DESIGNING AN	YTHING OTHER TH	AN STUDS, IE. T				-0/1000	THIS CELL WITH A	λN "X"	X	
STUD CHARACTERISTICS			QUIRED SQUAR			Jane -				
NOMINAL STUD SIZE		]		E POOTIN	IG SIZE:	2.0	ft (Footing sized for	bearing of		
STUD SIZE (actual)	b(in.) 3.5	1 x [	d(in.) 5.5	TRIB	. WIDTH	38	inches	SERVICE STREET, SALES	25'x3	.5
STUD LENGTH	12.5	ft.	0.0	ECCEN			inches (at top of wa	all)		
MATERIAL PROPERTIES	WALL STUDS	2		во	TTOM PL	ATE	2			
Material:	LVL 2.0E	]	Materia	al:	LVL	2.0E				
F <sub>b</sub>			F	f <sub>c</sub> =	568 750		O.K.			
Emin					000,000	577	<u>O.K.</u>			
ANALYSIS bending CF		1 /2 1 5 2 1	2 200 4 2 404 4	<b>5</b> )	20. 4 . 1 .					
comp. CF	1		.3, 2x8-1.2, 4x4-1. 1.1, 2x8-1.05, 4x4				igineered lumber er engineered lumbe	er		
	Bending effective	length				Ç	compression effecti	ve length		
unsupported length, l <sub>u1</sub> unsupported length, l <sub>u2</sub>	150 12	I <sub>u1</sub> /d <sub>1</sub> =	27.27 in.			ted length, l <sub>u1</sub>	150	k <sub>e1</sub> =	1.00	
unbraced length, l <sub>e1</sub>	261.00	$I_{u2}/d_2 =$	3.43 in.			ted length, l <sub>u2</sub> ced length, l <sub>e1</sub>	12	k <sub>e2</sub> =	1.00	
unbraced length, le2	24.72					ced length, l <sub>e1</sub>	150.0 12.0	d <sub>1</sub> = d <sub>2</sub> =	5.5 in. 3.5 in.	
bending C <sub>r</sub>	1	(Gravity Load Co	ombs.)		unbiac	$I_{e1}/d_1 =$	27.27	u <sub>2</sub> -	3.5	
Wind bending C,	1	(Only for Combs	. Including Wind)			$l_{e2}/d_2 =$	3.43			
Load Combination #1	D+L		(formul	la 16.0\						
C <sub>f</sub> =				F <sub>bE</sub> =	10410			f <sub>b</sub> =	0.0 psi	
C <sub>D</sub> =	0.9			C <sub>L</sub> =	1.000			Fc* =	2259 psi	
F <sub>b</sub> * =	2602	psi		F <sub>b</sub> ' =	2602			F <sub>cE</sub> =	1123 psi	
R <sub>B</sub> =	10.8			S <sub>x</sub> =	17.6			K <sub>t</sub> =	1	
f <sub>c</sub> = Load at Base=	89 2019		ce	F <sub>c</sub> ' = (	975 0.008	o.K.		C <sub>p</sub> =	0.432	
					0.000	O.K.				
Load Combination #2 C <sub>r</sub> =	D+S			la 16-10)	10410					
C <sub>D</sub> =	1			F <sub>bE</sub> = C <sub>L</sub> =	1.000			f <sub>b</sub> = F <sub>c</sub> * =	0.0 psi 2510 psi	
F <sub>b</sub> * =	2891	psi		F <sub>b</sub> ' =	2891	psi		F <sub>cE</sub> =	1123 psi	
R <sub>B</sub> =	10.8			S <sub>x</sub> =	17.6			K <sub>f</sub> =	1	
f <sub>c</sub> =	568	psi		F <sub>c</sub> ' =	993	psi		C <sub>p</sub> =	0.396	
Load at Base=	11234	lb	c.s	.R = (	0.327	O.K.				
Load Combination #3	D + 0.75L + 0.75S		(formul	la 16-11)						
C <sub>r</sub> =	1			F <sub>bE</sub> =	10410			f <sub>b</sub> =	0.0 psi	
C <sub>D</sub> =	1			C <sub>L</sub> =	1.000			F <sub>c</sub> * =	2510 psi	
F <sub>b</sub> * = R <sub>B</sub> =	2891 10.8	psi		F <sub>b</sub> ' = S <sub>x</sub> =	2891 17.6			F <sub>cE</sub> =	1123 psi	
f <sub>c</sub> =	448	psi		F <sub>c</sub> ' =	993			K <sub>f</sub> = C <sub>p</sub> =	1 0.396 in	
Load at Base=	8930		C.S.			O.K.		ор	0.000 111	
Load Combination #4	D + 0.6W		(formul	a 16-12)						
C <sub>r</sub> =	1			F <sub>bE</sub> =	10410			f <sub>b</sub> =	0.0 psi	
C <sub>D</sub> =	1.6			CL =	1			F <sub>c</sub> * =	4016 psi	
F <sub>b</sub> * =	4626	psi		F <sub>b</sub> ' =	4626			F <sub>cE</sub> =	1123 psi	
$R_{\theta} = f_{c} =$	10.8	:		S <sub>x</sub> =	17.6			K <sub>f</sub> =	1	
Load at Base=	89 2019		c.s.	F <sub>c</sub> ' = .R = .0	1049 0.085	O.K.		C <sub>p</sub> =	0.261 in	
1 4 C										
Load Combination #5 C <sub>r</sub> =	D + 0.7E			a 16-12) F <sub>bE</sub> =	10410			f <sub>b</sub> =	95.6 psi	
C <sub>D</sub> =				C <sub>L</sub> =	1			F <sub>c</sub> * =	4016 psi	
F <sub>b</sub> * =	4626			F <sub>b</sub> ' =	4626	psi		F <sub>cE</sub> =	1123 psi	
R <sub>B</sub> =				S <sub>x</sub> =	17.6			K <sub>f</sub> =	1	
f <sub>c</sub> = Load at Base=	89 2019			F <sub>c</sub> ' = .R = 0	1049 0.030			C <sub>p</sub> =	0.261 in	
I I C										
Load Combination #6 C <sub>r</sub> =	D + 0.75L + 0.75S +			a 16-13) F <sub>bE</sub> =	10410			f <sub>b</sub> =	0.0 psi	
C <sub>D</sub> =	1.6			C <sub>L</sub> =	1			F <sub>c</sub> * =	4016 psi	
F <sub>b</sub> * =	4626	psi		F <sub>b</sub> ' =	4626			F <sub>cE</sub> =	1123 psi	
R <sub>B</sub> =	10.8			S <sub>x</sub> =	17.6			K <sub>f</sub> =	1	
f <sub>c</sub> = Load at Base=	448 8930		c.s.	F <sub>c</sub> ' = .R = .0	1049 0.427			C <sub>p</sub> =	0.261 in	
Load Combination #7	D + 0.75L + 0.2125S			a 16-13)						
C <sub>r</sub> =	1			a 16-13) F <sub>bE</sub> =	10410		snow load f	actor = f <sub>b</sub> =	0.2 per exc 71.7 psi	eption No. 2
C <sub>D</sub> =	1.6			C <sub>L</sub> =	1			F <sub>c</sub> * =	4016 psi	
F <sub>b</sub> * =	4626	psi		F <sub>b</sub> ' =	4626			F <sub>cE</sub> =	1123 psi	
R <sub>B</sub> =	10.8			S <sub>x</sub> =	17.6			K <sub>f</sub> =	1	
f <sub>c</sub> = Load at Base=	191 2019		c.s.	F <sub>c</sub> ' =	1049 0.052			C <sub>p</sub> =	0.261 in	
	S equation 15.4-1:		5.6.							
Dased of ND	o equation 15.4-1:									



## Wood Stud Design Based on IBC 2015 & 2015 NDS Version Date: May 25, 2017 JOB TITLE: NAC Rec Center WALL LOCATION: 5ft opening king

9-Aug-18 10:13 AM

18121 TMD JOB #: ENGINEER:

APPLIED VERTICAL LOADS UNIFORM SNOW UNIFORM LIVE UNIFORM DEAD	0	pif pif Fin	WALL WEIGHT SELF WEIGHT: NISHES WEIGHT: S <sub>ds</sub> :	10 5			
DESIGN ROOF SNOW LOAD BUILDING ELEVATION	: 97 : 5500	psf ft.	I <sub>e</sub> :	RAL LOAD	s		
ALLOWABLE SOIL BEARING	1500	]psf	WIND (W): SEISMIC (E):	3.2			
IF YOU ARE DESIGNING AN	IVTUING OTHER TH	AN STUDE IE TRIM	and the same of th	_	1	THO OF LAMETHAN IN	
	ITTHING OTHER TH	ad the state of th	-		MNS MARK	THIS CELL WITH AN "X"	X
STUD CHARACTERISTICS NOMINAL STUD SIZE	1.75x5.5 b(in.)	REQUIF d(in	RED SQUARE FOOT	ING SIZE:	2.0	ft (Footing sized for bearing	g only)
STUD SIZE (actual	1.75	X 5.5	TRI	B. WIDTH		inches	
STUD LENGTH MATERIAL PROPERTIES	20	ft.	ECCE	NTRICITY	0	inches (at top of wall)	
/	WALL STUDS			OTTOM PL	ATE	, J	
Material:		]	Material:	LVL 2			
\ F.			f <sub>c</sub> = F <sub>cperp</sub> ' =	49 750		O.K.	
Emir	1,016,535			2,000,000		A STATE OF STATE OF	
ANALYSIS					September 1		
bending CF comp. CF		(2x4-1.5, 2x6-1.3, 2x (2x4-1.15, 2x6-1.1.3)	8-1.2, 4x4-1.5) verify 2x8-1.05, 4x4-1.15) ve	with table 4	A - Not for er	igineered lumber	
	Bending effective	length	X0-1.00, 4X4-1.10) VE	silly with tal	ne 47 - Not ic	compression effective leng	th
unsupported length, lu		$l_{u1}/d_1 = 43.6$			ed length, l <sub>u1</sub>	240 k <sub>e1</sub> =	1.00
unsupported length, lu		$I_{u2}/d_2 = 6.86$	in.		ed length, l <sub>u2</sub>	12 k <sub>e2</sub> =	1.00
unbraced length, l <sub>e</sub> unbraced length, l <sub>e</sub> ;					ed length, l <sub>e1</sub>	240.0 d <sub>1</sub> =	
bending C		(Gravity Load Combs	<b>Y</b>	ulibraci	$l_{e1}/d_1 =$	12.0 d <sub>2</sub> = 43.64	1.75 in.
Wind bending C		(Only for Combs. Incli	THE PERSON NO.		$I_{e2}/d_2 =$	6.86	
		,			62 2		
Load Combination #1 C <sub>r</sub> =	D+L : 1		(formula 16-9)	4000		2	122 3
C <sub>D</sub> =			F <sub>b∈</sub> = C <sub>L</sub> =	1666 1.000		f <sub>b</sub> = F <sub>c</sub> * =	32.50 (\$2.00)
F <sub>b</sub> * =			F <sub>b</sub> ' =	2602	nsi	F <sub>cE</sub> =	
R <sub>B</sub> =			S <sub>x</sub> =	8.8		K <sub>f</sub> =	
f <sub>c</sub> =	49	psi	F <sub>c</sub> ' =	420	psi	C <sub>p</sub> =	
Load at Base=	950	lb	C.S.R =	0.014	O.K.		
Load Combination #2	D + S		(formula 16-10)				
C,=			F <sub>bE</sub> =	1666		f <sub>b</sub> =	0.0 psi
C <sub>D</sub> =			C <sub>L</sub> =	1.000		F <sub>c</sub> * =	2259 psi
F <sub>b</sub> * =		psi	F <sub>b</sub> ' =	2602		F <sub>cE</sub> =	
R <sub>B</sub> =		V_2;	S <sub>x</sub> =	8.8		K <sub>1</sub> =	
I <sub>c</sub> = Load at Base=		psi lb	F <sub>c</sub> ' = C.S.R =	420 j 0.014	0.K.	C <sub>p</sub> =	0.186
		ID.		0.014	O.K.		
Load Combination #3 C <sub>r</sub> =	D + 0.75L + 0.75S		(formula 16-11)	1000			122 8
C <sub>D</sub> =			F <sub>bE</sub> = C <sub>L</sub> =	1666 1.000		f <sub>b</sub> = F <sub>c</sub> * =	3
F <sub>b</sub> * =			F <sub>b</sub> ' =	2602	osi	F <sub>cE</sub> =	
R <sub>B</sub> =			S <sub>x</sub> =	8.8		K <sub>f</sub> =	
f <sub>c</sub> =		psi	F <sub>c</sub> ' =	420		C <sub>p</sub> =	
Load at Base=	950	lb	C.S.R =	0.014	O.K.		
Load Combination #4	D + 0.6W		(formula 16-12)				
C <sub>r</sub> =	1		F <sub>bE</sub> =	1666		f <sub>b</sub> =	3876.3 psi
C <sub>D</sub> =			C <sub>L</sub> =	1		F <sub>c</sub> * =	
F <sub>b</sub> * =		psi	F <sub>b</sub> ' =	4626		F <sub>cE</sub> =	
R <sub>8</sub> = f <sub>c</sub> =		psi	S <sub>x</sub> = F <sub>c</sub> ' =	8.8 i		K <sub>1</sub> =	
Load at Base=			C.S.R =	429 p	D.K.	C <sub>p</sub> =	0.107 in
				2-10-70-70E			
Load Combination #5 C <sub>r</sub> =	D + 0.7E		(formula 16-12) $F_{bE} =$	1666		f <sub>b</sub> =	489.3 psi
C <sub>D</sub> =			C <sub>L</sub> =	1000		η <sub>ь</sub> = F <sub>c</sub> * =	
F <sub>b</sub> * =			F <sub>b</sub> ' =	4626	osi	F <sub>cE</sub> =	
R <sub>B</sub> =			S <sub>x</sub> =	8.8 i	n³	K <sub>f</sub> =	
f <sub>c</sub> = Load at Base=		psi lb	F <sub>c</sub> ' = C.S.R =	429 j		C <sub>p</sub> =	0.107 in
				0.102	O.I.C.		
Load Combination #6 C <sub>r</sub> =	D + 0.75L + 0.75S +		(formula 16-13)	1000		20 <b>2</b> 000	2007.2
C <sub>D</sub> =			F <sub>bE</sub> = C <sub>L</sub> =	1666 1		f <sub>b</sub> =	
F <sub>b</sub> * =			F <sub>b</sub> ' =	4626	osi	F <sub>cE</sub> =	
R <sub>B</sub> =			S <sub>x</sub> =	8.8 i	n³	K <sub>f</sub> =	
f <sub>c</sub> =		psi	F <sub>c</sub> ' =	429		C <sub>p</sub> =	0.107 in
Load at Base=	950	lb	C.S.R =	0.721	O.K.		
Load Combination #7 C <sub>r</sub> =	D + 0.75L + 0.21258		(formula 16-13)	1000		snow load factor =	
C <sub>D</sub> =			F <sub>bE</sub> = C <sub>L</sub> =	1666 1		f <sub>b</sub> = F <sub>c</sub> * =	
F <sub>b</sub> * =			F <sub>b</sub> ' =	4626	osi	F <sub>c</sub> =	
R <sub>B</sub> =			S <sub>x</sub> =	8.8		K <sub>f</sub> =	
f <sub>c</sub> =		psi	F <sub>c</sub> ' =	429		C <sub>p</sub> =	
Load at Base=	950 S equation 15.4-1;	lb	C.S.R =	0.103	D.K.		
Dased off INC	_ = = = = = = = = = = = = = = = = = = =						



#### Wood Stud Design Based on IBC 2015 & 2015 NDS

9-Aug-18

10:05 AM Version Date: May 25, 2017

JOB TITLE: NAC Rec Center

WALL LOCATION: 12ft opening trimmer ENGINEERS JOB #: ENGINEER: This program will design a wood stud/column based upon the compression and uniaxial bending interaction equation of the 2015 NDS Section 15.4 and the IBC 2015. WALL WEIGHTS
SELF WEIGHT:
FINISHES WEIGHT: APPLIED VERTICAL LOADS UNIFORM SNOW: UNIFORM LIVE: 970 plf 0 plf 150 plf 97 psf 5500 ft. 1500 psf 10 psf 5 psf 0.541 g UNIFORM DEAD DESIGN ROOF SNOW LOAD: BUILDING ELEVATION; ALLOWABLE SOIL BEARING: APPLIED LATERAL LOADS
WIND (W): 0 psf
SEISMIC (E): 3.2 psf IF YOU ARE DESIGNING ANYTHING OTHER THAN STUDS, IE. TRIMMERS, KING STUDS, OR COLUMNS MARK THIS CELL WITH AN "X" STUD CHARACTERISTICS
NOMINAL STUD SIZE (2)1.75x5.5
b(in.) REQUIRED SQUARE FOOTING SIZE: 3.0 ft (Footing sized for bearing only) STUD SIZE (actual) STUD LENGTH MATERIAL PROPERTIES inches inches (at top of wall) 2.5' x 3.5' TRIB. WIDTH ECCENTRICITY

MATERIAL PROPERTIES	3							1			
Mater		WALL STUDS LVL 2.0E	1		Material:	OTTOM P	LATE 2.0E	1 /			
Mater	Fb	2891	psi psi		f <sub>c</sub> =		psi				
	Fq	2510			F <sub>cperp</sub> ' =		) psi	O.K.			
	E <sub>min</sub>	1,016,535			E = 2	2,000,000		San			
INALYSIS			-								
bending		1						engineered lumber			
comp.	CF	1	(2x4-1.15, 2x6	5-1.1, 2x8-	1.05, 4x4-1.15) v	erify with ta	able 4A - Not	for engineered lumber			
unsupported length	1.	Bending effective 132	l <sub>u1</sub> /d <sub>1</sub> =	24.00	in.	uncumpa	rted length, l <sub>u1</sub>	Compression effective		4.00	
unsupported length		12	I <sub>u2</sub> /d <sub>2</sub> =	3.43	in.		rted length, l <sub>u1</sub>		k <sub>e1</sub> =	1.00	
unbraced length		231.66	1627 42	3.43	HI.		iced length, l <sub>et</sub>		k <sub>e2</sub> =	1.00	
unbraced length		24.72					iced length, l <sub>et</sub> iced length, l <sub>et</sub>		d <sub>1</sub> =		n.
bendin		1	(Gravity Load C	Samba \		unbra	1074150141779	R 80000000	d <sub>2</sub> =	3.5 ii	n.
Wind bending		1	(Only for Comb		a Mind		l <sub>e1</sub> /d <sub>1</sub> = l <sub>e2</sub> /d <sub>2</sub> =				
VVIII d Deliain	a o'l		I(Only for Conin	is. iriciuuli	ig vvina)		i <sub>e2</sub> /u <sub>2</sub> =	. 3.43			
oad Combination #1		D+L			(formula 16-9)						
	C <sub>r</sub> =	1			F <sub>bE</sub> =	11728	Ĺ		f <sub>b</sub> =	0.0 g	osi
C	C <sub>D</sub> =	0.9			C <sub>L</sub> =	1.000	j		F.* =	2259 p	osi
F	b* =	2602	psi		F <sub>b</sub> ' =	2602	psi		F <sub>cE</sub> =	1451 p	
F	R <sub>B</sub> =	10.2			S, =	17.6			K <sub>f</sub> =	1	
	f <sub>c</sub> =	81	psi		F <sub>c</sub> ' =	1187			C <sub>p</sub> =	0.526	
Load at Ba	se=	2100	lb		C.S.R =	0.005				No. (10.00)	
oad Combination #2	C <sub>r</sub> =	D+S			(formula 16-10)					7/2/60	a a
		1			F <sub>bE</sub> =	11728			f <sub>b</sub> =	0.0 p	
	D =	1			C <sub>L</sub> =	1.000			F <sub>c</sub> * =	2510 p	
	ь <b>*</b> =	2891	7.		F <sub>b</sub> ' =	2891			F <sub>cE</sub> =	1451 p	osi
	R <sub>B</sub> =	10.2			S <sub>x</sub> =	17.6			K <sub>f</sub> =	1	
	f <sub>c</sub> =	416			F <sub>c</sub> ' =	1220			C <sub>p</sub> =	0.486	
Load at Ba	se=	8567	lb		C.S.R =	0.117	O.K.				
oad Combination #3		D + 0.75L + 0.75S			(formula 16-11)						
	C,=	1			F <sub>bE</sub> =	11728			f <sub>b</sub> =	0.0 p	nsi
	) <sub>D</sub> =	1			C <sub>L</sub> =	1.000			F <sub>c</sub> * =	2510 p	
	b* =	2891			F <sub>b</sub> ' =	2891			F <sub>cE</sub> =	1451 p	
	R <sub>B</sub> =	10.2			S <sub>x</sub> =	17.6		91	K <sub>r</sub> =	1451 p	)SI
	f <sub>c</sub> =	332			F <sub>c</sub> ' =	1220			C <sub>p</sub> =		
Load at Ba		6950			C.S.R =	0.074			C <sub>p</sub> =	0.486 ii	a
Loud at Da	J.	0000	10		0.5.K =	0.074	U.K.				
oad Combination #4		D + 0.6W			(formula 16-12)						
	C <sub>r</sub> =	1			F <sub>bE</sub> =	11728			$f_b =$	0.0 p	osi
	) <sub>D</sub> =	1.6			C <sub>L</sub> =	- 1			F <sub>c</sub> * =	4016 p	osi
F	<sub>b</sub> * =	4626	psi		F <sub>b</sub> ' =	4626	psi		F <sub>cE</sub> =	1451 p	osi
F	$R_B =$	10.2			S <sub>x</sub> =	17.6	, in <sup>3</sup>		$K_f =$	1	
	f <sub>c</sub> =	81	psi		F <sub>c</sub> ' =	1321	psi		C <sub>p</sub> =	0.329 is	n
Load at Ba	se=	2100	lb		C.S.R =	0.061	O.K.				
		D . 0.7E									
oad Combination #5	C,=	D + 0.7E			(formula 16-12)	11728					
	) <sub>D</sub> =	1.6			F <sub>bE</sub> = C <sub>L</sub> =	11728			f <sub>b</sub> =	155.8 p	
	p. =	4626							F <sub>c</sub> * =	4016 p	
	ь = R <sub>B</sub> =				F <sub>b</sub> ' =	4626		8	F <sub>cE</sub> =	1451 p	ISI
	f <sub>c</sub> =	10.2			S <sub>x</sub> =	17.6			K <sub>f</sub> =	1	
Load at Ba		81 2100	psi Ib		F <sub>c</sub> ' = C.S.R =	1321			C <sub>p</sub> =	0.329 in	a
Load at Ba	JC=	2100	10		6.5.K =	0.039	O'V'				
oad Combination #6		D + 0.75L + 0.75S +	0.45W		(formula 16-13)						
	C,=	1			F <sub>bE</sub> =	11728			f <sub>b</sub> =	0.0 p	osi
	) <sub>D</sub> =	1.6			C <sub>L</sub> =	1			F. =	4016 p	
F	b* =	4626	psi		F <sub>b</sub> ' =	4626			F <sub>cE</sub> =	1451 p	
F	R <sub>B</sub> =	10.2			S <sub>x</sub> =	17.6			K <sub>f</sub> =	1	
	f <sub>c</sub> =	332	psi		F <sub>c</sub> ' =	1321			C <sub>p</sub> =	0.329 ir	n
Load at Ba		6950			C.S.R =	0.252			5	020	75
10 11 7 #-	11						esecutes and the				
oad Combination #7		D + 0.75L + 0.21258			(formula 16-13)			snow load fac			per exception No.
(	C <sub>r</sub> =	1			F <sub>bE</sub> =	11728			f <sub>b</sub> =	116.9 p	
12 <u>4</u>	D =	1.6			C <sub>L</sub> =	1			F <sub>c</sub> * =	4016 p	
	. =	4626			F <sub>b</sub> ' =	4626		1	F <sub>cE</sub> =	1451 p	osi
F <sub>i</sub>						47.0	ID.		K <sub>f</sub> =		
F <sub>i</sub>	R <sub>B</sub> =	10.2			S <sub>x</sub> =	17.6				1	
F <sub>i</sub>	R <sub>B</sub> = f <sub>c</sub> =	10.2 152 2100	psi		S <sub>x</sub> = F <sub>c</sub> ' = C.S.R =	1321 0.041	psi		C <sub>p</sub> =	0.329 ir	n

Wood Stud Design Based on IBC 2015 & 2015 NDS

Version Date: May 25, 2017

JOB TITLE: NAC Rec Center

WALL LOCATION: 12ft opening king

9-Aug-18 10:05 AM

JOB #: ENGINEER:

							Jecuon 1	
APPLIED VERTICAL LOA		71_14	WALL WEIGH		1 .			
UNIFORM SNO UNIFORM LIV			BELF WEIGHT:		psf			
UNIFORM DEA		plf FINIS	HES WEIGHT:		psf			
			S <sub>ds</sub> :	0.541	19			
DESIGN ROOF SNOW LOA BUILDING ELEVATION		psf	le:	- January and				
ALLOWABLE SOIL BEARIN			WIND (W):		psf			
LECTIVIBLE GOIL BEARING	0	Праг	SEISMIC (E):	3.2				
					/			
IF YOU ARE DESIGNING	NYTHING OTHER TH	IAN STUDS, IE. TRIMME	RS, KING STUDS	OR COL	UMNS MARK	THIS CELL WITH A	N "X"	X
CTUD GUADAGTERIGI					***************************************			
NOMINAL STUD SI		7 REQUIREL	SQUARE FOOT	ING SIZE:	2.0	ft (Footing sized for	bearing o	nly)
140MINAL STOD SI	b(in.)	d(in.)						
STUD SIZE (actu		X 5.5	TR.	B. WIDTH	80	inches		
STUD LENGT		ft.		NTRICITY		inches (at top of wall	1)	
MATERIAL PROPERTIES		<del>-</del>			Newson Committee of the		<b>5</b> 0.	
724. 7	WALL STUDS	1		OTTOM P		, ]		
Materia		J .	Material:		2.0E	A STATE OF THE STA		
	F <sub>b</sub> 2891		f <sub>c</sub> =		psi	and the same of th		
	F <sub>c</sub> 2510		F <sub>cperp</sub> ' =		psi	O.K.		
	nin 1,016,535	psi	E = :	2,000,000	psi			
ANALYSIS	·	7 /2-4 4 5 2-5 4 2 2-5 4	0.4.4.5	The same of the same of the same of		20 20 2		
bending ( comp. (		(2x4-1.5, 2x6-1.3, 2x8-1 (2x4-1.15, 2x6-1.1, 2x8-	1.2, 4x4-1.5) Verity	with table	4A - Not for e	ngineered lumber		
comp. c	Bending effective	lenath	-1.05, 4X4-1.15) V	enry with ta		Compression effective		
unsupported length,	u1 240	l <sub>u1</sub> /d <sub>1</sub> = 43.64	in.	unsunnoi	ted length, l <sub>u1</sub>	240	k <sub>e1</sub> =	1.00
unsupported length,		l <sub>u2</sub> /d <sub>2</sub> = 2.29	in.		ted length, l <sub>u2</sub>	12	k <sub>e2</sub> =	1.00
unbraced length,		J "02"-2" 2.29	AL.		ced length, l <sub>e1</sub>	240.0		
unbraced length,					ced length, l <sub>e1</sub> ced length, l <sub>e2</sub>		d <sub>1</sub> =	5.5 in.
bending		Cravity Lord C		anbra		12.0	d <sub>2</sub> =	5.25 in.
Wind bending		(Gravity Load Combs.)	100 0		l <sub>e1</sub> /d <sub>1</sub> =	43.64		
vvina bending	C, 1	(Only for Combs. Including	ng vVind)		$I_{e2}/d_2 =$	2.29		
oad Combination #1	D+L		(formul= 10.0)					
	= 1		(formula 16-9)	14994			f -	0.0:
C			F <sub>bE</sub> =				f <sub>b</sub> =	0.0 psi
			C <sub>L</sub> =	1.000			F <sub>c</sub> * =	2259 psi
F <sub>b</sub>		-	F <sub>b</sub> ' =	2602			F <sub>cE</sub> =	439 psi
R			S <sub>x</sub> =	26.5			K <sub>f</sub> =	1
		psi	F <sub>c</sub> ' =	420	psi		C <sub>p</sub> =	0.186
Load at Bas	2000	lb	C.S.R =	0.007	O.K.			
_oad Combination #2	D+S		//					
C			(formula 16-10)					2021 032
			F <sub>bE</sub> =	14994			f <sub>b</sub> =	0.0 psi
C			C <sub>L</sub> =	1.000			F <sub>c</sub> * =	2259 psi
F <sub>b</sub> *			F <sub>b</sub> ' =	2602			F <sub>cE</sub> =	439 psi
R			S <sub>x</sub> =	26.5	in		K <sub>f</sub> =	1
		psi	F <sub>c</sub> ' =	420	psi		C <sub>p</sub> =	0.186
Load at Basi	= 2000	lb	C.S.R =	0.007	O.K.			
	D - 0.751 - 0.750		141					
Load Combination #3 C	D + 0.75L + 0.75S		(formula 16-11)				20.00	E021000000
			F <sub>bE</sub> =	14994			f <sub>b</sub> =	0.0 psi
Cc			C <sub>L</sub> =	1.000	1000		F <sub>c</sub> * =	2259 psi
F <sub>b</sub> *			F <sub>b</sub> ' =	2602			F <sub>cE</sub> =	439 psi
R <sub>B</sub>			S <sub>x</sub> =	26.5	in		K <sub>f</sub> =	1
f <sub>c</sub>		psi	F <sub>c</sub> ' =	420	psi		C <sub>p</sub> =	0.186 in
Load at Base	= 2000	lb	C.S.R =	0.007	O.K.		2001	
C	D . 0 0111		25 25 25 25					
oad Combination #4	D + 0.6W		(formula 16-12)					
C,			F <sub>bE</sub> =	14994			f <sub>b</sub> =	2720.2 psi
C			C <sub>L</sub> =	1			F <sub>c</sub> * =	4016 psi
F₀*		Owner,	F <sub>b</sub> ' =	4626			F <sub>cE</sub> =	439 psi
R <sub>B</sub>	= 9.0		S <sub>x</sub> =	26.5	in³		K <sub>f</sub> =	1
f		psi	F <sub>c</sub> ' =	429	psi		C <sub>p</sub> =	0.107 in
Load at Base	= 2000	lb	C.S.R =		O.K.		5	
	D . B 7F		227 N 725 Fast					
oad Combination #5	D + 0.7E		(formula 16-12)	8				
C,			F <sub>bE</sub> =	14994			f <sub>b</sub> =	343.4 psi
CD			C <sub>L</sub> =	1			F <sub>c</sub> * =	4016 psi
F <sub>b</sub> *		psi	F <sub>b</sub> ' =	4626			F <sub>cE</sub> =	439 psi
R <sub>B</sub>	= 9.0		S <sub>x</sub> =	26.5			K <sub>f</sub> =	1
f <sub>c</sub>	= 35	psi	F <sub>c</sub> ' =	429			Cp =	0.107 in
Load at Base			C.S.R =	0.087				
					1000000			
.oad Combination #6	D + 0.75L + 0.75S +		(formula 16-13)					
C,			F <sub>bE</sub> =	14994			f <sub>b</sub> =	2040.1 psi
CD			C <sub>L</sub> =	1			F <sub>c</sub> * =	4016 psi
F <sub>b</sub> *			F <sub>b</sub> ' =	4626	psi		F <sub>cE</sub> =	439 psi
R <sub>B</sub>	= 9.0		S <sub>x</sub> =	26.5			K <sub>f</sub> =	1
f <sub>c</sub>		psi	F <sub>c</sub> ' =	429			C <sub>p</sub> =	0.107 in
Load at Base			C.S.R =	0.485				
oad Combination #7	D + 0.75L + 0.2125		(formula 16-13)			snow load fa	ctor =	0.2 per exception
C,			F <sub>bE</sub> =	14994			f <sub>b</sub> =	257.5 psi
CD	= 1.6		C <sub>L</sub> =	1			F. =	4016 psi
	= 4626	psi	F <sub>b</sub> ' =	4626			F <sub>cE</sub> =	439 psi
F <sub>b</sub> •	4020							
			S <sub>x</sub> =	26.5	In-		N =	1
F <sub>b</sub> *	= 9.0			26.5 429			K <sub>f</sub> = C <sub>n</sub> =	1 0.107 in
F₀• R <sub>B</sub>	= 9.0 = 35	psi	S <sub>x</sub> = F <sub>c</sub> ' = C.S.R =	26.5 429 0.067	psi		C <sub>p</sub> =	0.107 in

ENGINEERS

Simply Supported Wood Beam Design (IBC 2015 and NDS 2015)
Version Date: October 28, 2016

JOB TITLE: NAC Rec
DESCRIPTION: HEADERS

JOB #: 18121 DESIGNER: TMD

o cario		23					52.000000					CHOOSE	MATERIAL P					
2x4 2x6 2x8 2x10 2x12 (2) 2x4 (2) 2x6 (2) 2x10 (2) 2x12 (3) 2x5 (3) 2x6 (3) 2x8 (3) 2x10 (3) 2x12 UNBRAC Lu	PROPERTIES   b   1.5	d 3.5 5.5 7.25 9.25 9.25 3.5 5.5 7.25 9.25 3.5 5.5 7.25 9.25 11.25 9.25 11.25	A 5.25 8.25 10.875 10.875 10.875 10.5 16.875 10.5 16.5 21.75 27.76 33.75 15.75 24.75 32.625 41.625 50.625	\$ 3.063 7.563 13.141 21.391 31.641 6.125 15.125 26.281 9.188 22.688 39.422 64.172 94.922	1 5.359 20.797 47.635 98.932 177.979 41.594 95.270 197.863 355.957 16.078 62.391 142.904 296.795 533.936		ADJUSTME  G <sub>F</sub> 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.1 1.5 1.3 1.2 1.3 1.2 1.3 1.3 1.2 1.3 1.3 1.2 1.3 1.3 1.3 1.3 1.3 1.3 1.3 1.3 1.3 1.3	NT FACTORS  Lyd 3.43 3.43 2.18 1.66 1.30 1.07 3.43 2.18 1.66 1.30 1.07 3.43 2.18 1.66 1.30 1.07 3.43 2.18 1.66 1.30 1.07	24.72 24.72 24.72 24.72 24.72 24.72 24.72 24.72 24.72 24.72 24.72 24.72 24.72 24.72	R <sub>B</sub> 6.20 7.77 8.92 10.08 11.12 3.10 3.89 4.46 5.04 5.56 2.07 2.59 3.36 3.71 8.71 8.72 8.72 8.72 8.72 8.72 8.72 8.72 8.72	F <sub>b</sub> * 1350 1170 1080 990 900 1350 1170 1080 990 900 1350 1170 1080 990 900 1350 1170 1080 990 900 1350 900 900 1350 900 900 900 900 900 900 900 900 900 9	E' 1500000 1500000 1500000 1500000 1500000 1500000 1500000 1500000 15000000 15000000 1500000 1500000 1500000 1500000 1500000 1500000 1500000 15000000 15000000 15000000 15000000 15000000 15000000 150000000 15000000 15000000 150000000 15000000 150000000 150000000 15000000 1500000000	E*mm 5800000	Fac 18100 11518 8738 6849 5631 72399 46072 34951 27394 22524 162899 103663 78641 61637 50680	C <sub>L</sub> 0.996 0.994 0.993 0.992 0.991 0.999 0.998 0.998 1.000 0.999 0.999 0.999 0.999	F <sub>5</sub> ' 1345 1163 1072 982 892 1349 1168 1078 988 898 1349 1169 989 899 11679 989		
Co C <sub>M</sub> C <sub>1</sub> C <sub>1</sub> C <sub>1</sub> C <sub>2</sub> C <sub>3</sub> C <sub>4</sub> C <sub>4</sub> C <sub>5</sub> C <sub>7</sub> C <sub>7</sub> Density	1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00	<u>10</u>			L	ON CRITERIA ./ 240 ./ 360	Total Load Live Load	MATERIAL PR DF-L S DF-L 1 BTR DF-L 1 DF-L 2 HEM-FIR S HEM-FIR 1 B HEM-FIR 1 HEM-FIR 2	F <sub>b</sub> 1500 1200 1000 900 1400 1100 975 850	F <sub>1</sub> 1000 800 675 575 925 725 625 525	F <sub>v</sub> 180 180 180 180 150 150 150	F <sub>CD41B</sub> 625 625 625 625 405 405 405 405	F <sub>coat</sub> 1700 1550 1500 1350 1500 1350 1350 1350	E 1900000 1800000 1700000 1600000 1500000 1500000 1300000	E <sub>min</sub> 690000 660000 620000 580000 580000 550000 550000 470000	F <sub>v</sub> ' 180 180 180 180 180 150 150 150		
						V proposition and a second								ABLE UNIFO	RM LOAD		Ŧ	a hhat
SPAN (FT)	4 6 6 7 8 9 10 11 12 13 14 15 16	2x4 170 109 75 55 42 33 26 21 18 15 13 11	2x6 365 233 161 118 90 71 57 47 39 33 28 24	2x8 585 373 258 189 144 114 91 75 63 53 45 39 34	2x10 872 557 386 283 216 170 137 113 94 80 68 59	2x12 2x12 1172 748 519 380 290 228 184 152 127 107 92 80 70	UNIFORM TO (2) 2x4 342 218 151 110 84 66 53 43 36 30 26 22 19	77AL LOAD (PL. (2) 2x6 (733 468 524 237 180 142 114 94 78 66 56 49 42	F) Based o. (2) 2x8 1176 751 520 381 290 228 184 151 126 107 91 79 69	n bending stre (2) 2x10 1755 1121 776 569 434 342 275 227 189 160 137 119	ss ω = 8*S* (2) 2x12 2360 1508 1045 766 584 460 371 305 255 216 186 161 140	(3) 2x4 (3) 2x4 513 327 226 165 126 98 79 65 54 45 39 33	2 (3) 2x6 1100 702 486 355 271 213 171 140 117 99 85 73 63	(3) 2x8 1765 1127 780 571 436 343 276 227 189 160 137 119	(3) 2x10 (2635 1683 1166 854 652 513 414 340 284 241 206 179 156	(3) 2x12 35x5 2264 1569 1150 877 691 557 459 384 325 279 241 211	6	P WORST ASE 1120 PLF GRID B
D	F-L 2	2x4	2x6	2x8	ALL 0	WABLE UNIFO		OAD (PLF) Bas										
SPAN (FT)	4 5 6 7 8 9 10 11 12 13 14 15	297 151 87 54 36 25 18 13 10 7 6 4 3	1153 590 340 214 143 100 72 54 41 32 25 20	2684 2644 1352 782 491 328 230 167 125 96 75 59 48	2x10 5493 2811 1625 1022 684 479 349 261 200 157 125 101 83	2812 9884 5059 2926 1841 1232 864 629 472 362 284 227 184 151	(2) 2x4 593 302 174 109 72 50 36 26 20 15 11 9	(2) 2x6 2307 1179 681 427 285 199 144 107 62 64 50 40	(2) 2x8 5288 2705 1563 983 657 460 334 250 191 149 118 95 78	(2) 2x10 10986 5622 3251 2045 1368 959 697 522 401 314 250 202 165	(2) 2x12 19768 10117 5852 3682 2464 1728 1258 943 725 568 453 367 301	(3) 2x4 890 454 261 163 108 75 54 39 29 22 17 13	(3) 2x6 3460 1769 1021 641 428 299 216 161 123 95 75 60 48	(3) 2x8 7932 4057 2345 1474 985 690 501 374 287 224 178 143	(3) 2x10 16479 8433 4876 3067 2052 1438 1046 783 601 471 375 303 248	(3) 2x12 29651 15176 8777 5523 3696 2593 1887 1415 1087 853 680 551 452		
D	F-L 2	2x4	2x6	2x8	AL. 2x10	LOWABLE UNI 2x12	FORM LIVE L (2) 2x4	OAD (PLF) Bas (2) 2x6	ed on defle (2) 2x8	ction ω = 384 (2) 2x10	*E*I*12/(LIVE (2) 2x12	LOAD*5*(L*1:	2)^3) (3) 2x6	(3) 2x8	(3) 2x10	(3) 2x12		
SPAN (FT)	4 5 6 7 8 9 10 11 12 13 14 15	197 100 58 36 24 16 12 8 6 5 3 3	768 392 226 142 94 66 47 35 27 21 16 13	1762 901 520 327 218 152 110 82 63 49 39 31 25	3661 1873 1082 681 455 319 231 173 133 104 82 66 54	6588 3371 1949 1226 820 575 418 313 240 188 150 121	395 201 115 72 47 32 23 17 12 9 7 5	1537 785 453 284 189 131 95 70 53 41 32 25	3524 1802 1040 653 436 305 221 165 126 98 77 62 50	7322 3746 2165 1361 910 637 463 346 265 207 165 133	13176 6742 3899 2452 1640 1150 836 626 481 376 300 242 198	592 301 173 108 71 49 35 25 18 14 10 8	2305 1177 679 425 283 197 142 105 80 62 48 38 30	5285 2702 1561 980 654 457 331 247 189 147 116 93 75	10983 5619 3247 2042 1365 956 694 519 398 311 247 199 162	19764 10113 5848 3678 2460 1725 1254 939 721 564 450 363 297		
D	F-L 2	2x4	2x6	2x8	2x10	LLOWABLE UN 2x12	IIFORM TOTA (2) 2x4	AL LOAD (PLF) ( (2) 2x6	Based on s (2) 2x8	hear stress (2) 2x10	= 2*b*d*F <sub>v</sub> . (2) 2x12	12/(3*((L*12)/2- (3) 2x4	d)) (3) 2x6	(3) 2x8	(3) 2x10	(3) 2x12		
SPAN (FT)	4 5 6 7 8 9 10 11 12 13 14 15	368 284 231 195 169 148 133 120 109 100 93 86 81	640 483 388 324 278 243 216 194 177 162 149 139	932 686 542 448 382 332 294 264 239 219 202 187 174	1351 960 744 607 512 443 391 349 315 287 264 244 227	1902 1292 978 786 657 565 495 440 396 360 330 305 283	735 568 463 390 337 297 265 240 218 201 185 172 161	1281 966 775 647 555 486 432 389 354 324 299 277 259	1865 1372 1084 896 764 665 589 528 479 438 403 374 348	2703 1919 1487 1214 1025 887 781 698 630 575 528 489 454	3804 2584 1956 1573 1315 1129 989 880 792 720 660 609 566	1103 852 694 585 506 445 398 359 327 301 278 259 242	1921 1449 1163 971 833 729 648 583 530 486 448 416 388	2797 2058 1627 1344 1145 997 883 792 718 657 605 560 522	4054 2879 2231 1821 1537 1330 1172 1047 946 862 792 733 681	5706 3876 2934 2359 1972 1694 1484 1320 1188 1081 990 914 849		

016

#### **General Notes**

- Table is based on:
  - Uniform loads (beam weight considered) and the more restrictive of simple or continuous span.
  - Deflection criteria of L/240 total load and L/360 live load.
- For live load deflection limits of L/240 or L/480, multiply live load values by 1.5 or 0.75, respectively. The resulting live load shall not exceed the total load shown

Also see How to Use This Table on page 18 and General Assumptions on page 5.

GRIDK / 448 PUF 3360 PLF

GRID H

1.9E Microllam® LVL: Floor-100% (PLF) continued

		3½" Width (2-ply)								51/4" Win	th (3_nlv)	/			
Span	Condition	14"	16"	18"	20"	51/2"	71/4"	91/4"	91/2" (	111/4"	117/8"	14"	16"	18"	20"
	Total Load	3,589	3,917	3,917	3,917	1,297	2,287	3,082	3,188	3,972	4,272	5,384	5,875	5,875	5,875
61)	Live Load L/360	*	*	*	*	870	1,879	*	*	3,312	4,212	*	*	3,073	3,073
	Min. End/Int. Bearing (in.)	- X	4.5/11.3	and the second	4.5/11.3	1.5/3.5	1.8/4.4	2.4/5.9	2.4/6.1	3.0/7.6	3.3/8.2	4.1/10.3	4.5/11.3	4.5/11.3	4.5/11.3
	Total Load	2,414	2,885	2,932	2,932	438	978	2,086	2,193	2,745	2,935	3,621	4.328	4.399	4.3711
8'	Live Load L/360	*	*	*	*	380	842	1.666	1.792	*	/ *	*	*	*	*
55	Min. End/Int. Bearing (in.)	3.7/9.3	4.4/11.1	4.5/11.3	4.5/11.3	1.5/3.5	1.5/3.5	2.1/5.3	2.2/5.6	2.8/7.0	/3.0/7.5	3.7/9.3	4.4/11.1	4.5/11.3	4.5/11.3
	Total Load	1,937	2,294	2,466	2,466	219	498	1,475	1,551	2,128	2,354	2,905	3,441	3,699	3,699
9'-6"	Live Load L/360	*	*	*	*	*	*	1,032	1,112	1,778	2,061	*	*	*	*
	Min. End/Int. Bearing (in.)	3.5/8.8	4.2/10.5	4.5/11.3	4.5/11.3	1.5/3.5	1.5/3.5	1.8/4.5	1.9/4.7	2.6/6.5	2.9/7.2	3.5/8.8	4.2/10.5	4.5/11.3	4.5/11.
144	Total Load	1.817	2,147	2,342	2,342	177	406	1,325	1,398	1,919	2,123	2,725	3,221	3,513	3,513
10'	Live Load L/360	*	*	*	*	*	*	893	963	1,54/4	1,792	*	*	*	*
	Min. End/Int. Bearing (in.)	3.5/8.7	4.1/10.3	4.5/11.3	4.5/11.3	1.5/3.5	1.5/3.5	1.7/4.3	1.8/4.5	2.5/6.1	2.7/6.8	3.5/8.7	4.1/10.3	4.5/11.2	4.5/11.
	Total Load	1,333	1,709	1,948	1.948	82	193	781	844	1,327	1,469	2,000	2,563	2,922	2,922
12'	Live Load L/360	1,138	1,635	*	*	*	*	530	572	927	1,080	1,707	2,453	*	*
_	Min. End/Int. Bearing (in.)	3.1/7.7	3.9/9.9	4.5/11.3	4.5/11.3	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	2.0/5.1	2.3/5.7	3.1/7.7	3.9/9.9	4.5/11.2	4.5/11.2
	Total Load	975	1,253	1,563	1,667		100	494	535	879	1,028	1,463	1.880	2,345	2,500
141	Live Load L/360	741	1,075	1,483	*		*	339	366	597	697	1,112	1,613	2,225	*
	Min. End/Int. Bearing (in.)	2.6/6.6	3.4/8.5	4.2/10.5	4.5/11.3		1.5/3.5	1.5/3.5	1.5/3.5	1.6/4.0	1.9/4.7	2.6/6.6	3.4/8.5	4.2/10.5	4.5/11.2
	Total Load	684	897	1,120	1,365			300	326	540	634	1,026	1,346	1,680	2,048
16'-6"	Live Load L/360	465	680	945	1,263			209	227	371	435	698	1,020	1,418	1,895
	Min. End/Int. Bearing (in.)	2.2/5.5	2.9/7.2	3.6/8.9	4.4/10.9			1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	2.2/5.5	2.9/7.2	3.6/8.9	4.4/10.9
	Total Load	488	710	887	1,082	2.		210	228	382	449	733	1,066	1,331	1,623
18'-6"	Live Load L/360	335	491	686	922			149	162	266	311	502	737	1,030	1,383
	Min. End/Int. Bearing (in.)	1.8/4.4	2.6/6.4	3.2/8.0	3.9/9.7			1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.8/4.4	2.6/6.4	3.2/8.0	3.9/9.7
	Total Load	387	573	756	922			164	178	300	354	580	860	1,134	1,384
20'	Live Load L/360	267	393	550	741			119	128	212	248	401	590	826	1,112
	Min. End/Int. Bearing (in.)	1.5/3.8	2.2/5.6	3.0/7.4	3.6/9.0			1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.8	2.2/5.6	3.0/7.4	3.6/9.0
- 15	Total Load	289	432	611	759			120	131	223	263	434	648	916	1,138
22'	Live Load L/360	202	298	419	566			89	97	160	187	304	448	629	850
1515	Min. End/Int. Bearing (in.)		1.9/4.7	2.6/6.6	3.3/8.2		***	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.9/4.7	2.6/6.6	3.3/8.2
	Total Load	221	332	471	634			89	98	168	199	332	498	707	951
24'	Live Load L/360	157	232	326	442			69	75	123	145	235	348	490	663
	Min. End/Int. Bearing (in.)	1.5/3.5	1.6/4.0	2.2/5.6	3.0/7.5			1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.6/4.0	2.2/5.6	3.0/7.5
15/AS	Total Load	172	259	370	506			67	74	129	153	258	389	555	760
26'	Live Load L/360	124	183	259	351			54	59	97	114	186	275	388	527
15.1	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.9/4.8	2.6/6.5			1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.9/4.8	2.6/6.5
	Total Load	135	205	294	405			51	56	100	120	203	308	442	607
28'	Live Load L/360	99	148	208	283			43	47	78	92	149	222	313	425
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.7/4.2	2.3/5.7			1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.7/4.2	2.3/5.7
	Total Load	108	164	237	327			1000		78	94	162	247	356	491
30'	Live Load L/360	81	120	170	232		4			63	75	122	181	256	348
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.5/3.7	2.0/5.0					1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.7	2.0/5.0

<sup>\*</sup>Indicates Total Load value controls.

Title Block Line 1 You can change this area using the "Settings" menu item and then using the "Printing & Title Block" selection.

Project Title: Engineer: Project ID: Project Descr:

Printed: 8 AUG 2018, 12:28PM

Licensee : ARW ENGINEERS

#### Title Block Line 6 Steel Beam

File = Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\Other\18121.ec6

Lic. #: KW-06002489

Description /

HSS Girt @ Grid 1/D-E (out-of-plane)

#### **CODE REFERENCES**

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10

Load Combination Set: ASCE 7-10

#### **Material Properties**

Analysis Method: Allowable Strength Design

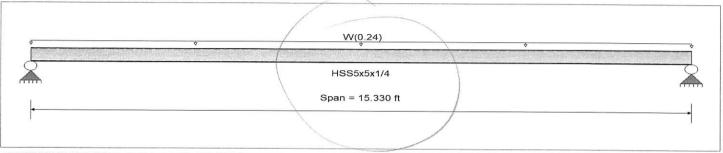
Beam is Fully Braced against lateral-torsional buckling Beam Bracing:

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 NOT internally calculated and added Uniform Load: W = 0.030 ksf, Tributary Width = 8.0 ft

DESI	GN	SI	IMI	MA	RY
	UIV	20		m	111

Max Downward Total Deflection

Max Upward Total Deflection

Design OK Maximum Bending Stress Ratio = 0.223:1 Maximum Shear Stress Ratio = 0.031:1 Section used for this span HSS5x5x1/4 Section used for this span HSS5x5x1/4 Ma: Applied 4.230 k-ft Va : Applied 1.104 k Mn / Omega: Allowable 18.987 k-ft Vn/Omega: Allowable 36.005 k Load Combination +0.60W Load Combination +0.60W Location of maximum on span 7.665ft Location of maximum on span 0.000 ft Span # where maximum occurs Span #1 Span # where maximum occurs Span #1 Maximum Deflection Max Downward Transient Deflection 0.000 in Ratio = 0 < 360 Max Upward Transient Deflection 0.000 in Ratio = 0 < 360

633 >=180

0 < 180

0.291 in Ratio =

0.000 in Ratio =

Load Combination		Max Stres	ss Ratios		Sı	ımmary of Mo	ment Valu	ies			Summary of Shear \		
Segment Length	Span #	М	V	Mmax + M	1max -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 15.29 ft	1		0.000				31.71	18.99	1.00	1.00	-0.00	60.13	36.00
Dsgn. L = 0.04 ft +0.60W	1		0.000				31.71	18.99	1.00	1.00	-0.00	60.13	36.00
Dsgn. L = 15.29 ft	1	0.223	0.031	4.23		4.23	31.71	18.99	1.00	1.00	1.10	60.13	36.00
Dsgn. L = 0.04 ft	1	0.003	0.031	0.05		0.05	31.71	18.99	1.00	1.00	1.10	60.13	36.00
+0.450W												00.10	00.00
Dsgn. L = 15.29 ft	1	0.167	0.023	3.17		3.17	31.71	18.99	1.00	1.00	0.83	60.13	36.00
Dsgn. L = 0.04 ft	1	0.002	0.023	0.04		0.04	31.71	18.99	1.00	1.00	0.83	60.13	36.00
<b>Overall Maximum</b>	Defle	ctions											
Load Combination		Span	Max. "-" Defl	Location in S	Span	Load Comb	oination			Max	. "+" Defl	Location	n in Span
+0.450W		1	0.2906	7.70	9						0.0000		0.000
Vertical Reaction	S			ç	Support n	otation : Far I	eft is #1			Values in	KIPS		

+0.450W	1	0.2906	7.709	0.0000	0.000
Vertical Reactions			Support notation : Far left is #1	Values in KIPS	
Load Combination	Support 1	Support 2			
Overall MAXimum	1.840	1.840			
Overall MINimum	0.828	0.828			
+0.60W	1.104	1.104			
+0.450W	0.828	0.828			
W Only	1.840	1.840			

Title Block Line 1 You can change this area using the "Settings" menu item and then using the "Printing & Title Block" selection.

Project Title: Engineer: Project ID: Project Descr:

0.400

0.0 plf

Title Block Line 6

Printed: 9 AUG 2018, 10:39AM

#### Cantilevered Retaining Wall

File = Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\Other\18121.ec6

	net a	Trees.						
Π	ic	# .	KV	V_O	60	024	RQ	

Screen Wall Description:

CMU

Licensee: ARW ENGINEERS

Criteria

Retained Height 2.50 ft Wall height above soil 6.00 ft = Slope Behind Wall 0.00:1 Height of Soil over Toe 30.00 in Water height over heel 0.0 ft

Vertical component of active Lateral soil pressure options:

NOT USED for Soil Pressure. NOT USED for Sliding Resistance. NOT USED for Overturning Resistance.

#### Surcharge Loads

Surcharge Over Heel = 0.0 Used To Resist Sliding & Overturning 0.0 psf Surcharge Over Toe 0.0 psf Used for Sliding & Overturning

#### Axial Load Applied to Stem

Axial Dead Load 0.0 lbs 0.0 lbs 0.0 in Axial Live Load Axial Load Eccentricity

Soil Data

Allow Soil Bearing 1,800.0 psf

Equivalent Fluid Pressure Method

Heel Active Pressure

Passive Pressure

Soil Density, Toe Friction Coeff btwn Ftg & Soil =

Soil height to ignore

for passive pressure

45.0 psf/ft Toe Active Pressure 30.0 psf/ft 389.0 psf/ft Soil Density, Heel 110.00 pcf 110.00 pcf

12.00 in

Lateral Load Applied to Stem Lateral Load

...Height to Top 0.00 ft ...Height to Bottom 0.00 ft =

Wind on Exposed Stem = 30.0 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	=	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

Title Block Line 1
You can change this area
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Title Block Line 6

Project Title: Engineer: Project ID: Project Descr:

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#### **Cantilevered Retaining Wall**

File = Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\Other\18121.ec6

Lic. #: KW-06002489

Description : Screen Wall

Licensee : ARW ENGINEERS

Design Summary			Ste
Wall Stability Ratios Overturning Sliding	= =	3.56 OK 11.39 OK	
Total Bearing Loadresultant ecc.	=	2,271 lbs 6.75 in	
Soil Pressure @ Toe Soil Pressure @ Heel Allowable Soil Pressure Less	= = = Than A	1,047 psf OK 89 psf OK 1,800 psf	
ACI Factored @ Toe ACI Factored @ Heel	=	1,256 psf 106 psf	
Footing Shear @ Toe Footing Shear @ Heel Allowable	= =	4.7 psi OK 7.5 psi OK 75.0 psi	
Sliding Calcs (Vertical Co Lateral Sliding Force less 100% Passive Force less 100% Friction Force	ompone = = • = •	ent NOT Used) 271.9 lbs 2,188.1 lbs 908.3 lbs	
Added Force Req'dfor 1.5 : 1 Stability	=	0.0 lbs OK 0.0 lbs OK	
Load Factors Dead Load Live Load Earth, H Wind, W Seismic, E		1.200 1.600 1.600 1.000 1.000	

tem Construction		Top Stem	2nd	<u> </u>
Design Height Above Ftg	 ft <i>=</i> ′	Stem OK 2.50	Stem OK 0.00	
Wall Material Above "Ht"	/=	Masonry	Concrete	
Thickness	/in =	8.00	8.00	
Rebar Size	=	# 5	# 5	
Rebar Spacing	in =	16.00	8.00	
Rebar Placed at	\ =	Center	Center	
Design Data ————— fb/FB + fa/Fa	1	0.475	0.146	
Total Force @ Section	lbs =	180.0	255.0	
MomentActual	ft-l =	540.0	1,052.5	
MomentAllowable	ft-l =	1,136.9	7,221.8	
ShearActual	psi =	4.0	7.6	
ShearAllowable	psi =	38.7	75.0	
Wall Weight	psf =	84.0	100.0	
Rebar Depth 'd'	in =	3.75	4.00	
Lap splice if above	in =	30.00	23.40	
Lap splice if below	in=	23.40	3.60	
Hook embed into footing	in =			
Masonry Data —	70000			
f'm	psi =	1,500	1,500	
Fyc	psi =		60,000	
Solid Grouting	=	Yes	Yes	
Modular Ratio 'n'	=	21.48	10.18	
Short Term Factor	=	1.000	1.000	
Equiv. Solid Thick.	in =	7.60	8.00	
Masonry Block Type	=	3		
Masonry Design Method	=	ASD		
Concrete Data				
fc	psi =		2,500.0	
Fy	psi =			

150.00	242 A.	0.40 (0.40)	
Fasting.	Dimensio	0 04	
Footing	Dimensio	ns & St	renarns

Toe Width	= /	1.67 ft
Heel Width	= /_	2.33
Total Footing Width	=	4.00
Footing Thickness	= \	12.00 in
Key Width	= \	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	2.00 ft
f'c = 2,500 psi	Fy =	60,000 psi 150.00 pcf
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top 2.00	@ Bt	tm.= 3.00 in

#### **Footing Design Results**

Factored Pressure Mu' : Upward Mu' : Downward Mu: Design	= = =	<u>Toe</u> 1,256 1,523 708 814	Heel 106 psf 0 ft-lb 708 ft-lb 708 ft-lb
Actual 1-Way Shear Allow 1-Way Shear Toe Reinforcing Heel Reinforcing Key Reinforcing	=	4.68 75.00 #7 @ 16.00 in #6 @ 16.00 in None Spec'd	7.46 psi 75.00 psi

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S \* Fr Heel: Not req'd, Mu < S \* Fr Key: No key defined

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,20

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#### **Cantilevered Retaining Wall**

Lic. #: KW-06002489

Licensee : ARW ENGINEERS

Description : Screen Wal

Summary of Overturni	ng & Resisting	Forces & Moments
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			VERTURNING					SISTING	
Item		Force lbs	Distance ft	Moment ft-lb			Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	275.6	1.17	321.6	Soil Over Heel	=	458.3	3.17	1,451.4
Surcharge over Heel	=				Sloped Soil Over Heel	=			1,10111
Toe Active Pressure	=	-183.8	1.17	-214.4	Surcharge Over Heel	=			
Surcharge Over Toe	=				Adjacent Footing Load	=			
Adjacent Footing Load	=				Axial Dead Load on Stem	=			
Added Lateral Load	=				* Axial Live Load on Stem	=			
Load @ Stem Above Soil	=	180.0	6.50	1,170.0	Soil Over Toe	=	458.3	0.83	381.9
					Surcharge Over Toe	=			001.0
					Stem Weight(s)	=	754.0	2.00	1,508.0
225 8 2					Earth @ Stem Transitions	=			1,000.0
Total	=	271.9	O.T.M. =	1,277.2	Footing Weight	=	600.0	2.00	1,200.0
Resisting/Overturning	Ratio		=	3.56	Key Weight	=		2.00	1,200.0
Vertical Loads used	for So	oil Pressure	= 2,270	.7 lbs	Vert. Component	=			
					Tota			bs <b>R.M.</b> =	4,541.3

<sup>\*</sup> Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Project Title: Engineer: Project ID: Project Descr:

12.00 in

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#### Cantilevered Retaining Wall

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Lic.	#:	KW-	-060	024	RQ		1	

Screen Wall Description:

CONC

Criteria		
Retained Height	= /	2.50 ft
Wall height above soil	7	6.00 ft
Slope Behind Wall	/=	0.00:1
Height of Soil over Toe	=	30.00 in
Water height over heel	\ <del>=</del>	0.0 ft
Vertical component of act Lateral soil pressure option NOT USED for Soil I	ons: 🔪	
	7220 55	

NOT USED for Sliding Resistance. NOT USED for Overturning Resistance.

#### Surcharge Loads

Surcharge Over Heel = 0.0 pst
Used To Resist Sliding & Overturning
Over Toe = 0.0 psf Surcharge Over Toe = Used for Sliding & Overturning

#### **Axial Load Applied to Stem**

0.0 lbs 0.0 lbs 0.0 in Axial Dead Load Axial Live Load
Axial Load Eccentricity

#### **Design Summary**

Wall Stability Ratios Overturning Sliding	=	3.71 OK 11.53 OK
Total Bearing Loadresultant ecc.	= =	2,367 lbs 6.48 in
Soil Pressure @ Toe Soil Pressure @ Heel Allowable Soil Pressure Less	= = = Than <i>A</i>	1,071 psf OK 113 psf OK 1,800 psf
ACI Factored @ Toe ACI Factored @ Heel	=	1,285 psf 135 psf
Footing Shear @ Toe Footing Shear @ Heel Allowable	= =	5.0 psi OK 7.5 psi OK 75.0 psi
Sliding Calcs (Vertical C Lateral Sliding Force less 100% Passive Force less 100% Friction Force	ompon = = • = •	ent NOT Used) 271.9 lbs
Added Force Req'd	=	0.0 lbs OK

Load Factors ———	
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

0.0 lbs OK

....for 1.5: 1 Stability

Soil Data	/	
Allow Soil Bearing	=	1,800.0 psf
Equivalent Fluid Pressure Me	ethod	i .
Heel Active Pressure	=	45.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	389.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Friction Coeff btwn Ftg & Soil	=	0.400

Soil height to ignore for passive pressure

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 30.0 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	=	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	1	op Stem	2nd	1
Design Height Above Ftg	 ft.∉	Stem OK 2.50	Stem OK 0.00	1
Wall Material Above "Ht"	/=	Concrete	Concrete	
Thickness	/in =	8.00	8.00	\
Rebar Size	/ =	# 5	# 5	
Rebar Spacing	in =	16.00	16.00	1
Rebar Placed at	\ =	Center	Center	/
Design Data ————	_		BANK BANKAN	
fb/FB + fa/Fa	/=	0.137	0.270	
Total Force @ Section	lbs =	180.0	255.0	
MomentActual	ft-l =	540.0	1,052.5	
MomentAllowable	ft-l =	3,945.8	3,898.0	
ShearActual	psi =	6.0	7.6	
ShearAllowable	psi =	82.2	75.0	
Wall Weight	psf =	100.0	100.0	
Rebar Depth 'd'	in =	4.00	4.00	
Lap splice if above	in =	21.36	23.40	
Lap splice if below	in =	21.36	6.00	
Hook embed into footing	in =	21.36	6.00	
Concrete Data —	7.47-0217	150 100005000	P-20-0-0-00000	
f'c	psi =	3,000.0	2,500.0	
Fy	psi =	60,000.0	60,000.0	

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Lic. #: KW-06002489

Description:

Screen Wall

Licensee: ARW ENGINEERS

Footing Dimensi	ions &	Streng	ths	
Toe Width		=	1	.67 ft
Heel Width		=	2	2.33
Total Footing Wid	lth	=	4	1.00
Footing Thickness	3	=	12	.00 in
Key Width		=	0	.00 in
Key Depth		=	0	.00 in
Key Distance fron	n Toe	=	2	.00 ft
f'c = 2,500	) psi	Fy =	60,0	000 psi 0.00 pcf
Footing Concrete	Density	=	150	0.00 pct
Min. As %		=	0.00	018
Cover @ Top	2.00	@	Btm.=	3.00 ir

Footing Design Res			
		<u>Toe</u>	Heel
Factored Pressure	=	1,285	135 psf
Mu': Upward	=		0 ft-lb
Mu' : Downward	=	708	708 ft-lb
Mu: Design	=	854	708 ft-lb
Actual 1-Way Shear	=	4.95	7.46 psi
Allow 1-Way Shear	=	75.00	75.00 psi
Toe Reinforcing	=	#7@16.00 in	
Heel Reinforcing		# 6 @ 16.00 in	
Key Reinforcing	=	None Spec'd	
Other Acceptable Sizes	8 8	Spacings	

Toe: Not req'd, Mu < S \* Fr Heel: Not req'd, Mu < S \* Fr Key: No key defined

Summary of Overturning & Resisting Forces & Moments

ltem	<u> </u>	0 Force lbs	VERTURNING. Distance ft	 Moment ft-lb			Force	SISTING Distance ft	Moment ft-lb
Heel Active Pressure	=	275.6	1.17	321.6	Soil Over Heel	=	458.3	3.17	1,451,4
Surcharge over Heel	=				Sloped Soil Over Heel	=			.,
Toe Active Pressure	=	-183.8	1.17	-214.4	Surcharge Over Heel	=			
Surcharge Over Toe	=				Adjacent Footing Load	=			
Adjacent Footing Load	=				Axial Dead Load on Stem	=			
Added Lateral Load	=				* Axial Live Load on Stem	=			
Load @ Stem Above Soil	=	180.0	6.50	1,170.0	Soil Over Toe	=	458.3	0.83	381.9
					Surcharge Over Toe	=			301.0
					Stem Weight(s)	=	850.0	2.00	1,700.0
<u> </u>	E-				Earth @ Stem Transitions	=			1,1 00.0
Total	=	271.9	O.T.M. =	1,277.2	Footing Weight	=	600.0	2.00	1,200.0
Resisting/Overturning Ratio = 3.71		Key Weight	=		2.00	.,			
Vertical Loads used	for So	oil Pressure	= 2,366	.7 lbs	Vert. Component	=			
					Tota			bs <b>R.M.</b> =	4,733.3

\* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.