A R W ENGINEERS
structural consultants

Structural Calculations

For

NAC Recreation Center – Park City

Project Number: 18121

August 9, 2018

PROFESSIONAL STRUCTURAL ENGINEER
No. 374996
Troy M. Dye
STATE OF UTAH
08/09/18

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

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CALCULATIONS INDEX

ROOF
FRAMING

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ROOF FRAMING

ROOF A: DEAD LOADS

			Comments
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	

ROOF B: DEAD LOADS

			Comments
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	

SNOW LOADS

Ground Snow Load :	139.0	psf
Snow Exposure Factor, C_e :	1	
Snow Load Importance Factor, I_s :	1	
Thermal Factor, C_t :	1	
Flat Roof Snow Load :	**97**	psf

PLYWOOD BENDING CAPACITY

ROOF 19/32" 40/20 SPAN RATING

$F_b S$ = 688 LB-IN/FT WIDTH 0° 4 PLY
180 " " 90° 4 PLY

BENDING

$W_b = \dfrac{120 F_b S}{L_b^2}$ $L_b = 24$ IN

$W_b = 143$ PSF

SHEAR

$W_s = \dfrac{20 F_s}{L_s}$ $F_s = 246$

$L_s = 24 - 1.5 = 22.5$"

$W_s = 219$ PSF

DEFLECTION

$W_{LL} = \dfrac{1743 \, EI \, \Delta_{LL}}{L_\Delta^4}$ $L_\Delta = 24 - 1.5 = 22.5$"

$EI = 247500$

$W_{LL} = 168$ PSF $\Delta_{LL} = 24/240 = 0.1$"

$W_{TL} = \dfrac{1743 \, EI \, \Delta_{TL}}{L_\Delta^4}$ $\Delta_{TL} = 24/180 = 0.13$"

$W_{TL} = 219$ PSF

CANTILEVER EAVE

$F_b S = \dfrac{W}{96 L^2} \left[L + a\right]^2 \left[L - a\right]^2$ $L = 24$"
$a = 12$"

$W_b = 204$ PSF

$W_b = \dfrac{24 F_b S}{L_b^2}$ $L_b = 12$"

$W_b = 114$ PSF USE 12" MAX OVERHANG LENGTH

A3

Table A
Wood Structural Panel Design Capacities Based on Span Ratings[a]

Span Rating		Strength							Planar Shear		Stiffness and Rigidity				
		Bending $F_b S$ (lb-in/ft of width)		Axial Tension $F_t A$ (lb/ft of width)		Axial Compression $F_c A$ (lb/ft of width)		Shear through the thickness [b,c] $F_v t_v$ (lb/in of shear-resisting panel length)	Planar Shear F_s (lb/Q) (lb/ft of width)		Bending EI (lb-in^2/ft of width)		Axial EA (lb/ft of width x 10^6)		Rigidity through the thickness $G_v t_v$ (lb/in of panel depth)
		Capacities relative to strength axis [d]													
		0°	90°	0°	90°	0°	90°	0° / 90°	0°	90°	0°	90°	0°	90°	0° / 90°
Sheathing Span®															
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®															
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

(a) 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_G, has already been incorporated into these design values—do not apply the C_G 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.
(b) Shear through the thickness design capacities are limited to sections two feet or less in width; wider sections may require further reductions.
(c) 5-ply applies to plywood with 5 or more layers; for 5-ply/3-layer plywood, use values for 4-ply plywood.
(d) 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.

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Title Block" selection.
Title Block Line 6

Project Title:
Engineer:
Project Descr:

Project ID: A4

Printed: 17 MAY 2018, 11:02AM

Wood Beam

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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
Load Combination : ASCE 7-10

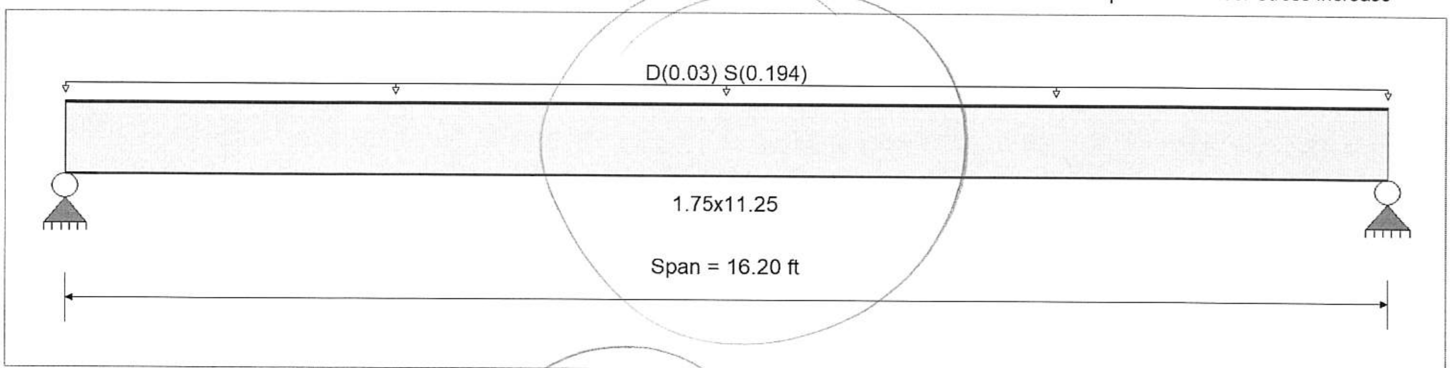
Wood Species : Trus Joist
Wood Grade : MicroLam LVL 2.0 E

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

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

E : Modulus of Elasticity
Ebend- xx 2000 ksi
Eminbend - xx 1016.535 ksi

Density 42 pcf
Repetitive Member 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 OK

Maximum Bending Stress Ratio	=	**0.695 : 1**	Maximum Shear Stress Ratio	=	**0.376 : 1**
Section used for this span		**1.75x11.25**	Section used for this span		**1.75x11.25**
fb : Actual	=	2,388.79 psi	fv : Actual	=	123.10 psi
FB : Allowable	=	3,438.50 psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.100 ft	Location of maximum on span	=	15.313 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.728 in	Ratio = 266 >=240
Max Upward Transient Deflection	0.000 in	Ratio = 0 <240
Max Downward Total Deflection	0.841 in	Ratio = 231 >=180
Max Upward Total Deflection	0.000 in	Ratio = 0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D 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 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

Wood Beam

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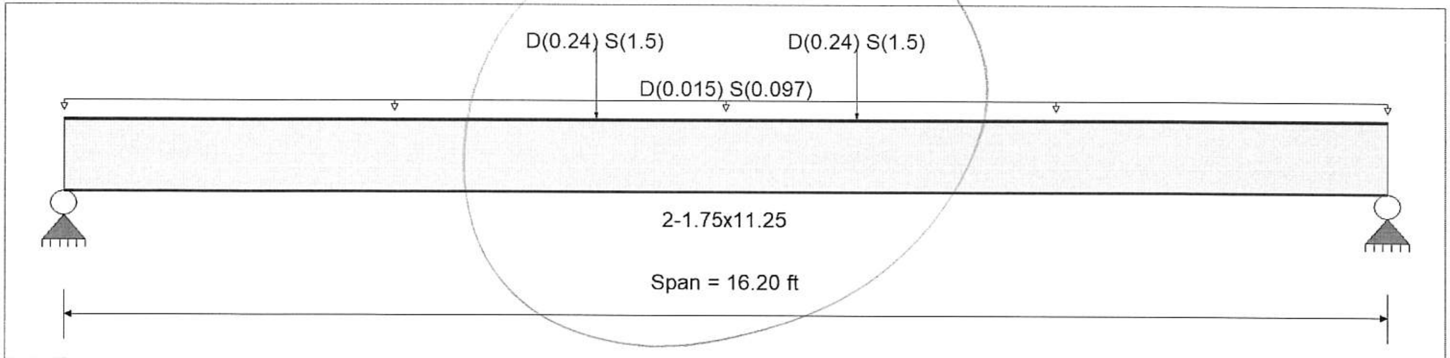
Description : Activity Room Joists at Roof Hoods

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,600.0 psi	*E : Modulus of Elasticity*	
Load Combination	ASCE 7-10	Fb -	2,600.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,510.0 psi	Eminbend - xx	1,016.54 ksi
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-torsional buckling			Repetitive Member 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 = 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

Design OK

Maximum Bending Stress Ratio	=	**0.708 : 1**	Maximum Shear Stress Ratio	=	**0.296 : 1**
Section used for this span		**2-1.75x11.25**	Section used for this span		**2-1.75x11.25**
fb : Actual	=	2,435.52 psi	fv : Actual	=	97.06 psi
FB : Allowable	=	3,438.50 psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.100 ft	Location of maximum on span	=	15.313 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.708 in	Ratio =	274 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	0.820 in	Ratio =	237 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D 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			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			0.00	0.00	0.00	0.00

Wood Beam

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Description : Activity Room Joists at Roof Hoods

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 16.20 ft	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
+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	0.042	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

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.8198	8.159		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.647	2.647
Overall MINimum	2.286	2.286
D Only	0.362	0.362
+D+S	2.647	2.647
+D+0.750S	2.076	2.076
+0.60D	0.217	0.217
S Only	2.286	2.286

Wood Beam

Description : Activity Room Joists

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.8408	8.159		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.814	1.814
Overall MINimum	1.571	1.571
D Only	0.243	0.243
+D+S	1.814	1.814
+D+0.750S	1.422	1.422
+0.60D	0.146	0.146
S Only	1.571	1.571

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Project Title:
Engineer:
Project ID:
Project Descr:

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Wood Beam

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Description : Beam supporting windows above overhead doors

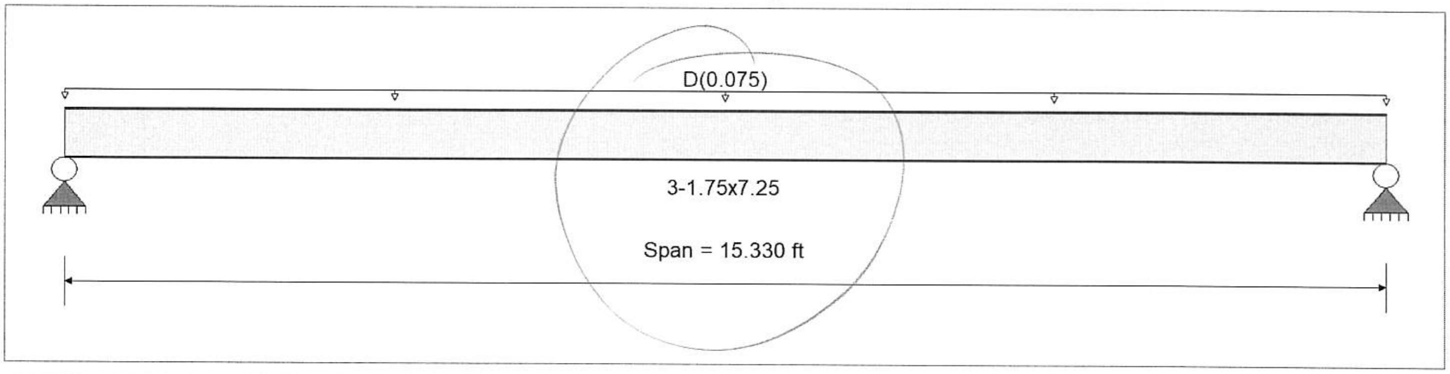
CODE REFERENCES

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 Elasticity*	
Load Combination	ASCE 7-10	Fb -	2,600.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,510.0 psi	Eminbend - xx	1,016.54 ksi
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-torsional buckling



Applied Loads

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

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

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.282** 1	Maximum Shear Stress Ratio	=	**0.094** : 1
Section used for this span		**3-1.75x7.25**	Section used for this span		**3-1.75x7.25**
fb : Actual	=	659.94 psi	fv : Actual	=	24.11 psi
FB : Allowable	=	2,340.00 psi	Fv : Allowable	=	256.50 psi
Load Combination		D Only	Load Combination		D Only
Location of maximum on span	=	7.665 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 <240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	0.323 in	Ratio =	569 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D 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			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

Overall Maximum Deflections

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

Wood Beam

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Description : Beam supporting windows above overhead doors

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

Steel Beam

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Description : Angle supporting windows above overhead doors

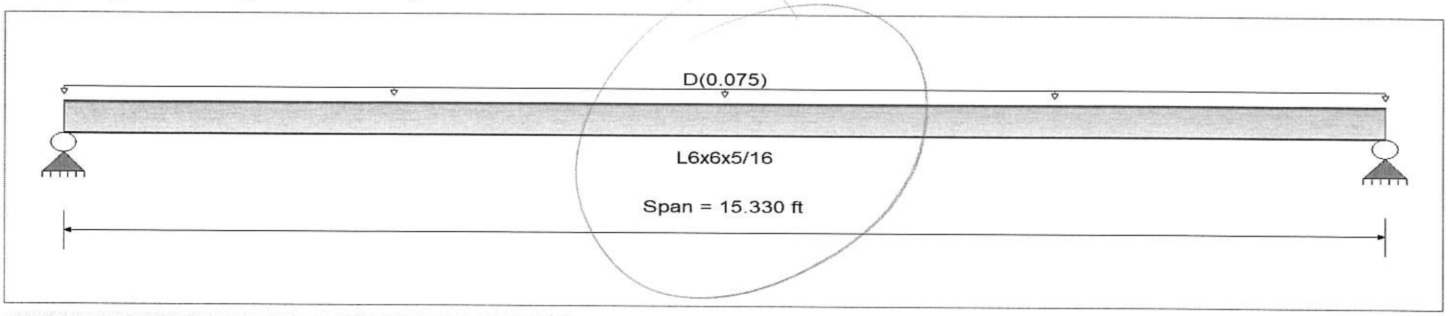
CODE REFERENCES

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 : 36.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

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

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

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.382 : 1	Maximum Shear Stress Ratio =	0.028 : 1
Section used for this span	L6x6x5/16	Section used for this span	L6x6x5/16
Ma : Applied	2.567 k-ft	Va : Applied	0.6699 k
Mn / Omega : Allowable	6.722 k-ft	Vn/Omega : Allowable	24.252 k
Load Combination	D Only	Load Combination	D Only
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
Max Downward Total Deflection	0.289 in	Ratio =	636 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 15.29 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													
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

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D Only	1	0.2894	7.709		0.0000	0.000

Vertical Reactions

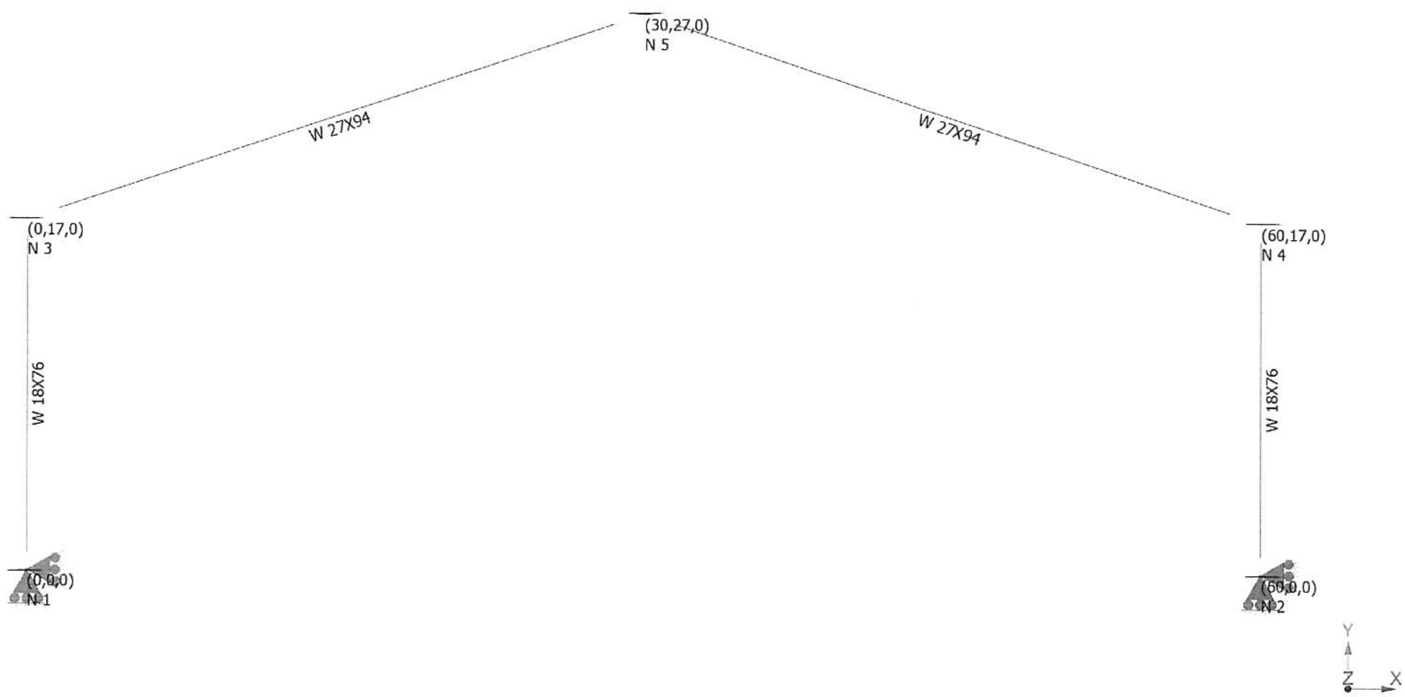
Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.670	0.670
Overall MINimum	0.402	0.402
D Only	0.670	0.670
+0.60D	0.402	0.402

Bentley

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\

A11



Bentley

A12

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

Bentley

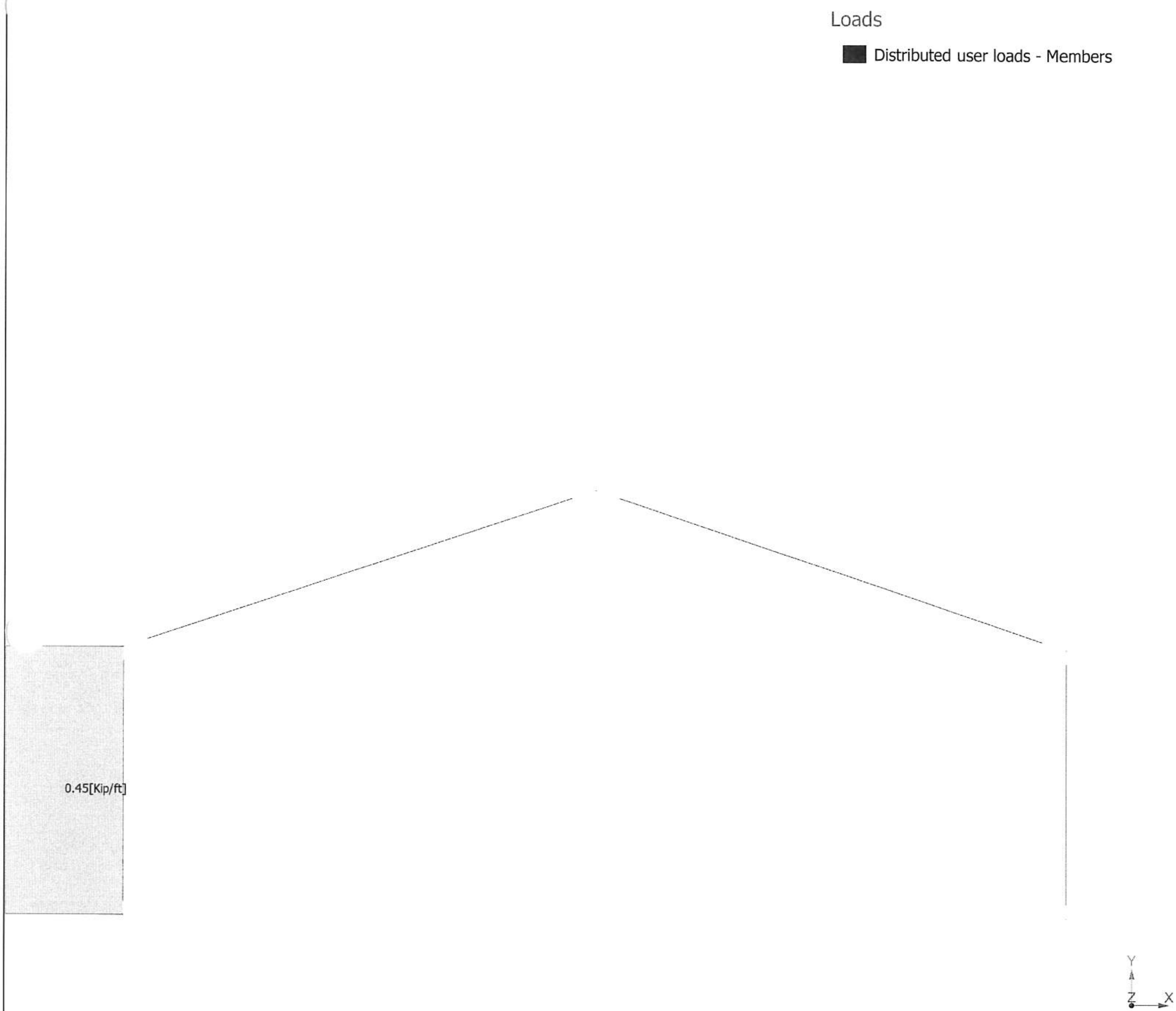
A13

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

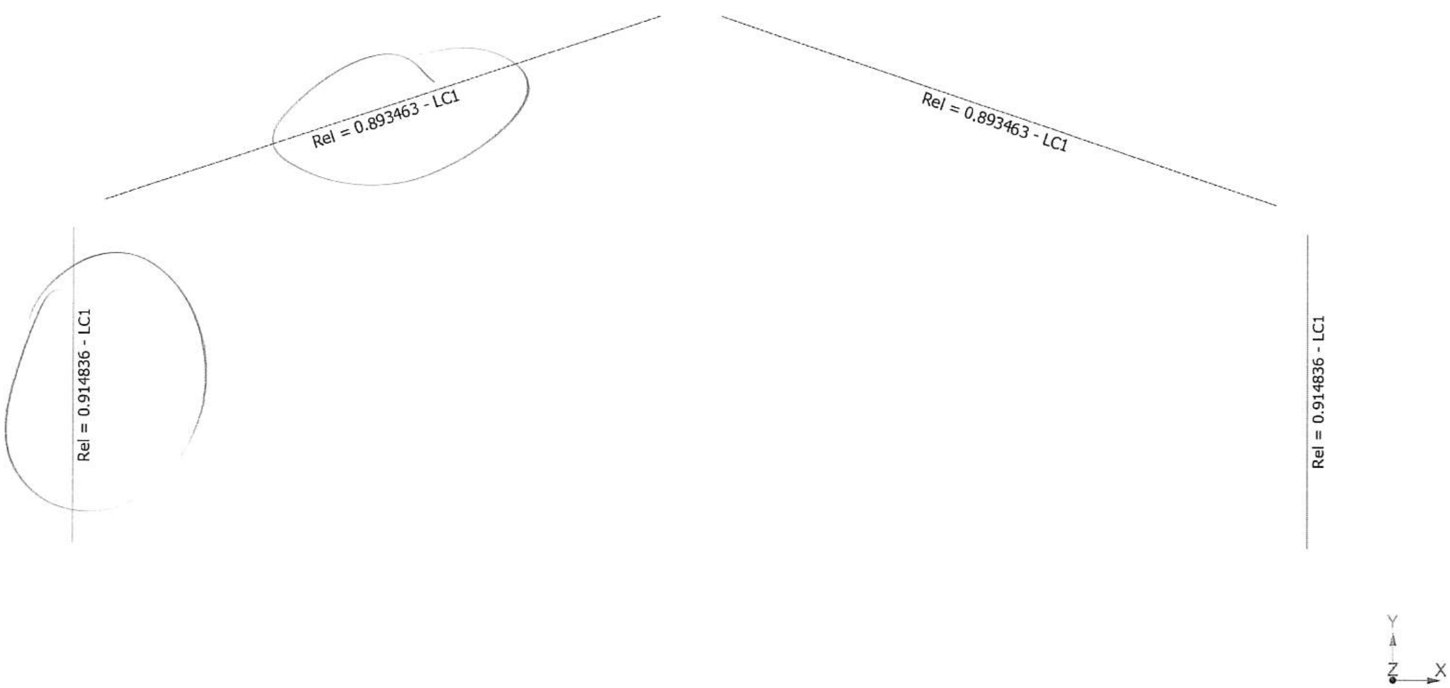
Bentley

A14

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



A15


Rel = 0.893463 - LC1
Rel = 0.893463 - LC1
Rel = 0.914836 - LC1
Rel = 0.914836 - LC1
Y
Z
X

Bentley

Current Date: 8/3/2018 1:47 PM
Units system: English
File name: Y:\Projects 2018\18121 - NAC Recreation Center\Engineering\Calculations\RAM\18121 - Activity Room Frame.etz\

Steel Code Check

Report: Concise

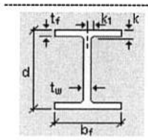
Members: Hot-rolled
Design code: AISC 360-2010 ASD

Member	:	**1 (column)**
Design status	:	**OK**

Section information

Section name: W 18X76 **(US)**

Dimensions



bf	=	11.000	[in]	Width
d	=	18.200	[in]	Depth
k	=	1.080	[in]	Distance k
k1	=	1.063	[in]	Distance k1
tf	=	0.680	[in]	Flange thickness
tw	=	0.425	[in]	Web thickness

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')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	7.723	2.611
Radius of gyration (principal axes) (r')	[in]	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)	[in]	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)	[in]	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

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	17.00

Distance between member lateral bracing points

Length (Lb) [ft] Top	Bottom
17.00	17.00

Laterally unbraced length

	Length [ft]			Effective length factor	
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
17.00	17.00	17.00	1.0	1.0	1.0

Additional assumptions	
Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✓

Axial tension

Ratio	:	0.00			
Capacity	:	667.66 [Kip]	Reference	:	Eq. Sec. D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(P_n/Ω)	[Kip]	667.66	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✓

Compression in the major axis 33

A16

Ratio	: 0.09		
Capacity	: 630.24 [Kip]	Reference	: Sec. E1
Demand	: 55.85 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(P_{n33}/Ω)	[Kip]	630.24	Sec. E1

Compression in the minor axis 22

Ratio	: 0.13		
Capacity	: 427.25 [Kip]	Reference	: Sec. E1
Demand	: 55.85 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(P_{n22}/Ω)	[Kip]	427.25	Sec. E1
Factored torsional or flexural-torsional buckling strength(P_{n11}/Ω)	[Kip]	506.21	Sec. E4

FLEXURAL DESIGN ✓

Bending about major axis, M33

Ratio	: 0.87		
Capacity	: 406.69 [Kip*ft]	Reference	: Sec. F1
Demand	: -354.03 [Kip*ft]	Ctrl Eq.	: LC1 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(M_n/Ω)	[Kip*ft]	406.69	Sec. F1
Factored lateral-torsional buckling strength(M_n/Ω)	[Kip*ft]	406.69	Sec. F1

Bending about minor axis, M22

Ratio	: 0.00		
Capacity	: 105.29 [Kip*ft]	Reference	: Sec. F1
Demand	: 0.00 [Kip*ft]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(M_n/Ω)	[Kip*ft]	105.29	Sec. F1

DESIGN FOR SHEAR ✓

Shear in major axis 33

Ratio	: 0.00		
Capacity	: 268.74 [Kip]	Reference	: Sec. G1
Demand	: 0.00 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(V_n/Ω)	[Kip]	268.74	Sec. G1

Shear in minor axis 22

Ratio	: 0.13		
Capacity	: 154.80 [Kip]	Reference	: Sec. G2.1(a)
Demand	: -20.83 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(V_n/Ω)	[Kip]	154.80	Sec. G2.1(a)

COMBINED ACTIONS DESIGN ✓

Combined flexure and axial compression

Ratio	: 0.91		
Ctrl Eq.	: LC1 at 100.00%	Reference	: Eq. H1-1b

Intermediate results	Unit	Value	Reference
Interaction for doubly symmetric members for in-plane bending	--	0.91	Eq. H1-1b
Interaction for doubly symmetric members for out-of-plane bending	--	0.46	Eq. H1-2

Combined flexure and axial tension

Ratio	: 0.87		
Ctrl Eq.	: LC1 at 100.00%	Reference	: Eq. H1-1b

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio	: N/A		
Ctrl Eq.	: --	Reference	:

Combined flexure and axial tension about local axis

Ratio	: N/A		
Ctrl Eq.	: --	Reference	:

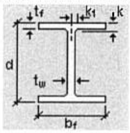
Member : 2 (column)
Design status : OK

A17

Section information

Section name: W 18X76 **(US)**

Dimensions



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d	=	18.200	[in]	Depth
k	=	1.080	[in]	Distance k
k1	=	1.063	[in]	Distance k1
tf	=	0.680	[in]	Flange thickness
tw	=	0.425	[in]	Web thickness

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')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	7.723	2.611
Radius of gyration (principal axes) (r')	[in]	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)	[in]	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)	[in]	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

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	17.00

Distance between member lateral bracing points

Length (Lb) [ft] Top	Bottom
17.00	17.00

Laterally unbraced length

Length [ft] Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Effective length factor Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
17.00	17.00	17.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✓

Axial tension

Ratio	: 0.00			
Capacity	: 667.66 [Kip]	Reference	: Eq. Sec. D2
Demand	: 0.00 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(P_n/Ω)	[Kip]	667.66	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✓

Compression in the major axis 33

Ratio	: 0.09		
Capacity	: 630.24 [Kip]	Reference	: Sec. E1
Demand	: 55.85 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(P_{n33}/Ω)	[Kip]	630.24	Sec. E1

Compression in the minor axis 22

Ratio	: 0.13		
Capacity	: 427.25 [Kip]	Reference	: Sec. E1
Demand	: 55.85 [Kip]	Ctrl Eq.	: LC1 at 0.00%

A18

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(P_{n22}/Ω)	[Kip]	427.25	Sec. E1
Factored torsional or flexural-torsional buckling strength(P_{n11}/Ω)	[Kip]	506.21	Sec. E4

FLEXURAL DESIGN ✓

Bending about major axis, M33

Ratio	: 0.87		
Capacity	: 406.69 [Kip*ft]	Reference	: Sec. F1
Demand	: 354.03 [Kip*ft]	Ctrl Eq.	: LC1 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(M_n/Ω)	[Kip*ft]	406.69	Sec. F1
Factored lateral-torsional buckling strength(M_n/Ω)	[Kip*ft]	406.69	Sec. F1

Bending about minor axis, M22

Ratio	: 0.00		
Capacity	: 105.29 [Kip*ft]	Reference	: Sec. F1
Demand	: 0.00 [Kip*ft]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(M_n/Ω)	[Kip*ft]	105.29	Sec. F1

DESIGN FOR SHEAR ✓

Shear in major axis 33

Ratio	: 0.00		
Capacity	: 268.74 [Kip]	Reference	: Sec. G1
Demand	: 0.00 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(V_n/Ω)	[Kip]	268.74	Sec. G1

Shear in minor axis 22

Ratio	: 0.13		
Capacity	: 154.80 [Kip]	Reference	: Sec. G2.1(a)
Demand	: 20.83 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(V_n/Ω)	[Kip]	154.80	Sec. G2.1(a)

COMBINED ACTIONS DESIGN ✓

Combined flexure and axial compression

Ratio	: 0.91		
Ctrl Eq.	: LC1 at 100.00%	Reference	: Eq. H1-1b

Intermediate results	Unit	Value	Reference
Interaction for doubly symmetric members for in-plane bending	--	0.91	Eq. H1-1b
Interaction for doubly symmetric members for out-of-plane bending	--	0.46	Eq. H1-2

Combined flexure and axial tension

Ratio	: 0.87		
Ctrl Eq.	: LC1 at 100.00%	Reference	: Eq. H1-1b

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio	: N/A		
Ctrl Eq.	: --	Reference	:

Combined flexure and axial tension about local axis

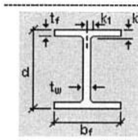
Ratio	: N/A		
Ctrl Eq.	: --	Reference	:

Member : 3 (beam)
Design status : OK

Section information

Section name: W 27X94 **(US)**

Dimensions



bf	=	10.000	[in]	Width
d	=	26.900	[in]	Depth
k	=	1.340	[in]	Distance k
k1	=	1.063	[in]	Distance k1
tf	=	0.745	[in]	Flange thickness
tw	=	0.490	[in]	Web thickness

A19

Properties

Section properties	Unit	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')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	10.865	2.116
Radius of gyration (principal axes) (r')	[in]	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)	[in]	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)	[in]	11.069	
Area for shear (Aw)	[in2]	14.900	13.180
Torsional constant. (C)	[in3]	5.077	

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

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	31.62

Distance between member lateral bracing points

Length (Lb) [ft] Top	Bottom
0.00	31.62

Laterally unbraced length

Length [ft] Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Effective length factor Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
31.62	31.62	31.62	1.0	1.0	1.0

Additional assumptions	
Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✓

Axial tension

Ratio	: 0.00		
Capacity	: 829.34 [Kip]	Reference	: Eq. Sec. D2
Demand	: 0.00 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(P_n/Ω)	[Kip]	829.34	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✓

Compression in the major axis 33

Ratio	: 0.05		
Capacity	: 701.52 [Kip]	Reference	: Sec. E1
Demand	: 37.42 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(P_{n33}/Ω)	[Kip]	701.52	Sec. E1

Compression in the minor axis 22

Ratio	: 0.29		
Capacity	: 129.43 [Kip]	Reference	: Sec. E1
Demand	: 37.42 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(P_{n22}/Ω)	[Kip]	129.43	Sec. E1
Factored torsional or flexural-torsional buckling strength(P_{n11}/Ω)	[Kip]	367.56	Sec. E4

FLEXURAL DESIGN ✓

Bending about major axis, M33

Ratio	: 0.87		
Capacity	: 408.44 [Kip*ft]	Reference	: Sec. F1
Demand	: -354.03 [Kip*ft]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(M_n/Ω)	[Kip*ft]	693.61	Sec. F1
Factored lateral-torsional buckling strength(M_n/Ω)	[Kip*ft]	408.44	Sec. F1

Bending about minor axis, M22

A20

Ratio	: 0.00			
Capacity	: 96.81 [Kip*ft]	Reference	: Sec. F1	
Demand	: 0.00 [Kip*ft]	Ctrl Eq.	: LC1 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(M_n/Ω)	[Kip*ft]	96.81	Sec. F1

DESIGN FOR SHEAR ✓

Shear in major axis 33

Ratio	: 0.00		
Capacity	: 267.66 [Kip]	Reference	: Sec. G1
Demand	: 0.00 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(V_n/Ω)	[Kip]	267.66	Sec. G1

Shear in minor axis 22

Ratio	: 0.18		
Capacity	: 263.60 [Kip]	Reference	: Sec. G2.1(a)
Demand	: 46.39 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(V_n/Ω)	[Kip]	263.60	Sec. G2.1(a)

COMBINED ACTIONS DESIGN ✓

Combined flexure and axial compression

Ratio	: 0.89		
Ctrl Eq.	: LC1 at 0.00%	Reference	: Eq. H1-1b

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

Combined flexure and axial tension

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

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio	: N/A		
Ctrl Eq.	: --	Reference	:

Combined flexure and axial tension about local axis

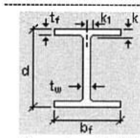
Ratio	: N/A		
Ctrl Eq.	: --	Reference	:

Member : 4 (beam)
Design status : OK

Section information

Section name: W 27X94 **(US)**

Dimensions



bf	=	10.000	[in]	Width
d	=	26.900	[in]	Depth
k	=	1.340	[in]	Distance k
k1	=	1.063	[in]	Distance k1
tf	=	0.745	[in]	Flange thickness
tw	=	0.490	[in]	Web thickness

Properties

Section properties	Unit	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')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	10.865	2.116
Radius of gyration (principal axes) (r')	[in]	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)	[in]	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)	[in]	11.069	
Area for shear (Aw)	[in2]	14.900	13.180
Torsional constant. (C)	[in3]	5.077	

Material : A992 Gr50

A21

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

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	31.62

Distance between member lateral bracing points

Length (Lb) [ft] Top	Bottom
0.00	31.62

Laterally unbraced length

Length [ft] Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Effective length factor Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
31.62	31.62	31.62	1.0	1.0	1.0

Additional assumptions	
Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✓

Axial tension

Ratio	: 0.00		
Capacity	: 829.34 [Kip]	Reference	: Eq. Sec. D2
Demand	: 0.00 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(P_n/Ω)	[Kip]	829.34	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✓

Compression in the major axis 33

Ratio	: 0.05		
Capacity	: 701.52 [Kip]	Reference	: Sec. E1
Demand	: 37.42 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(P_{n33}/Ω)	[Kip]	701.52	Sec. E1

Compression in the minor axis 22

Ratio	: 0.29		
Capacity	: 129.43 [Kip]	Reference	: Sec. E1
Demand	: 37.42 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(P_{n22}/Ω)	[Kip]	129.43	Sec. E1
Factored torsional or flexural-torsional buckling strength(P_{n11}/Ω)	[Kip]	367.56	Sec. E4

FLEXURAL DESIGN ✓

Bending about major axis, M33

Ratio	: 0.87		
Capacity	: 408.44 [Kip*ft]	Reference	: Sec. F1
Demand	: -354.03 [Kip*ft]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(M_n/Ω)	[Kip*ft]	693.61	Sec. F1
Factored lateral-torsional buckling strength(M_n/Ω)	[Kip*ft]	408.44	Sec. F1

Bending about minor axis, M22

Ratio	: 0.00		
Capacity	: 96.81 [Kip*ft]	Reference	: Sec. F1
Demand	: 0.00 [Kip*ft]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(M_n/Ω)	[Kip*ft]	96.81	Sec. F1

DESIGN FOR SHEAR ✓

Shear in major axis 33

Ratio	: 0.00		
Capacity	: 267.66 [Kip]	Reference	: Sec. G1
Demand	: 0.00 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(V_n/Ω)	[Kip]	267.66	Sec. G1

A22

Shear in minor axis 22

Ratio	: 0.18		
Capacity	: 263.60 [Kip]	Reference	: Sec. G2.1(a)
Demand	: 46.39 [Kip]	Ctrl Eq.	: LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(V_n/Ω)	[Kip]	263.60	Sec. G2.1(a)

COMBINED ACTIONS DESIGN ✓

Combined flexure and axial compression

Ratio	: 0.89		
Ctrl Eq.	: LC1 at 0.00%	Reference	: Eq. H1-1b

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

Combined flexure and axial tension

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

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio	: N/A		
Ctrl Eq.	: --	Reference	:

Combined flexure and axial tension about local axis

Ratio	: N/A		
Ctrl Eq.	: --	Reference	:

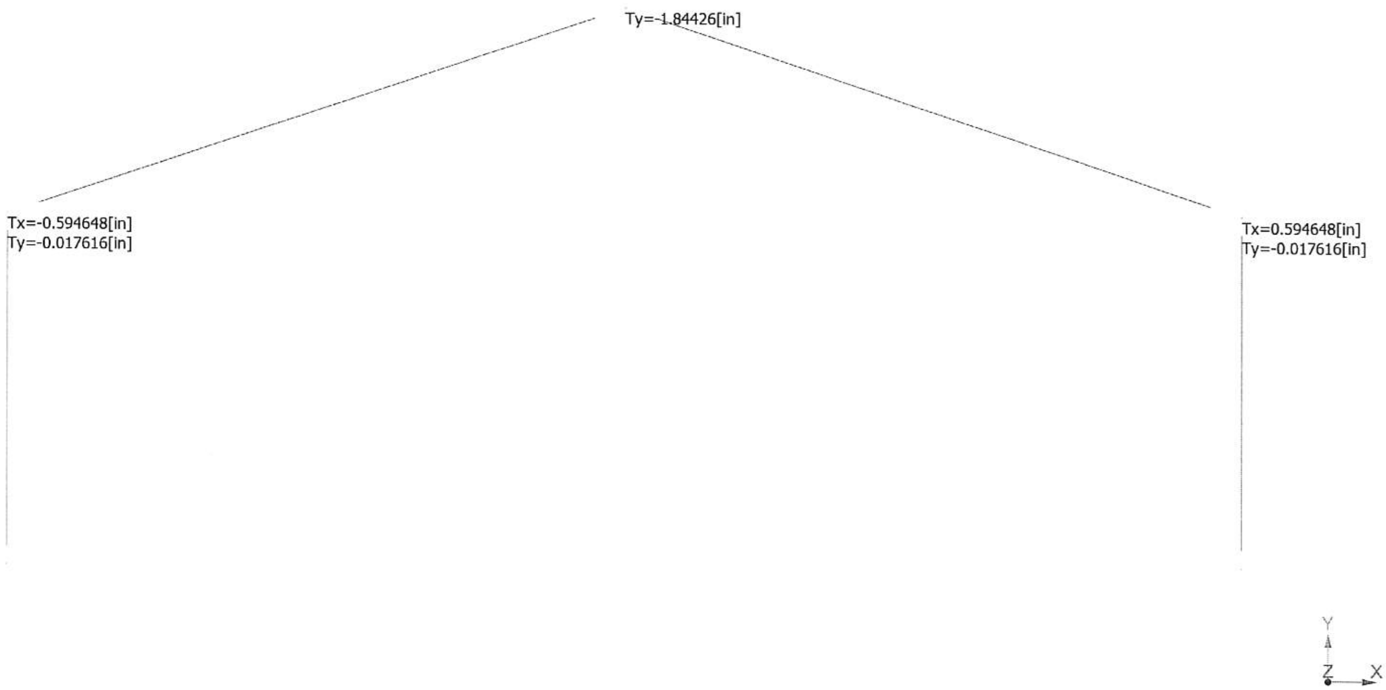
A23

Bentley

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

A24

L/391



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Current Date: 8/2/2018 9:54 AM
Units system: English
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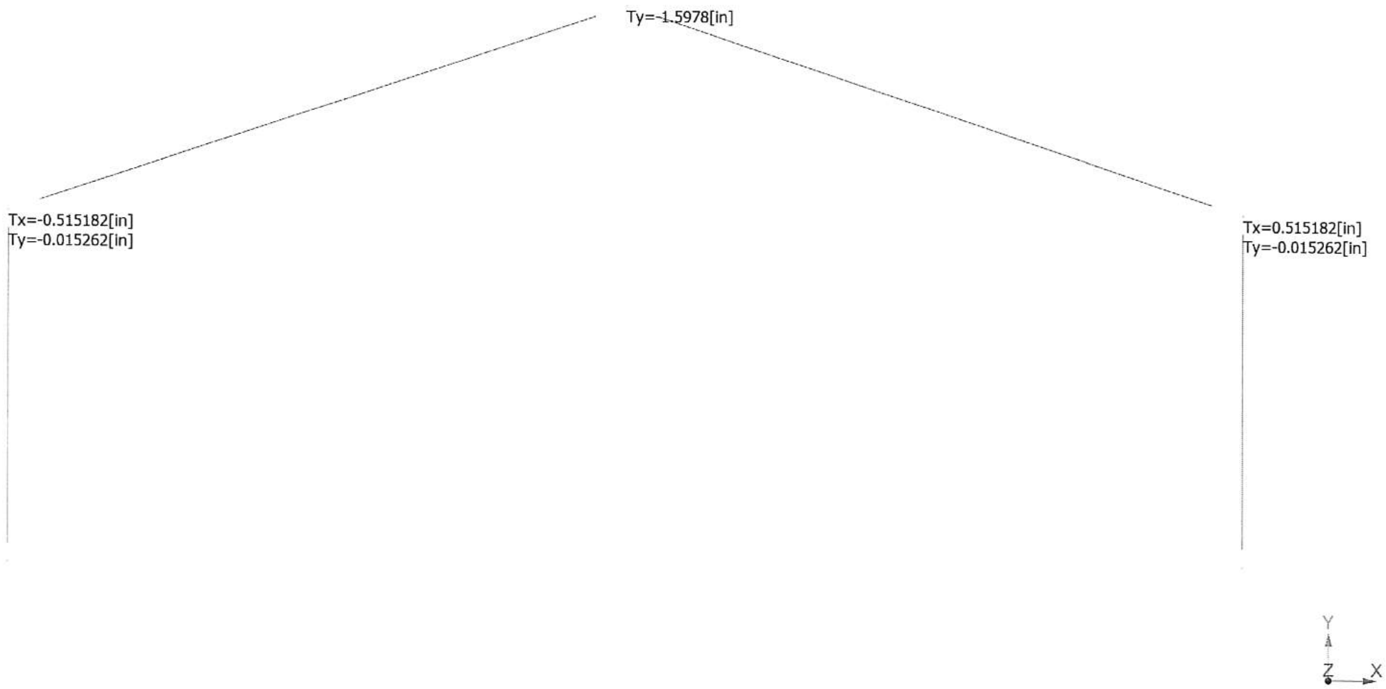
Load condition: SL=Snow Load

A25

L/450



![Bentley]

A26

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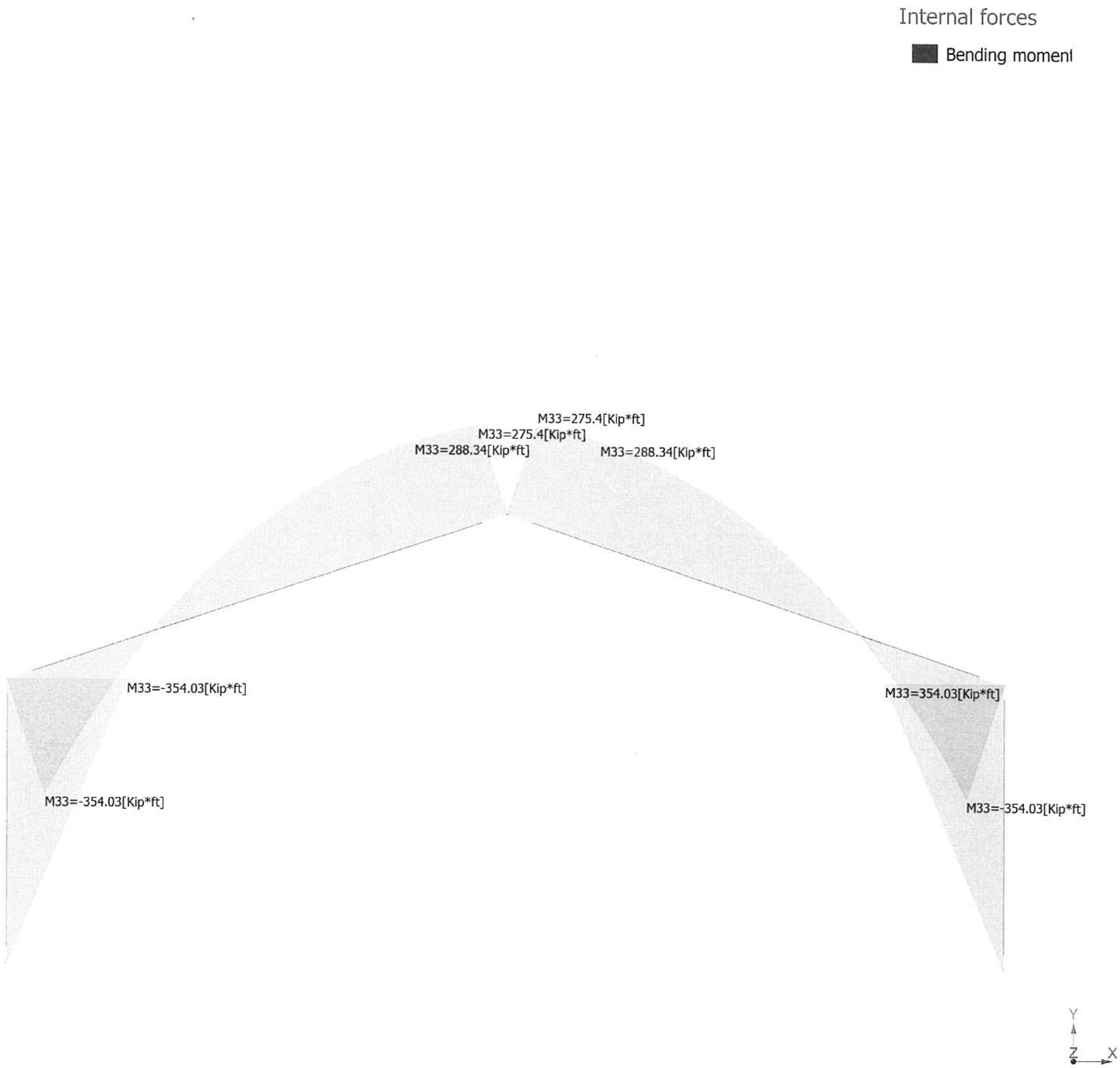

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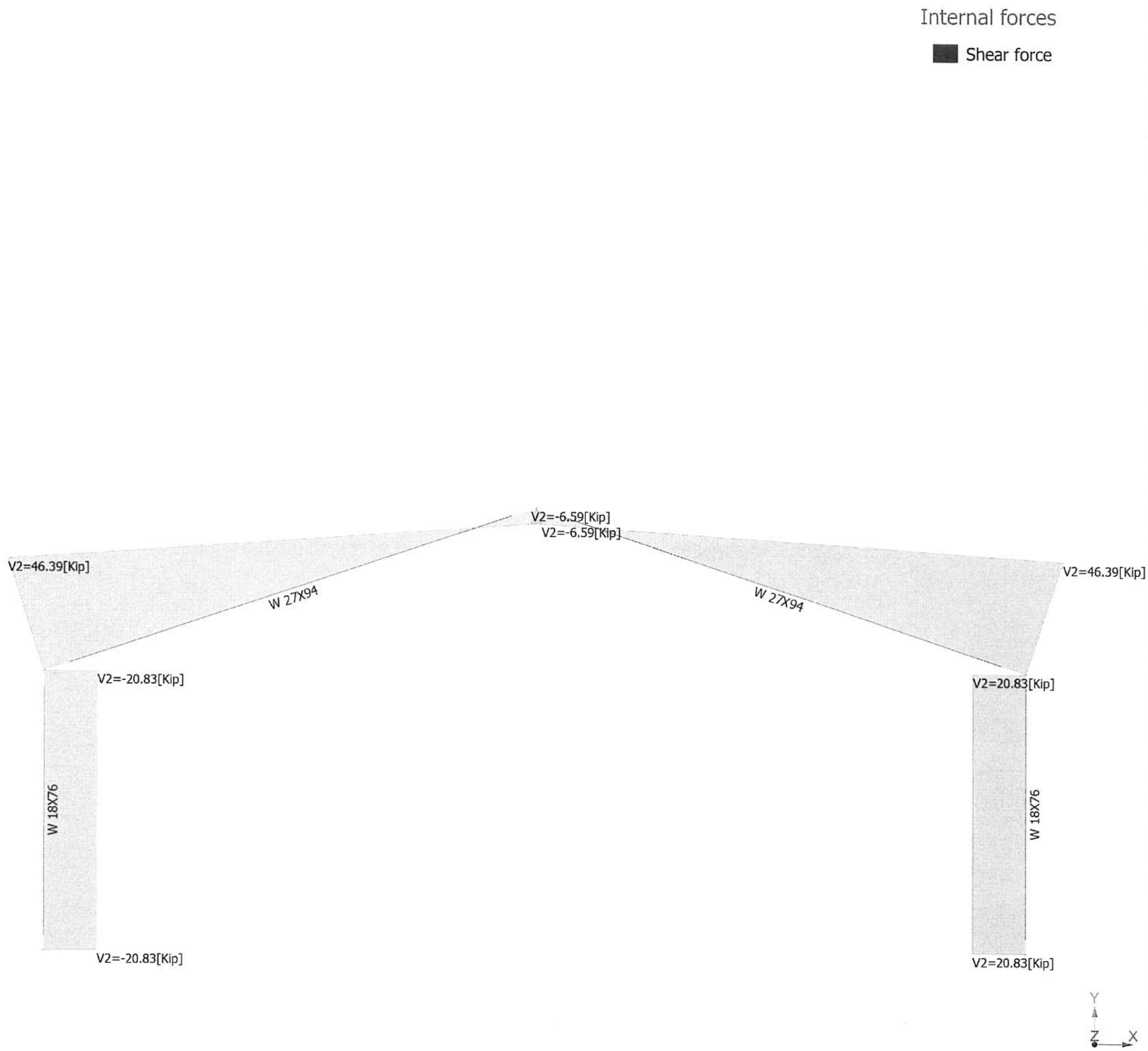
A27



Bentley

A28

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Units system: English
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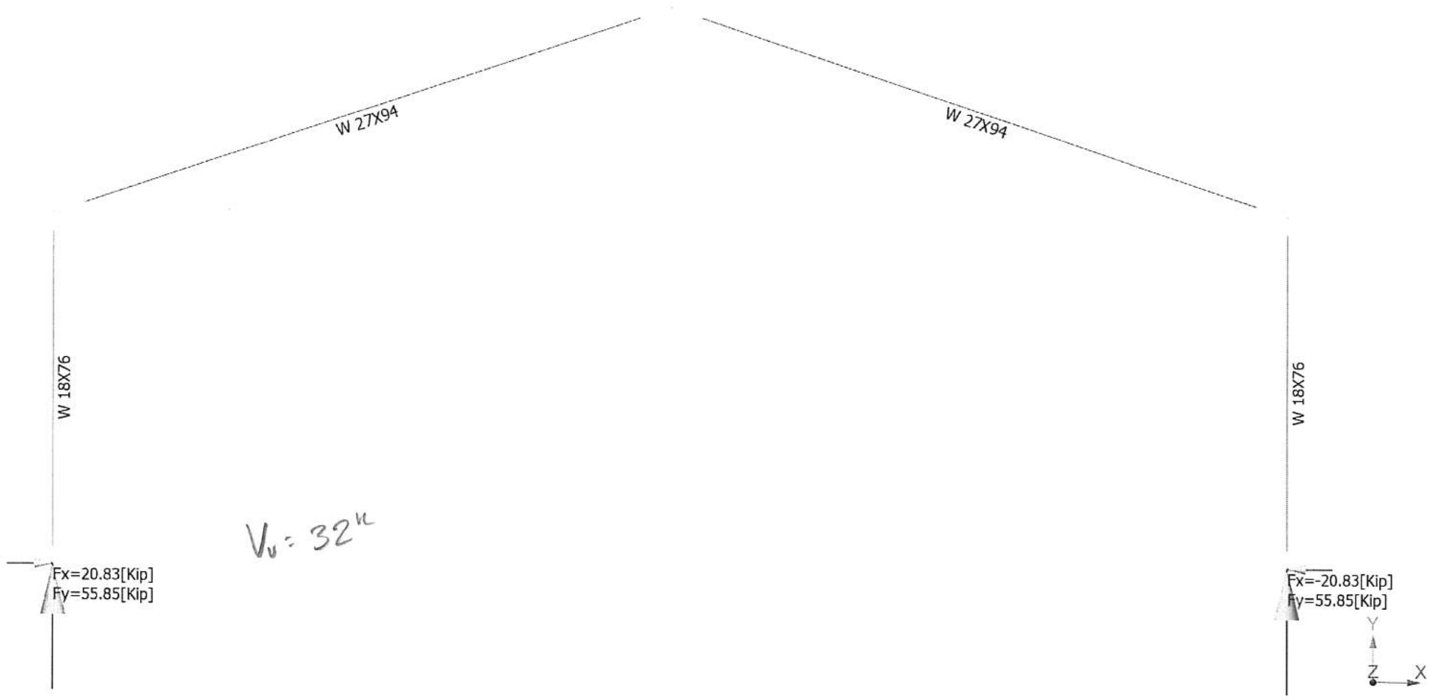
Load condition: LC1=DL+SL



FOOTING

$$W = \sqrt{\frac{56^k}{1.8_{ksf}}} = 5.6' \quad \underline{\underline{\text{USE } F6}}$$

LATERAL

$$\underset{\text{FRICTION}}{0.3(56^k)} + \underset{\text{PASSIVE SOIL}}{0.3_{KSF}(3.5')\tfrac{1}{2}(3.5')(15')} = 44.4^k > 20.8^k \quad \underline{\text{OK}}$$

A29

ENGINEERS

03-Aug-18
1:46 PM

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

Version Date: December 5, 2017 Author: Troy M. Dye Reviewed By: Scott Porter

JOB TITLE: NAC Recreation Center JOB #: 18121
DESCRIPTION: Typical W columns DESIGNER: TMD

Pier and Anchor Bolt Geometry

	Min Edge or Spacing		Bolt Quantities
Top Pier Edge	10		
AB-f5	0		
AB-f4	0		
AB-f3	0		
AB-f2	0		
AB-f1	2		2
Column centerline	0	<<	
AB-b1	2		2
AB-b2	0		
AB-b3	0		
AB-b4	0		
AB-b5	0		
Bottom Pier Edge	10		4 # Bolts in Group

Left Pier Edge	7	Pier Dimensions	
AB-l1	2	18.0	" wide
Column centerline	0 <<	24.0	" long
AB-r1	2	Bolt Group Dimensions	
Right Pier Edge	7	4.0	" wide
		4.0	" long

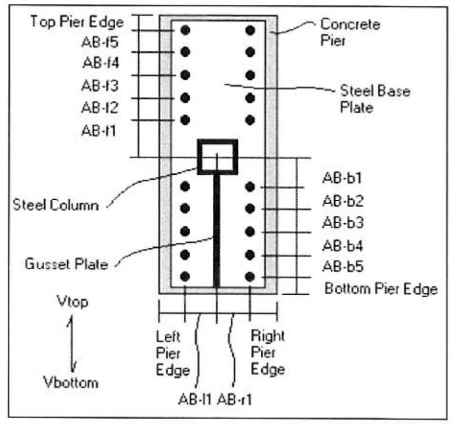


Figure 1. Generic Anchor Bolt Locations

Design Loads

Shear V_u : **32** kips *Use ACI 318 Section 5.3 load combinations*

Tension P_u: **0** kips

Is the seismic tension component more than 20% of the total factored tension force? NO

Increase Shear Vu and Tension Pu above by Omega

Pier confined for tension force? YES
Pier confined for shear force? YES

Anchors Torqued: NO
Bolt Grade: A36
Bolt Diameter: 1 inches
Embedment Length: 12 inches
Concrete f'_c: 3000 psi
Rebar f_y: 60000 psi
Anchor F_u : 58 ksi
Bolt Area A_{bolt} : 0.61 in^2
Seismic Design Category Factor: 1 *Section 17.2.3.4.4*

Pier Ht / Ftg Thick: 30 inches **OK**
Bolt type: H

Omega Level Force 17.2.3.4.3d & 17.2.3.5.3c

Bearing Area:	0.98	in^2
Steel ϕ_t:	0.75	*Section 17.3.3*
Steel ϕ_s:	0.65	*Section 17.3.3*
Concrete ϕ:	0.75	*Section 17.3.3*
λ :	1	*Section 19.2.4*
λ_a :	1	*Section 17.2.6*

Eccentricity for Tension
0 in
Eccentricity for Shear
0

Bolt Diameter (in^2)	*Threads per (inch)*	*Tensile Stress Area (in^2)*
0.5	13	0.14
0.625	11	0.23
0.75	10	0.33
0.875	9	0.46
1	8	0.61
1.125	7	0.76
1.25	7	0.97
1.375	6	1.15
1.5	6	1.41
1.75	5	1.90
2	4.5	2.50
2.25	4.5	3.25
2.5	4	4.00
2.75	4	4.93
3	4	5.97

Tension Calculations

A_N : 432 in^2
A_{NO} : 400 in^2

	c_{a2}			c_{a1}	
Left Edge Dist. :	7	inches	Top Edge Dist. :	10	inches
Right Edge Dist. :	7	inches	Bottom Edge Dist. :	10	inches

Concrete Tension Capacity (Section 17.4.2) - Breakout Strength

$\psi_{ec,N}$: 1.00 $\psi_{ed,N}$: 0.82 $\psi_{c,N}$: 1

N_b = 20.7 kips N_{cb} = 18.3 kips

Concrete Tension Capacity (Section 17.4.3) - Pullout Strength

$\psi_{c,P}$: 1 N_{pn} = 23.5 kips

N_p = 23.5 kips Capacity of Group: 94.1 kips

Concrete Tension Capacity (Section 17.4.4) - Side-Face Blowout Strength

Use A_N : 432 in^2

N_{sb} = N/A kips N_{sbg} = N/A kips

ϕP_c : 70.6 kips **P_c :** $\boxed{94.1}$ kips Ultimate Concrete Strength Based on Pullout Only

Steel Tension Capacity (Section 17.4.1)

P_{ss}' : 35.1 kips / AB **P_{ss} :** $\boxed{140.5}$ kips Ultimate Steel Tension Capacity

ARW ENGINEERS

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 #: 18121
DESIGNER: TMD

Shear Calculations

Bolts Resisting Shear

B — Specify which system resists shear
Bolts resist tension and shear

Steel Shear Capacity (Section 17.5.1)

V_{sa} : 16.86 kips / AB — V_{sa} : $\boxed{67.5}$ kips — Ultimate Steel Shear Capacity based on Assumed Breakout Surface

n : 4 Number of anchors resisting shear based on assumed concrete breakout surface

Grout height below base plate, h_2 : 1.5 inches

Concrete Shear Breakout Capacity (Section 17.5.2)

$\psi_{ec,V}$: 1.00 $\psi_{ed,V}$: 0.84 $\psi_{c,V}$: 1.2 $\psi_{h,V}$: 1.00

Top Edge Bolts			Bottom Edge Bolts		
Edge Distance:	10.0	inches	Edge Distance :	10.0	inches
A_{Vc} =	270.0	in^2	A_{Vc} =	270.0	in^2
A_{Vco} =	450.0	in^2	A_{Vco} =	450.0	in^2
V_b =	18.4	kips	V_b =	18.4	kips
V_{cb} =	11.1	kips	V_{cb} =	11.1	kips

Concrete Shear Pryout Capacity (Section 17.5.3)

V_{cp} =	97.2	kips	N_b =	55.1	kips
Group Capacity V_n:	$\boxed{97.2}$	kips	Group Capacity V_n:	$\boxed{97.2}$	kips

Ultimate Concrete Capacity Based on Pryout Strength Only

Combined Tension and Shear

Shear Force Acting Towards Top Pier Edge, V_{top}

0.439	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)
0.730	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)

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

Shear Force Acting Towards Bottom Pier Edge, V_{bottom}

0.000	< 1 - OK	Concrete Tension ($P_u/\phi P_n$)
0.439	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)
0.439	< 1 - OK	Concrete Combined (Section D.7)
0.000	< 1 - OK	Steel Tension ($P_u/\phi P_{ss}$)
0.730	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)
0.730	< 1 - OK	Steel Combined (Section D.7)

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

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

A31

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

Version Date: December 5, 2017

JOB TITLE: NAC Recreation Center

DESCRIPTION: Typical W columns

03-Aug-18

1:46 PM

JOB #: 18121

DESIGNER: TMD

ENGINEERS

Anchor Reinforcing

Reinforcing Data

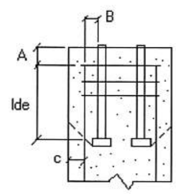
ψt :	1
ψe :	1
ψs :	1
γ :	0.8

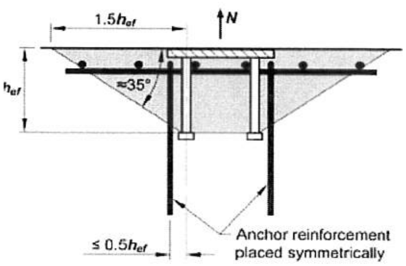
Assumes that vertical reinforcement layout is symmetrical around anchor bolt pattern

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) :	4	in
Cover above vert. reinf (A) :	2	in
c :	3	in
Rebar Area :	0.31	in^2
Rebar Diameter :	0.625	in
l_{db} :	21.36	in
l_{de} :	6.00	in
0.75*F_y of Rebar @ l_{de} - f_s :	12.64	ksi
A_{st} :	0.00	in^2
Total # of vertical bars required :	1	Quantity of reinforcement placed symmetrically around anchor bolts
Embedment of standard hook :	6.00	in





Section A-A

Pier Reinforcement to Resist Shear Breakout (17.5.2.9)

Hairpin/stirrup reinforcing size :	4	bar
Rebar area :	0.4	in^2
A_{st} :	0.71	in^2
Total # of hairpins/stirrups required :	2	Quantity of hairpins or stirrups wrapped around anchor bolts

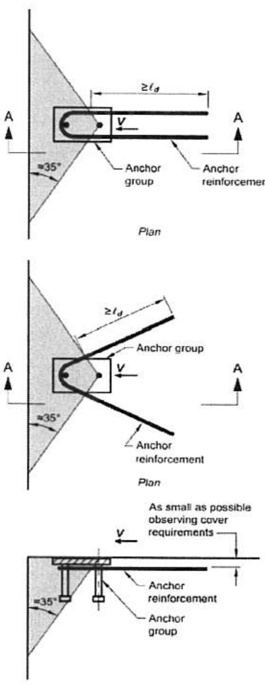


Fig. RD.6.2.9(a)—Hairpin anchor reinforcement for shear.

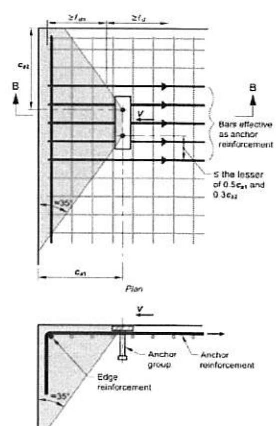


Fig. RD.6.2.9(b)—Edge reinforcement and anchor reinforcement for shear.

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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:

A32

Printed: 2 AUG 2018, 8:33AM

Wood Beam

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Description : Vestibule Glulam Beam

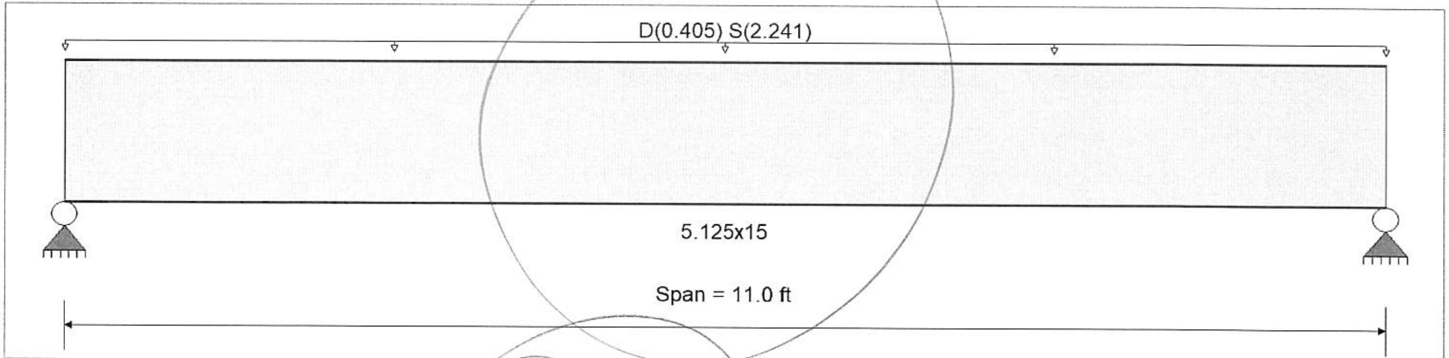
CODE REFERENCES

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

Material Properties

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

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



Applied Loads

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	=	**0.905**: 1	Maximum Shear Stress Ratio	=	**0.721** : 1
Section used for this span		**5.125x15**	Section used for this span		**5.125x15**
fb : Actual	=	2,498.86psi	fv : Actual	=	219.71 psi
FB : Allowable	=	2,760.00psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.500ft	Location of maximum on span	=	9.755 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.286 in	Ratio =	461 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	0.338 in	Ratio =	390 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D 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

Wood Beam

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Description : Vestibule Glulam Beam

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.3379	5.540		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	14.553	14.553
Overall MINimum	12.326	12.326
D Only	2.228	2.228
+D+S	14.553	14.553
+D+0.750S	11.472	11.472
+0.60D	1.337	1.337
S Only	12.326	12.326

A34

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Steel Column

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Description : 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 :	**HSS4x4x1/4**
Analysis Method :	Allowable Strength
Steel Stress Grade	, A500, Grade B, Fy = 46 ksi, Carbon
Fy : Steel Yield	46.0 ksi
E : Elastic Bending Modulus	29000 ksi

Overall Column Height 12.0 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 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

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

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 =	**0.003988** : 1
	Load Combination	+D+S
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Va : Applied	0.1014 k
	Vn / Omega : Allowable	25.423 k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top 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	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.046	PASS	0.00 ft	0.001	PASS	0.00 ft
+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

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
D Only	2.346			-0.015	0.015				
+D+S	14.746			-0.101	0.101				
+D+0.750S	11.646			-0.080	0.080				
+0.60D	1.408			-0.009	0.009				
S Only	12.400			-0.086	0.086				

$A = \frac{14.7^k}{1.8 \text{ KSF}} = 8.2 \text{ FT}^2$

$L = \frac{8.2'}{2.5'} = 3.2'$

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	14.746			-0.101	0.101				
"	Minimum	1.408			-0.009	0.009				

FC2.5 OK

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Project Title:
Engineer:
Project ID:
Project Descr:

A35

Printed: 2 AUG 2018, 10:48AM

Steel Column

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Description : Vestibule Column under GLB

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top (k)	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top (k-ft)	My - End Moments @ Base	My - End Moments @ Top
Reaction, X-X Axis Base	Maximum	2.346			-0.015	0.015				
"	Minimum	2.346			-0.015	0.015				
Reaction, Y-Y Axis Base	Maximum	1.408			-0.009	0.009				
"	Minimum	14.746			-0.101	0.101				
Reaction, X-X Axis Top	Maximum	2.346			-0.015	0.015				
"	Minimum	2.346			-0.015	0.015				
Reaction, Y-Y Axis Top	Maximum	2.346			-0.015	0.015				
"	Minimum	12.400			-0.086	0.086				
Moment, X-X Axis Base	Maximum	2.346			-0.015	0.015				
"	Minimum	2.346			-0.015	0.015				
Moment, Y-Y Axis Base	Maximum	2.346			-0.015	0.015				
"	Minimum	2.346			-0.015	0.015				
Moment, X-X Axis Top	Maximum	2.346			-0.015	0.015				
"	Minimum	2.346			-0.015	0.015				
Moment, Y-Y Axis Top	Maximum	2.346			-0.015	0.015				
"	Minimum	2.346			-0.015	0.015				

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	-0.013	in	7.007	ft
+D+S	0.0000	in	0.000	ft	-0.087	in	7.007	ft
+D+0.750S	0.0000	in	0.000	ft	-0.068	in	7.007	ft
+0.60D	0.0000	in	0.000	ft	-0.008	in	7.007	ft
S Only	0.0000	in	0.000	ft	-0.074	in	7.007	ft

Steel Section Properties : HSS4x4x1/4

Depth	=	4.000 in	I xx	=	7.80 in^4	J	=	12.800 in^4
Design Thick	=	0.233 in	S xx	=	3.90 in^3			
Width	=	4.000 in	R xx	=	1.520 in			
Wall Thick	=	0.250 in	Zx	=	4.690 in^3			
Area	=	3.370 in^2	I yy	=	7.800 in^4	C	=	6.560 in^3
Weight	=	12.181 plf	S yy	=	3.900 in^3			
			R yy	=	1.520 in			
Ycg	=	0.000 in						

Steel Column

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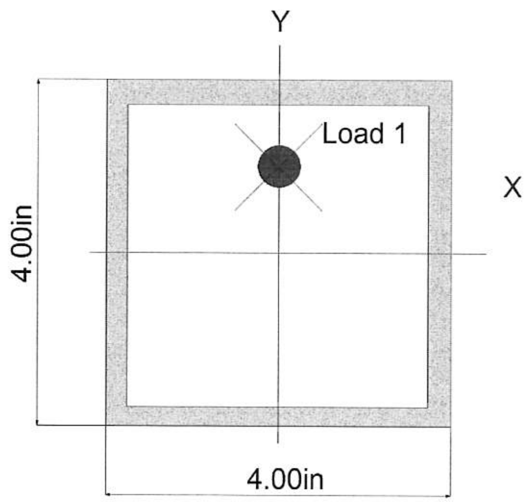
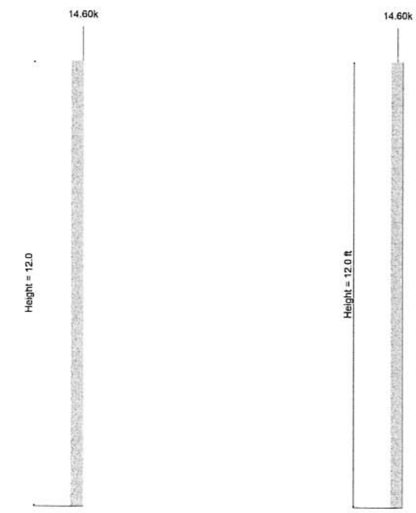
Description : Vestibule Column under GLB

Sketches





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Project Title:
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Project ID:
Project Descr:

A37

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Wood Beam

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

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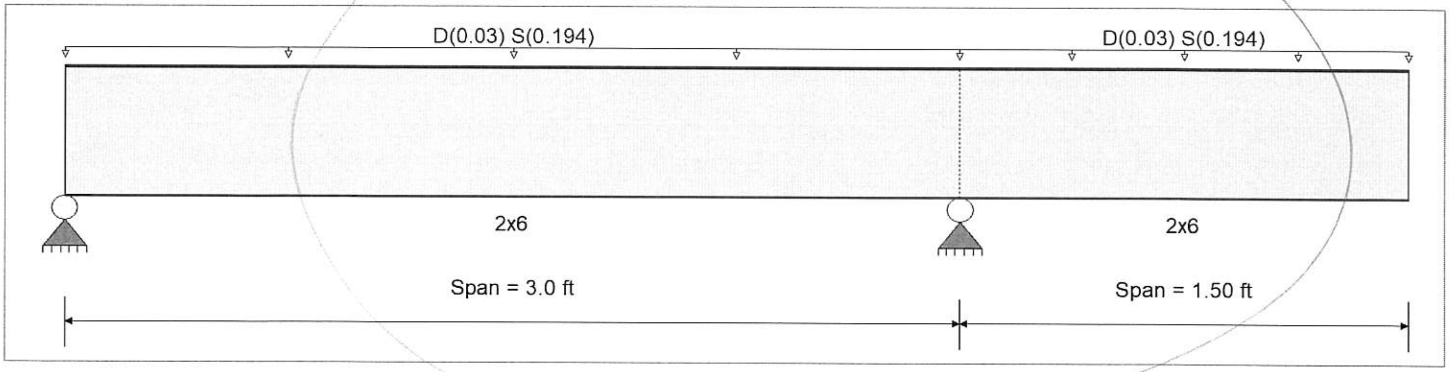
Description : Outlookers

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 Elasticity*	
Load Combination :	ASCE 7-10	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625.0 psi		
Wood Grade	: No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.20 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increase	



Applied Loads

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

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

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.258 : 1**	Maximum Shear Stress Ratio	=	**0.280 : 1**
Section used for this span		**2x6**	Section used for this span		**2x6**
fb : Actual	=	399.87 psi	fv : Actual	=	57.93 psi
FB : Allowable	=	1,547.33 psi	Fv : Allowable	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.000 ft	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 Deflection	0.006 in	Ratio =	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 =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D 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	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

Wood Beam

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Description : Outlookers

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 1.50 ft	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
+D+0.750S					1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.202	0.219	1.15	1.300	1.00	1.15	1.00	1.00	1.00	0.20	313.29	1547.33	0.25	45.39	207.00
Length = 1.50 ft	2	0.202	0.219	1.15	1.300	1.00	1.15	1.00	1.00	1.00	0.20	313.29	1547.33	0.18	45.39	207.00
+0.60D					1.300	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.015	0.016	1.60	1.300	1.00	1.15	1.00	1.00	1.00	0.02	32.13	2152.80	0.03	4.66	288.00
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	2152.80	0.02	4.66	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.0052	1.274		0.0000	0.000
+D+S	2	0.0073	1.500		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	0.252	0.756	
Overall MINimum	0.218	0.655	
D Only	0.034	0.101	
+D+S	0.252	0.756	
+D+0.750S	0.197	0.592	
+0.60D	0.020	0.061	
S Only	0.218	0.655	

LATERAL ANALYSIS

B1

ARW ENGINEERS

PROJECT DESIGN CRITERIA

Governing Building Code : IBC2015

WIND DESIGN

Basic Wind Speed, V_{3s} :	115	mph
Wind Importance Factor, I_w :	1	
Wind Exposure :	C	

SEISMIC DESIGN

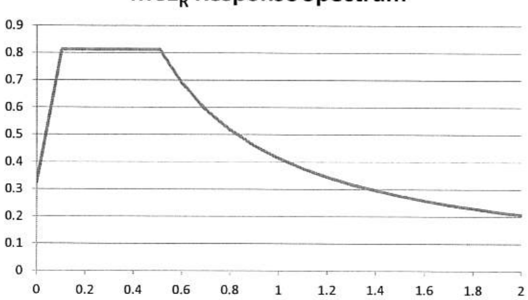
Seismic Importance Factor, I_e :	1	Street:	1000 Ability Way
USGS Design Code:	ASCE 7-10	City:	Park City
Site Class :	D	State:	UT
Seismic Risk Category :	II		
		Latitude :	40.6808539
		Longitude :	-111.475762

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

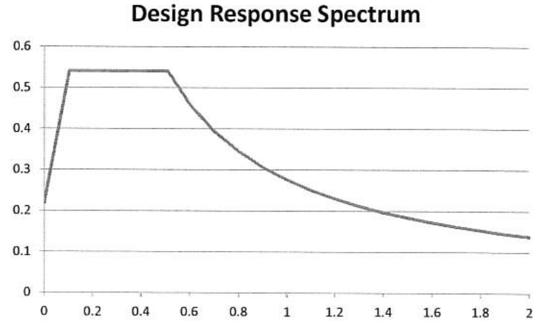
USGS–Provided Output

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_R Response Spectrum



Design Response Spectrum



B2

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

Query Date: Thu Jun 21 2018
Latitude: 40.6809
Longitude: -111.4758

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

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
MRI 100-Year:** 96

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

Use ctrl + scroll to zoom the map

Google

Map data ©2018 Google, INEGI

*Miles per hour
**Mean Recurrence Interval

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

Print your results

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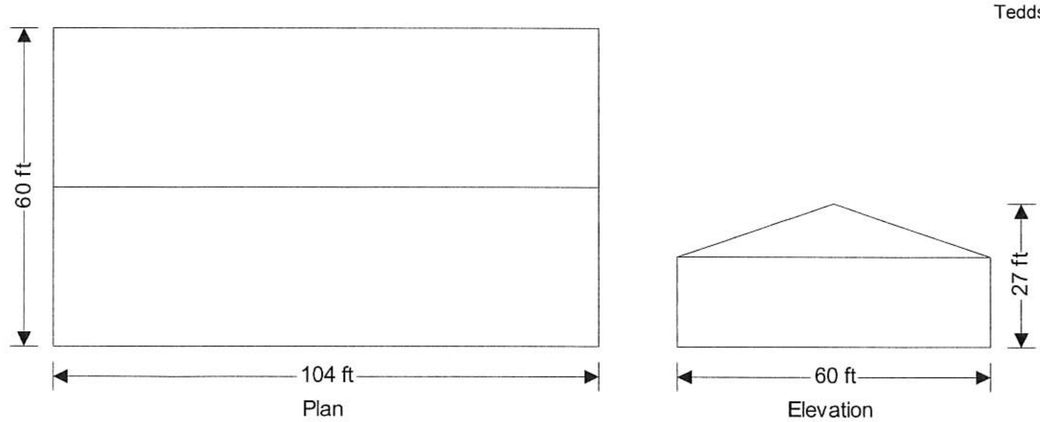
WIND LOADING (ASCE7-10)

HIGH ROOF

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

Using the directional design method

Tedds calculation version 2.0.20



Building data

Type of roof	Gable
Length of building	b = **104.00** ft
Width of building	d = **60.00** ft
Height to eaves	H = **17.00** ft
Pitch of roof	α_0 = **18.5** deg
Mean height	h = **22.02** ft

General wind load requirements

Basic wind speed	V = **115.0** mph
Risk category	II
Velocity pressure exponent coeff (Table 26.6-1)	K_d = **0.85**
Exposure category (cl.26.7.3)	C
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	$q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$

Velocity pressures table

z (ft)	K_z (Table 27.3-1)	q_z (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

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **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, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (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** kips

Total horizontal net force $F_{w,h}$ = **3.49** kips

N-S

Walls load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A_1	15.00	0.80	24.46	11.89	1560.00	18.54
A_2	17.00	0.80	25.04	12.28	208.00	2.55
B	22.02	-0.50	26.36	-15.95	1768.00	-28.20
C	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 $A_{vert\_w\_0} = b \times H$ = **1768.00** ft²

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0)$ = **1043.94** ft²

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** kips

Leeward net force $F_l = F_{w,wB}$ = **-28.2** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2}$ = **21.1** kips

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

Roof load case 2 - Wind 0, GC_{pi} -0.18, $-0c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (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** kips

Total horizontal net force $F_{w,h}$ = **14.48** kips

Walls load case 2 - Wind 0, GC_{pi} -0.18, $-0c_{pe}$

Tekla Tedds ARW Engineers	Project			Job Ref. B5	
	Section			Sheet no./rev. 3	
	Calc. by T	Date 7/2/2018	Chk'd by	Date	App'd by	Date

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	24.46	21.38	1560.00	33.35
A_2	17.00	0.80	25.04	21.77	208.00	4.53
B	22.02	-0.50	26.36	-6.46	1768.00	-11.42
C	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 $A_{vert\_w\_0} = b \times H = \mathbf{1768.00}\ ft^2$

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) = \mathbf{1043.94}\ ft^2$

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = \mathbf{36.64}$ kips

Leeward net force $F_l = F_{w,wB} = \mathbf{-11.4}$ kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} = \mathbf{37.9}$ kips

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

Roof load case 3 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (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 $F_{w,v} = \mathbf{-95.25}$ kips

Total horizontal net force $F_{w,h} = \mathbf{0.00}$ kips

E-W

Walls load case 3 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	24.46	11.89	900.00	10.70
A_2	20.00	0.80	25.90	12.87	273.10	3.51
A_3	27.04	0.80	27.52	13.97	148.08	2.07
B	22.02	-0.35	26.36	-12.66	1321.14	-16.73
C	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 $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 = \mathbf{1321.14}\ ft^2$

Projected vertical area of roof $A_{vert\_r\_90} = \mathbf{0.00}\ ft^2$

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{21.14}$ kips

Leeward net force $F_l = F_{w,wB} = \mathbf{-16.7}$ kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} = \mathbf{16.3}$ kips

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

Roof load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (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** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Walls load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	24.46	21.38	900.00	19.24
A_2	20.00	0.80	25.90	22.36	273.10	6.11
A_3	27.04	0.80	27.52	23.46	148.08	3.47
B	22.02	-0.35	26.36	-3.17	1321.14	-4.19
C	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 $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4$ = **1321.14** ft^2

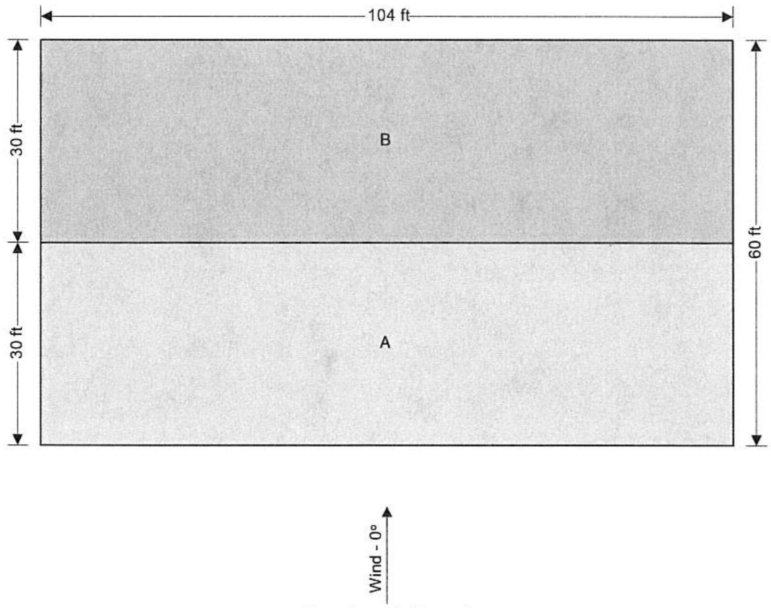
Projected vertical area of roof $A_{vert\_r\_90}$ = **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90}$ = **21.14** kips

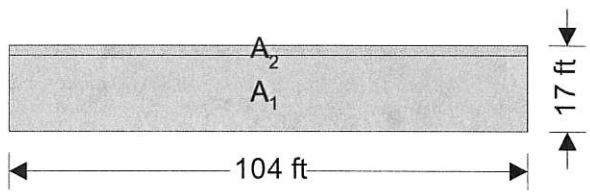
Leeward net force $F_l = F_{w,wB}$ = **-4.2** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3}$ = **28.8** kips

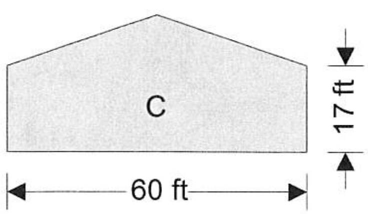
Overall horizontal loading $F_{w,total} = max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **33.0** kips



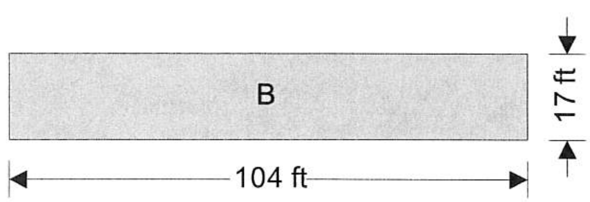
Plan view - Gable roof



Windward face

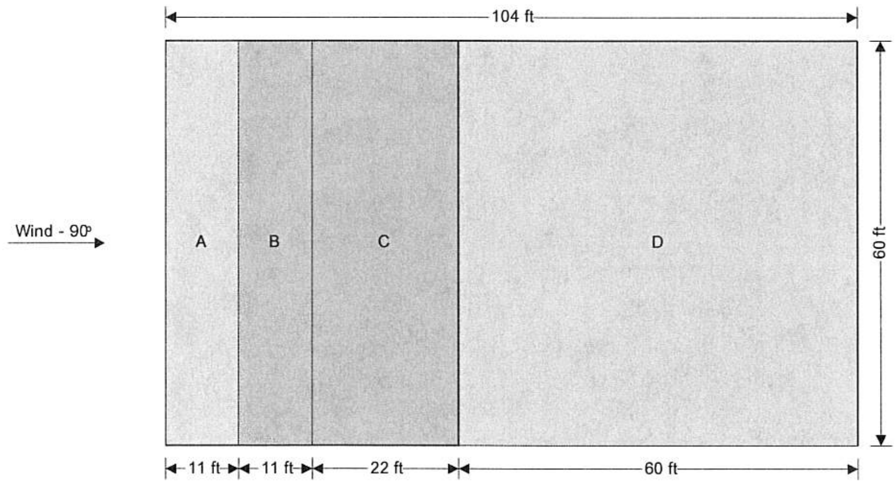


Side face

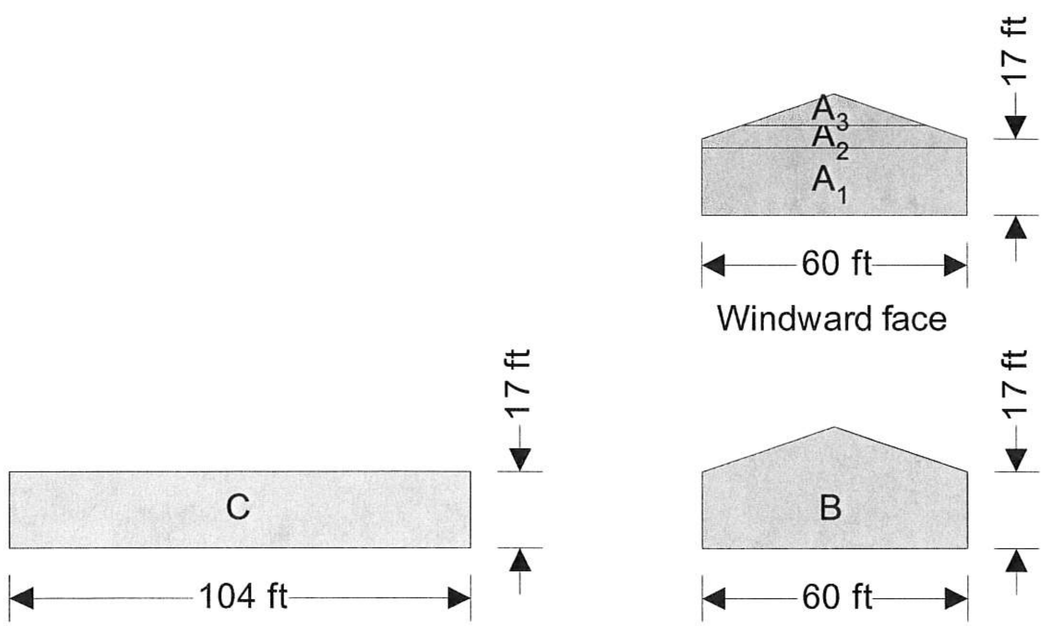


Leeward face

Tekla Tedds ARW Engineers	Project				Job Ref. B8	
	Section				Sheet no./rev. 6	
	Calc. by T	Date 7/2/2018	Chk'd by	Date	App'd by	Date



Plan view - Gable roof



Tekla Tedds ARW Engineers	Project			Job Ref. B9	
	Section			Sheet no./rev. 1	
	Calc. by T	Date 7/2/2018	Chk'd by	Date	App'd by	Date

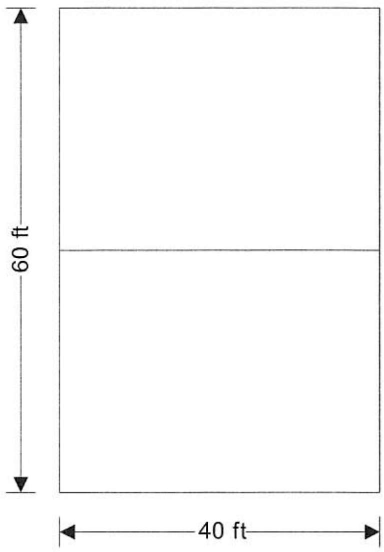
WIND LOADING (ASCE7-10)

LOW ROOF

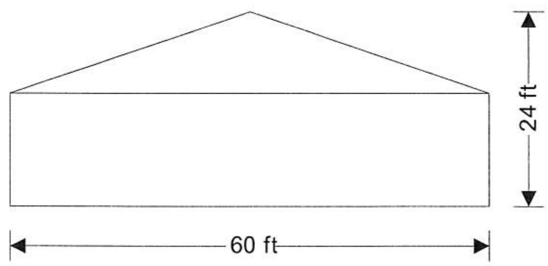
In accordance with ASCE7-10 incorporating Errata No. 1 and Errata No. 2

Using the directional design method

Tedds calculation version 2.0.20



Plan



Elevation

Building data

Type of roof	Gable
Length of building	b = **40.00** ft
Width of building	d = **60.00** ft
Height to eaves	H = **14.00** ft
Pitch of roof	α_0 = **18.5** 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_d = **0.85**
Exposure category (cl.26.7.3)	C
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	$q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$

Velocity pressures table

z (ft)	K_z (Table 27.3-1)	q_z (psf)
14.00	0.85	24.46

z (ft)	K_z (Table 27.3-1)	q_z (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** 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, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (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** kips

Total horizontal net force $F_{w,h}$ = **1.53** kips

N-S

Walls load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A	14.00	0.80	24.46	12.02	560.00	6.73
B	19.02	-0.40	25.62	-13.32	560.00	-7.46
C	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** ft²

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0)$ = **401.51** ft²

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0}$ = **12.17** kips

Leeward net force $F_l = F_{w,wB}$ = **-7.5** kips

Windward net force $F_w = F_{w,wA}$ = **6.7** kips

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

Roof load case 2 - Wind 0, GC_{pi} -0.18, $-0c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (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** kips

Total horizontal net force $F_{w,h}$ = **5.75** kips

Walls load case 2 - Wind 0, GC_{pi} -0.18, -0c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	14.00	0.80	24.46	21.24	560.00	11.90
B	19.02	-0.40	25.62	-4.10	560.00	-2.30
C	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 — $A_{vert\_w\_0} = b \times H = \mathbf{560.00}$ ft^2

Projected vertical area of roof — $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) = \mathbf{401.51}$ ft^2

Minimum overall horizontal loading — $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = \mathbf{12.17}$ kips

Leeward net force — $F_l = F_{w,wB} = \mathbf{-2.3}$ kips

Windward net force — $F_w = F_{w,wA} = \mathbf{11.9}$ kips

Overall horizontal loading — $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{19.9}$ kips

Roof load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (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 — $F_{w,v} = \mathbf{-46.62}$ kips

Total horizontal net force — $F_{w,h} = \mathbf{0.00}$ kips

E-W

Walls load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	24.46	12.02	897.01	10.78
A_2	15.00	0.80	24.46	12.02	0.00	0.00
A_3	24.04	0.80	26.83	13.63	244.18	3.33
B	19.02	-0.50	25.62	-15.50	1141.14	-17.69
C	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 — $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 = \mathbf{1141.14}$ ft^2

Projected vertical area of roof — $A_{vert\_r\_90} = \mathbf{0.00}$ ft^2

Minimum overall horizontal loading — $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{18.26}$ kips

Leeward net force — $F_l = F_{w,wB} = \mathbf{-17.7}$ kips

Windward net force — $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} = \mathbf{14.1}$ kips

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

Roof load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (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** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Walls load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	24.46	21.24	897.01	19.06
A_2	15.00	0.80	24.46	21.24	0.00	0.00
A_3	24.04	0.80	26.83	22.86	244.18	5.58
B	19.02	-0.50	25.62	-6.28	1141.14	-7.16
C	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 $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4$ = **1141.14** ft^2

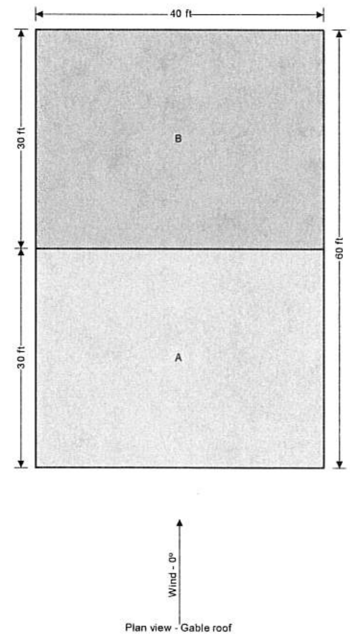
Projected vertical area of roof $A_{vert\_r\_90}$ = **0.00** ft^2

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** kips

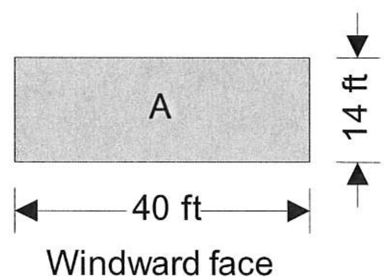
Leeward net force $F_l = F_{w,wB}$ = **-7.2** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3}$ = **24.6** kips

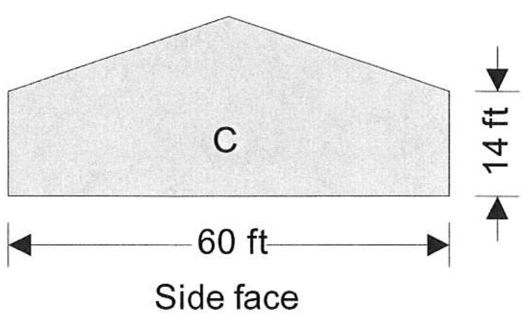
Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **31.8** kips



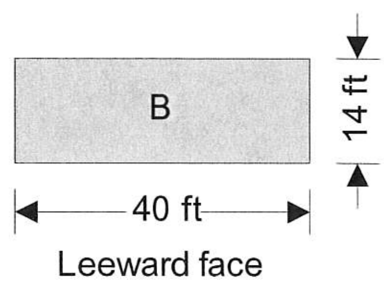
40 ft
30 ft
30 ft
60 ft
B
A
Wind - 0°
Plan view - Gable roof



A
14 ft
40 ft
Windward face

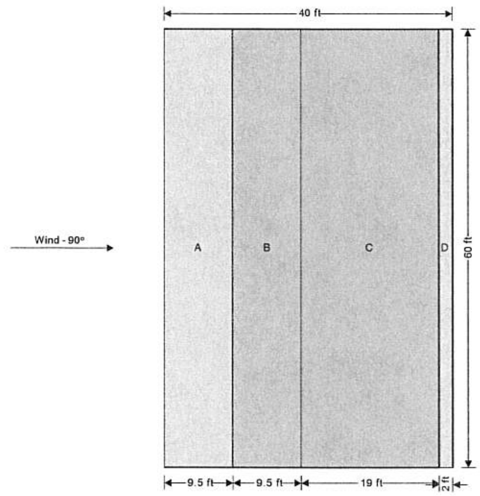


C
14 ft
60 ft
Side face

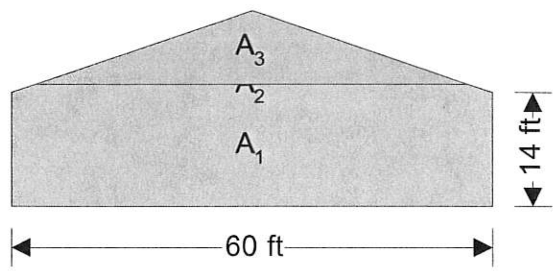


B
14 ft
40 ft
Leeward face

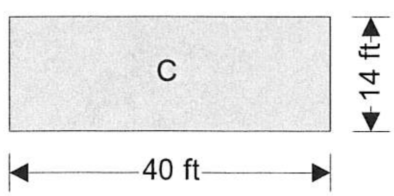
Tekla Tedds ARW Engineers	Project			Job Ref. B14	
	Section			Sheet no./rev. 6	
	Calc. by T	Date 7/2/2018	Chk'd by	Date	App'd by	Date



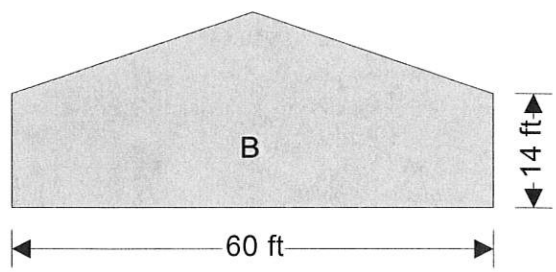
Plan view - Gable roof



Windward face



Side face



Leeward face

LATERAL

AREA A

E

$W_{NS} = 0.08 \left[34.5\text{ PSF} (53') + 2(11\text{ PSF})\left(\frac{17}{2}\right) \right] = 161\text{ PLF}$

$W_{EW} = 0.08 \left[34.5\text{ PSF} (9') + 11\text{ PSF} \left(\frac{17}{2}\right) \right] = 33\text{ PLF}$

W

$W_{NS} = 29\text{ PSF} \left(\frac{17}{2}\right) = 247\text{ PLF}$

$W_{EW} = 27\text{ PSF} \left(\frac{17}{2} + 3.5\right) = 180\text{ PLF}$

– CONTROLS

AREA B

E

$W_{NS} = 0.08 \left[34.5\text{ PSF} (60') + 2(11\text{ PSF})\left(\frac{19}{2}\right) \right] = 183\text{ PLF}$

$W_{EW} = 0.08 \left[34.5\text{ PSF} (98') + 2(11\text{ PSF})\left(\frac{19}{2}\right) \right] = 288\text{ PLF}$ CONTROLS

W

$W_{NS} = 29\text{ PSF} \left(\frac{19}{2}\right) = 278\text{ PLF}$ CONTROLS

$W_{EW} = 27\text{ PSF} (27.5 - 24.5) = 81\text{ PLF}$

DIFFERENCE BETWEEN HIGH & LOW ROOF

AREA C

E

$W_{NS} = 0.08 \left[34.5\text{ PSF} (60') + 2(11\text{ PSF})\left(\frac{17}{2}\right) \right] = 181\text{ PLF}$

$W_{EW} = 0.08 \left[34.5\text{ PSF} (38') + 11\text{ PSF} \left(\frac{17}{2}\right) \right] = 113\text{ PLF}$

W

$W_{NS} = 26\text{ PSF} \left(\frac{17}{2}\right) = 221\text{ PLF}$

$W_{EW} = 30\text{ PSF} \left(\frac{17}{2} + 3.5\right) = 360\text{ PLF}$

– CONTROLS

ARW ENGINEERS
structural consultants

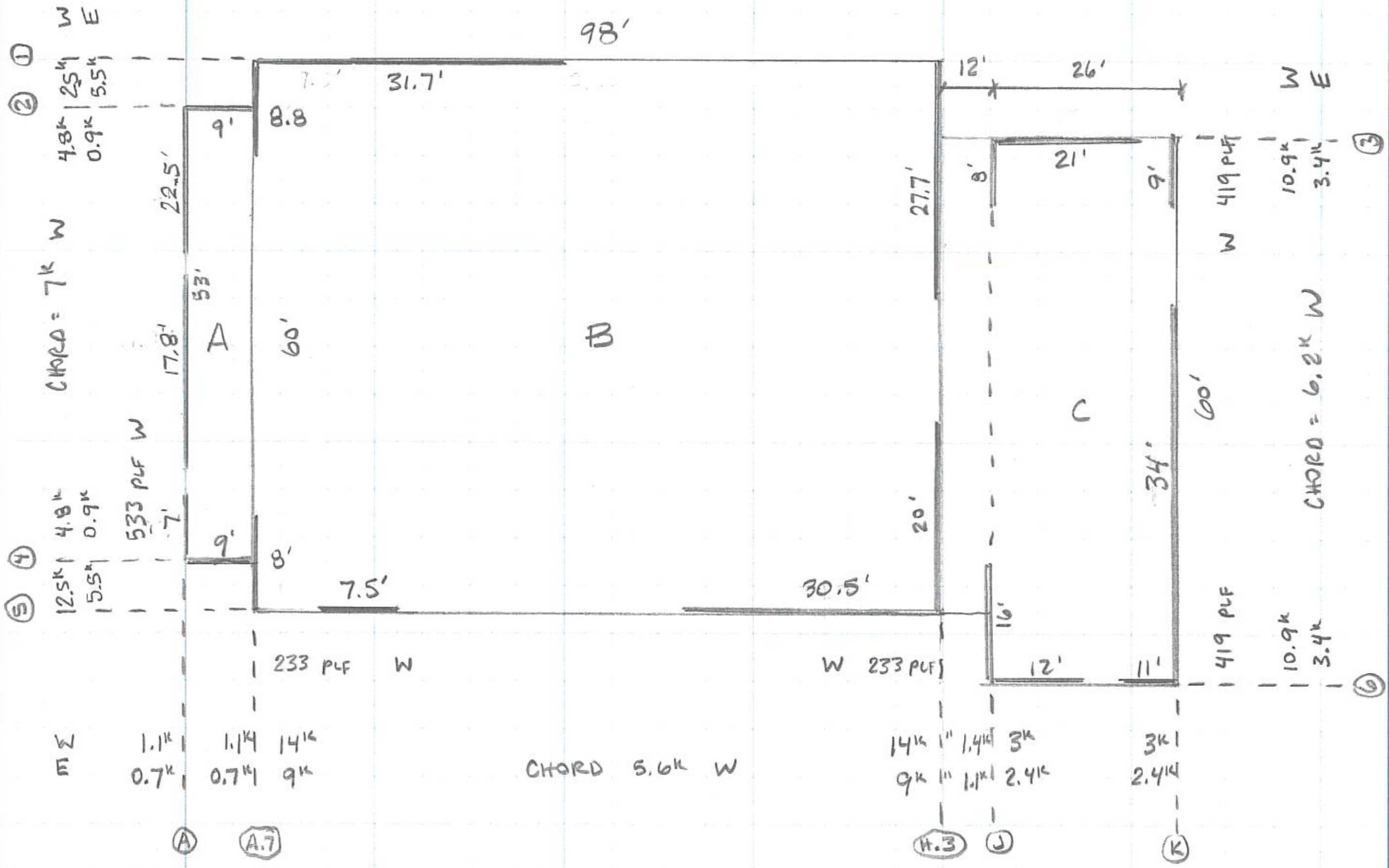
Project No. 18121 | Sheet No. B16

Project NAC RECREATION

Prepared By TMD | Date

LATERAL

↑ N



SEISMIC

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$$

$S_{DS} = 0.54$

$R = 6.5$

$I = 1.0$

$C_s = 0.08$

DIAPHRAGM

WORST CASE

AREA A

$V_{ASD} = 0.6(533 \text{ PLF}) = 320 \text{ PLF}$

UNBLOCKED 19/32" SHEATHING 10^d @ 6" OC

$V_w = \frac{800}{2} = 400 \text{ PLF}$ (WIND)

$V_e = \frac{570}{2} = 285 \text{ PLF}$ (EQ)

CHORDS

AREA A

$T_{ASD} = 0.6(7^k) = 4.2^k$

10^d NAIL $Z = 101^{\#}$ $C_d = 1.6$ $Z' = 162 \ ^{\#}/\text{NAIL}$

$\frac{4200}{162} = 26 \text{ NAILS}$

∴ USE (28) 10^d COMMON NAILS @ TOP PLATE SPLICES

2x6 DF-L No 2 $F_t = 575 \text{ PSI}$ $C_d = 1.6$ $F_t' = 920 \text{ PSI}$

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

AREA C

$T_{ASD} = 0.6(6.2) = 3.7^k$

CMSTC16 STRAP G.F 4.6^k

B18

ARW ENGINEERS | 07-Aug-18
ASD SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015 | 12:19 PM

Version: April 17, 2017 Author: Wayne Young, E.I.T. Reviewed By: Troy M. Dye, S.E.

JOB TITLE: NAC Recreation JOB #: 18121

DESCRIPTION: Grid 1 DESIGNER: TMD

INPUT :

Weight of wall =	10.0	psf		Wind (W)	Seismic (E)		
Weight of roof =	15.0	psf	Shear at wall line (V_u) =	2500	5500	lbs	Strength level
Roof Tributary length (bearing & uplift) =	1.0	ft	Shear at wall line (V_{ASD}) =	1500	3850	lbs	ASD level
Height of wall =	17.0	ft					
Wind roof uplift (W) =	27	psf					
Blocked shear wall?	YES	Shear wall capacity penalized if unblocked					
Field nailing (in)	12	in					
Stud spacing	16	in					

Shear Panel	L		d		H/L		E & W Red.	0.6W v (plf)	0.7E 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						

OUTPUT :

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

L = 31.7 ft Mr = 55.7 kips
d = 30.7 ft

	W	EQ		
Mo =	33.6	65.5	kips	
HDF =	-0.7	0.3	kips	
Rmax =	0.8	2.1	kips	**(2) 2x6 POST ..OK**

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

PAB 5

B19

ARW ENGINEERS
SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

07-Aug-18
12:19 PM

JOB TITLE: NAC Recreation
DESCRIPTION: Grid 1

JOB #: #REF!
DESIGNER: #REF!

Nominal W	Nominal E	Allowable W	Allowable E	Shear wall types
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.	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")
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 =	2.1	kips	Automatically calculates maximum compression force at end of shear wall

Compression Member Size

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

B20

ARW ENGINEERS

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 2

02-Jul-18
10:42 AM
Reviewed By: Troy M. Dye, S.E.
JOB #: 18121
DESIGNER: TMD

INPUT :

Weight of wall =	10.0	psf
Weight of roof =	15.0	psf
Roof Tributary length (bearing & uplift) =	1.0	ft
Height of wall =	14.0	ft
Wind roof uplift (W) =	27	psf
Blocked shear wall?	YES	Shear wall capacity penalized if unblocked
Field nailing (in)	12	in
Stud spacing	16	in

	Wind (W)	Seismic (E)		
Shear at wall line (V_u) =	4800	900	lbs	Strength level
Shear at wall line (V_{ASD}) =	2880	630	lbs	ASD level

Shear Panel	L		d		H/L		E & W Red.	0.6W v (plf)	0.7E 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						
#5		ft		ft						
#6		ft		ft						
#7		ft		ft						

OUTPUT :

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

L = 9.0 ft
d = 8.0 ft

Mr = 3.8 kips

	W	EQ		
Mo =	41.0	8.8	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

$\frac{8}{12}(150\text{ pcf})(2.5')(8')$ $\frac{12}{12}(150\text{ pcf})(2.5')(10')$ = $5.8^k > 4.7^k$

B21

ARW ENGINEERS

ARW ENGINEERS
SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

02-Jul-18
10:42 AM

JOB TITLE: NAC Recreation
DESCRIPTION: Grid 2

JOB #: #REF!
DESIGNER: #REF!

Shear wall types

Nominal W	Nominal E	Allowable W	Allowable 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.	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")
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 =	1		
Ct =	1		
Ci =	1		
Cb =	1.13		Eqn 3.10-2 2015 NDS
Fc' =	456	psi	
width =	3	in	(2) 2x6
depth =	5.5	in	
Max load =	7.52	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 =	1		
Cg =	1		
Cdelta =	1		
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	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

B22

ARW ENGINEERS

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 TITLE: NAC Recreation JOB #: 18121
DESCRIPTION: Grid 4 DESIGNER: TMD

02-Jul-18
10:42 AM

INPUT :

Weight of wall =	10.0	psf		Wind (W)	Seismic (E)	
Weight of roof =	15.0	psf	Shear at wall line (V_u) =	4800	900	lbs Strength level
Roof Tributary length (bearing & uplift) =	1.0	ft	Shear at wall line (V_{ASD}) =	2880	630	lbs ASD level
Height of wall =	14.0	ft				
Wind roof uplift (W) =	27	psf				
Blocked shear wall?	YES	Shear wall capacity penalized if unblocked				
Field nailing (in)	12	in				
Stud spacing	16	in				

Shear Panel	L		d		H/L		E & W Red.	0.6W v (plf)	0.7E 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						
#5		ft		ft						
#6		ft		ft						
#7		ft		ft						

OUTPUT :

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

L = 9.0 ft
d = 8.0 ft

Mr = 3.8 kips

	W	EQ	
Mo =	41.0	8.8	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" ~~SSTB~~28 anchor (foundation wall height min 26")

PAB7

B23

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 W	Nominal E	Allowable W	Allowable E	Shear wall types
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.	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")
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 =	1		
Ct =	1		
Ci =	1		
Cb =	1.13		Eqn 3.10-2 2015 NDS
Fc' =	456	psi	
width =	3	in	(2) 2x6
depth =	5.5	in	
Max load =	7.52	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 =	1		
Cg =	1		
Cdelta =	1		
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	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

B24

ARW ENGINEERS

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 5

07-Aug-18
12:26 PM
Reviewed By: Troy M. Dye, S.E.
JOB #: 18121
DESIGNER: TMD

INPUT :

Weight of wall =	10.0	psf		Wind (W)	Seismic (E)		
Weight of roof =	15.0	psf	Shear at wall line (V_u) =	2500	5500	lbs	Strength level
Roof Tributary length (bearing & uplift) =	1.0	ft	Shear at wall line (V_{ASD}) =	1500	3850	lbs	ASD level
Height of wall =	17.0	ft					
Wind roof uplift (W) =	27	psf					
Blocked shear wall?	YES	Shear wall capacity penalized if unblocked					
Field nailing (in)	12	in					
Stud spacing	16	in					

Shear Panel	L		d		H/L		E & W Red.	0.6W v (plf)	0.7E 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						
#4		ft		ft						
#5		ft		ft						
#6		ft		ft						
#7		ft		ft						

OUTPUT :

LOAD COMBINATIONS AUTOMATICALLY USED IN Mr CALCULATION
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 POST ..OK**

USE HDU2-SDS2.5 w/ (2) 2x and 5/8" ~~SSTB16~~ anchor (foundation wall height min 14")
PAB5

L =	30.5	ft	Mr = 51.6 kips		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 POST ..OK**

USE HDU2-SDS2.5 w/ (2) 2x and 5/8" ~~SSTB16~~ anchor (foundation wall height min 14")
PAB5

B25

ARW ENGINEERS

ARW ENGINEERS
SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

07-Aug-18
12:26 PM

JOB TITLE: NAC Recreation
DESCRIPTION: Grid 5

JOB #: #REF!
DESIGNER: #REF!

Nominal W	Nominal E	Allowable W	Allowable E	Shear wall types
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.	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")
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 =	1		
Ct =	1		
Ci =	1		
Cb =	1.13		Eqn 3.10-2 2015 NDS
Fc' =	456	psi	
width =	3	in	(2) 2x6
depth =	5.5	in	
Max load =	7.52	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 =	1		
Cg =	1		
Cdelta =	1		
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	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

B26

ARW ENGINEERS

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.

03-Aug-18
1:08 PM

JOB TITLE: NAC Recreation JOB #: 18121
DESCRIPTION: Grid A DESIGNER: TMD

INPUT :

Weight of wall =	10.0	psf		Wind (W)	Seismic (E)		
Weight of roof =	15.0	psf	Shear at wall line (V_u) =	1100	700	lbs	Strength level
Roof Tributary length (bearing & uplift) =	2.0	ft	Shear at wall line (V_{ASD}) =	660	490	lbs	ASD level
Height of wall =	19.0	ft					
Wind roof uplift (W) =	27	psf					
Blocked shear wall?	YES	Shear wall capacity penalized if unblocked					
Field nailing (in)	12	in					
Stud spacing	16	in					

Shear Panel	L		d		H/L		E & W Red.	0.6W v (plf)	0.7E 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						
#5		ft		ft						
#6		ft		ft						
#7		ft		ft						

OUTPUT :

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

							W	EQ		
L =	22.5	ft	Mr =	33.4	kips		W	EQ		
d =	21.5	ft				Mo =	14.2	4.4	kips	
						HDF =	-0.9	-1.3	kips	
						Rmax =	0.3	0.2	kips	**(2) 2x6 POST ..OK**

USE NO HOLDOWN REQUIRED

L =	17.8	ft	Mr =	20.9	kips		W	EQ		
d =	16.8	ft				Mo =	9.9	3.5	kips	
						HDF =	-0.7	-1.0	kips	
						Rmax =	0.3	0.2	kips	**(2) 2x6 POST ..OK**

USE NO HOLDOWN REQUIRED

L =	7.0	ft	Mr =	3.2	kips		W	EQ		
d =	6.0	ft				Mo =	2.6	1.4	kips	
						HDF =	-0.1	-0.3	kips	
						Rmax =	0.3	0.2	kips	**(2) 2x6 POST ..OK**

USE NO HOLDOWN REQUIRED

B27

ARW ENGINEERS

ARW ENGINEERS
SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

03-Aug-18
1:08 PM

JOB TITLE: NAC Recreation
DESCRIPTION: Grid A

JOB #: #REF!
DESIGNER: #REF!

Nominal W	Nominal E	Allowable W	Allowable E	Shear wall types
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.	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")
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 =	0.3	kips	Automatically calculates maximum compression force at end of shear wall

Compression Member Size

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

829

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.
02-Jul-18
10:42 AM

ENGINEERS

JOB TITLE: NAC Recreation — JOB #: 18121
DESCRIPTION: Grid A.7 — DESIGNER: TMD

INPUT :

Weight of wall =	10.0	psf		Wind (W)	Seismic (E)		
Weight of roof =	15.0	psf	Shear at wall line (V_u) =	15100	9700	lbs	Strength level
Roof Tributary length (bearing & uplift) =	2.0	ft	Shear at wall line (V_{ASD}) =	9060	6790	lbs	ASD level
Height of wall =	17.0	ft					
Wind roof uplift (W) =	27	psf					
Blocked shear wall?	YES	Shear wall capacity penalized if unblocked					
Field nailing (in)	12	in					
Stud spacing	16	in					

Shear Panel	L		d		H/L		E & W Red.	0.6W v (plf)	0.7E 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						
#4		ft		ft						
#5		ft		ft						
#6		ft		ft						
#7		ft		ft						

OUTPUT :

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

L = 8.8 ft
d = 8.0 ft
Mr = 4.6 kips

	W	EQ		
Mo =	81.9	60.5	kips	
HDF =	9.7	7.0	kips	
Rmax =	9.2	6.9	kips	**(4) 2x6 POST ..OK**

USE HDU11-SDS2.5 w/ (4) 2x and 1" ~~SB1x30~~ anchor (foundation wall height min 30")
PAB 8

L = 8.0 ft
d = 7.2 ft
Mr = 3.8 kips

	W	EQ		
Mo =	74.4	55.0	kips	
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" ~~SB1x30~~ anchor (foundation wall height min 30")
PAB 8

FTG UPLIFT

$\frac{8}{12}(150\ pcf)(2.5')(8') + \frac{24}{12}(150\ pcf)(5')(5') = 9.5^k \approx 9.7^k$ OK

USE F5 x 24" THICK

B29

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 W	Nominal E	Allowable W	Allowable E	Shear wall types
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.	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	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

ts =	1.5	in.	
Fc perpendicular =	405	psi	
Max Reaction =	9.3	kips	Automatically calculates maximum compression force at end of shear wall

Compression Member Size

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

Anchor Bolt Properties

bolt diameter =	0.625	in.	
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	**OK**

:: 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

B30

ENGINEERS

ARW ENGINEERS
ASD SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015
Version: April 17, 2017 Author: Wayne Young

03-Aug-18
1:08 PM

JOB TITLE: NAC Recreation
DESCRIPTION: Grid H.3

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

INPUT :

Weight of wall =	10.0	psf
Weight of roof =	15.0	psf
Roof Tributary length (bearing & uplift) =	2.0	ft
Height of wall =	19.0	ft
Wind roof uplift (W) =	27	psf
Blocked shear wall?	YES	Shear wall capacity penalized if unblocked
Field nailing (in)	12	in
Stud spacing	16	in

	Wind (W)	Seismic (E)		
Shear at wall line (V_u) =	15400	10100	lbs	Strength level
Shear at wall line (V_{ASD}) =	9240	7070	lbs	ASD level

Shear Panel	L		d		H/L		E & W Red.	0.6W v (plf)	0.7E v (plf)	Shear wall Type
#1	20.0	ft	19.0	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						

OUTPUT :

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

L = 20.0 ft Mr = 26.4 kips
d = 19.0 ft

	W	EQ		
Mo =	80.1	56.3	kips	
HDF =	2.8	1.6	kips	
Rmax =	3.7	2.8	kips	**(4) 2x6 POST ..OK**

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

L = 27.7 ft Mr = 50.6 kips
d = 26.7 ft

	W	EQ		
Mo =	114.4	78.0	kips	
HDF =	2.4	1.0	kips	
Rmax =	3.7	2.8	kips	**(4) 2x6 POST ..OK**

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

B31

ARW ENGINEERS

ARW ENGINEERS
SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

03-Aug-18
1:08 PM

JOB TITLE: NAC Recreation
DESCRIPTION: Grid H.3

JOB #: #REF!
DESIGNER: #REF!

Nominal W	Nominal E	Allowable W	Allowable E	Shear wall types
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.	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	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 =	1		
Ct =	1		
Ci =	1		
Cb =	1.06		Eqn 3.10-2 2015 NDS
Fc' =	430	psi	
width =	6	in	(4) 2x6
depth =	5.5	in	
Max load =	14.20	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 =	1		
Cg =	1		
Cdelta =	1		
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	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

B32

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 3

02-Jul-18
10:42 AM
Reviewed By: Troy M. Dye, S.E.
JOB #: 18121
DESIGNER: TMD

INPUT :

Weight of wall =	10.0	psf		Wind (W)	Seismic (E)		
Weight of roof =	15.0	psf	Shear at wall line (V_u) =	10900	3400	lbs	Strength level
Roof Tributary length (bearing & uplift) =	30.0	ft	Shear at wall line (V_{ASD}) =	6540	2380	lbs	ASD level
Height of wall =	14.0	ft					
Wind roof uplift (W) =	27	psf					
Blocked shear wall?	YES	Shear wall capacity penalized if unblocked					
Field nailing (in)	12	in					
Stud spacing	16	in					

Shear Panel	L		d		H/L		E & W Red.	0.6W v (plf)	0.7E 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 :

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

L = 21.0 ft
d = 20.0 ft

Mr = 78.1 kips

	W	EQ		
Mo =	198.7	33.3	kips	
HDF =	6.0	-2.2	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

B33

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!

Shear wall types

Nominal W	Nominal E	Allowable W	Allowable 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.	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")
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 =	4.4	kips	Automatically calculates maximum compression force at end of shear wall

Compression Member Size

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

B34

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

02-Jul-18
10:42 AM

ENGINEERS
JOB TITLE: NAC Recreation
DESCRIPTION: Grid 6

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

INPUT :

Weight of wall =	10.0	psf
Weight of roof =	15.0	psf
Roof Tributary length (bearing & uplift) =	30.0	ft
Height of wall =	14.0	ft
Wind roof uplift (W) =	27	psf
Blocked shear wall?	YES	Shear wall capacity penalized if unblocked
Field nailing (in)	12	in
Stud spacing	16	in

	Wind (W)	Seismic (E)		
Shear at wall line (V_u) =	10900	3400	lbs	Strength level
Shear at wall line (V_{ASD}) =	6540	2380	lbs	ASD level

Shear Panel	L		d		H/L		E & W Red.	0.6W v (plf)	0.7E 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						

OUTPUT :

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

L = 12.0 ft
d = 11.0 ft
Mr = 25.5 kips

	W	EQ		
Mo =	82.8	17.4	kips	
HDF =	5.2	-0.7	kips	
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

L = 11.0 ft
d = 10.0 ft
Mr = 21.4 kips

	W	EQ		
Mo =	73.2	15.9	kips	
HDF =	5.2	-0.5	kips	
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

JOB TITLE: NAC Recreation
DESCRIPTION: Grid 6

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.	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")
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	kips	Automatically calculates maximum compression force at end of shear wall

Compression Member Size

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

836

ARW ENGINEERS
ASD SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015
Version: April 17, 2017 Author: Wayne Young

02-Jul-18
10:42 AM

Reviewed By: Troy M. Dye

ENGINEERS

JOB TITLE: NAC Recreation
DESCRIPTION: Grid J

JOB #: 18121
DESIGNER: TMD

INPUT :

Weight of wall =	10.0	psf		Wind (W)	Seismic (E)		
Weight of roof =	15.0	psf	Shear at wall line (V_u) =	3400	3500	lbs	Strength level
Roof Tributary length (bearing & uplift) =	2.0	ft	Shear at wall line (V_{ASD}) =	2040	2450	lbs	ASD level
Height of wall =	16.0	ft					
Wind roof uplift (W) =	27	psf					
Blocked shear wall?	YES	Shear wall capacity penalized if unblocked					
Field nailing (in)	12	in					
Stud spacing	16	in					

Shear Panel	L		d		H/L		E & W Red.	0.6W v (plf)	0.7E 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 :

LOAD COMBINATIONS AUTOMATICALLY USED IN Mr CALCULATION
0.6D + 0.7E 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	kips	**(2) 2x6 POST ..OK**

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

PAB5

L = 8.0 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	kips	**(2) 2x6 POST ..OK**

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

PAB5

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 W	Nominal E	Allowable W	Allowable E	Shear wall types
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.	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	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 =	1.6	kips	Automatically calculates maximum compression force at end of shear wall

Compression Member Size

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

B38

ARW ENGINEERS

ASD SEGMENTED SHEAR WALL CALCULATIONS PER IBC 2015

Version: April 17, 2017 Author: Wayne Young

03-Aug-18
1:10 PM

ENGINEERS

JOB TITLE: NAC Recreation

DESCRIPTION: Grid K

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

INPUT :

Weight of wall =	10.0	psf		Wind (W)	Seismic (E)	
Weight of roof =	15.0	psf	Shear at wall line (V_u) =	3000	2400	lbs Strength level
Roof Tributary length (bearing & uplift) =	2.0	ft	Shear at wall line (V_{ASD}) =	1800	1680	lbs ASD level
Height of wall =	16.0	ft				
Wind roof uplift (W) =	27	psf				
Blocked shear wall?	YES	Shear wall capacity penalized if unblocked				
Field nailing (in)	12	in				
Stud spacing	16	in				

Shear Panel	L		d		H/L		E & W Red.	0.6W v (plf)	0.7E v (plf)	Shear wall Type
#1	34.0	ft	33.0	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						

OUTPUT :

LOAD COMBINATIONS AUTOMATICALLY USED IN Mr CALCULATION

0.6D + 0.7E 0.6D + 0.6W

L = 34.0 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 POST ..OK**

USE NO HOLDOWN REQUIRED

L = 9.0 ft	Mr = 4.6 kips		W	EQ		
d = 8.0 ft		Mo =	7.3	5.6	kips	
		HDF =	0.3	0.1	kips	
		Rmax =	0.7	0.6	kips	**(2) 2x6 POST ..OK**

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

03-Aug-18
1:10 PM

ENGINEERS

JOB TITLE: NAC Recreation
DESCRIPTION: Grid K

JOB #: #REF!
DESIGNER: #REF!

Shear wall types

Nominal W	Nominal E	Allowable W	Allowable 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
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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.	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	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 =	0.7	kips	Automatically calculates maximum compression force at end of shear wall

Compression Member Size

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

WALLS

ARW ENGINEERS

WALLS

WALL A: DEAD LOAD

			Comments
Framing:	5	psf	
Batting/Blown Insulation:	2	psf	
Sheathing:	2	psf	
Veneer:		psf	
Mechanical Ducts/Misc.:		psf	
Gypsum Board:	2	psf	
Collateral:		psf	
Total Dead Load:	**11**	psf	
Seismic Mass Dead Load:	**11**	psf	

WALL B: DEAD LOAD

			Comments
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

			Comments
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 D: DEAD LOAD

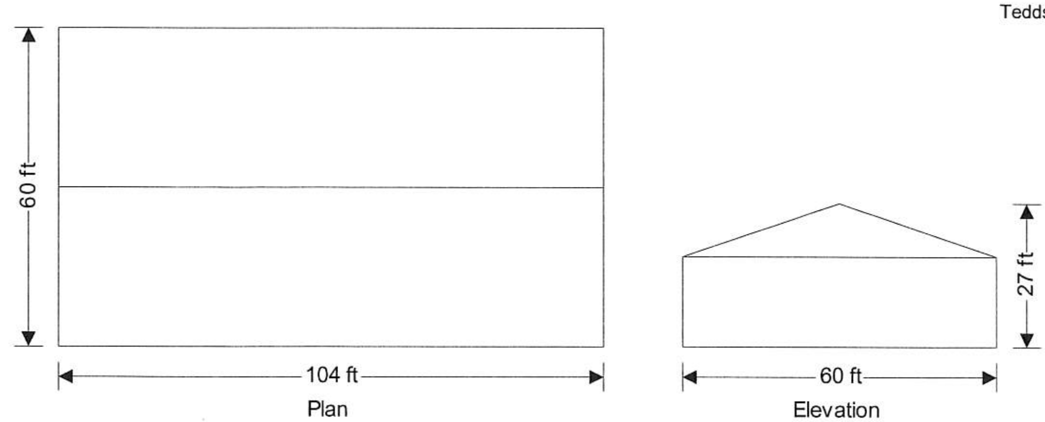
			Comments
Framing:		psf	
Batting/Blown Insulation:		psf	
Sheathing:		psf	
Veneer:		psf	
Mechanical Ducts/Misc.:		psf	
Gypsum Board:		psf	
Collateral:		psf	
Total Dead Load:	**0**	psf	
Seismic Mass Dead Load:	**0**	psf	

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	$b = \mathbf{104.00}$ ft
Width of building	$d = \mathbf{60.00}$ ft
Height to eaves	$H = \mathbf{17.00}$ ft
Pitch of roof	$\alpha_0 = \mathbf{18.5}$ deg
Mean height	$h = \mathbf{22.02}$ ft

General wind load requirements

Basic wind speed	$V = \mathbf{115.0}$ mph
Risk category	II
Velocity pressure exponent coeff (Table 26.6-1)	$K_d = \mathbf{0.85}$
Exposure category (cl.26.7.3)	C
Enclosure classification (cl.26.10)	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1)	$GC_{pi\_p} = \mathbf{0.18}$
Internal pressure coef –ve (Table 26.11-1)	$GC_{pi\_n} = \mathbf{-0.18}$
Gust effect factor	$G_f = \mathbf{0.85}$

Topography

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

Velocity pressure

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

Peak velocity pressure for internal pressure

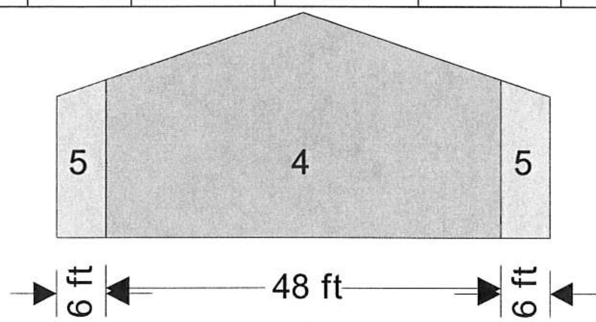
Peak velocity pressure – internal (as roof press.)	$q_i = \mathbf{26.36}$ psf

Equations used in tables

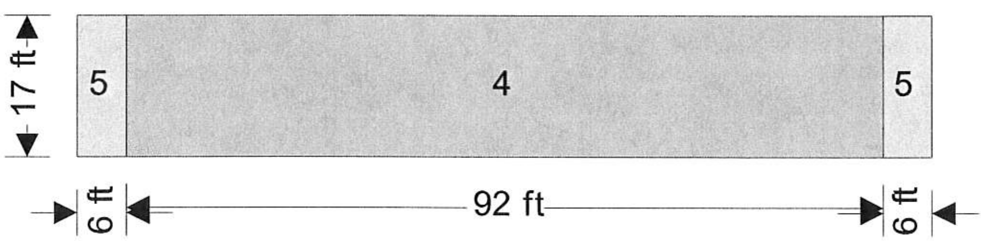
Net pressure	$p = q_h \times [GC_p - GC_{pi}]$

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

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	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



Elevation of gable wall



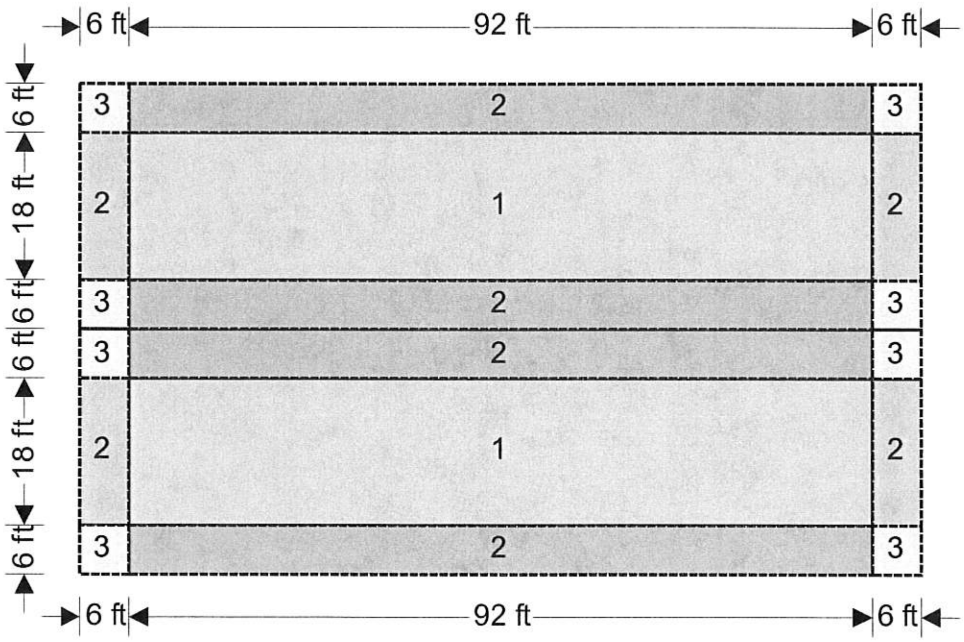
Elevation of side wall

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

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	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 (ft^2)	$+GC_p$	$-GC_p$	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



Plan on roof

C5

ARW ENGINEERS

Wood Stud Design Based on IBC 2015 & 2015 NDS

Version Date: May 25, 2017

9-Aug-18
9:05 AM

JOB TITLE: NAC Rec Center
WALL LOCATION: Grid 1 & 5

JOB #: 18121
ENGINEER: TMD

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			WALL WEIGHTS		
UNIFORM SNOW:	194	plf	SELF WEIGHT:	10	psf
UNIFORM LIVE:	0	plf	FINISHES WEIGHT:	5	psf
UNIFORM DEAD:	30	plf	S_{ds}:	0.541	g
DESIGN ROOF SNOW LOAD:	97	psf	I_e:	1	
BUILDING ELEVATION:	5500	ft.	**APPLIED LATERAL LOADS**		
ALLOWABLE SOIL BEARING:	1500	psf	WIND (W):	30	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

REQUIRED FOOTING WIDTH: 2.0 ft (Footing sized for bearing only)

NOMINAL STUD SIZE: 1.75x5.5

	b(in.)		d(in.)			
STUD SIZE (actual)	1.75	X	5.5	SPACED AT	16	inches o.c.
STUD LENGTH	17 ft.			ECCENTRICITY	0	inches (at top of wall)

MATERIAL PROPERTIES

WALL STUDS			BOTTOM PLATE		
Material:	LVL 2.0E		Material:	LVL 2.0E	
F_b	2891	psi	f_c =	49	psi
F_c	2510	psi	F_{cperp}' =	750	psi **O.K.**
E_{min}	1,016,535	psi	E =	2,000,000	psi

ANALYSIS

bending CF: 1 (2x4-1.5, 2x6-1.3, 2x8-1.2, 4x4-1.5) verify with table 4A - Not for engineered lumber
comp. CF: 1 (2x4-1.15, 2x6-1.1, 2x8-1.05, 4x4-1.15) verify with table 4A - Not for engineered lumber

Bending effective length					Compression effective length				
unsupported length, l_{u1}	204	l_{u1}/d_1 =	37.09	in.	unsupported length, l_{u1}	204	k_{e1} =	1.00	
unsupported length, l_{u2}	12	l_{u2}/d_2 =	6.86	in.	unsupported length, l_{u2}	12	k_{e2} =	1.00	
unbraced length, l_{e1}	349.02				unbraced length, l_{e1}	204.0	d_1 =	5.5	in.
unbraced length, l_{e2}	24.72				unbraced length, l_{e2}	12.0	d_2 =	1.75	in.
bending C_f	1.15	(Gravity Load Combs.)			l_{e1}/d_1 =	37.09			
Wind bending C_f	1.35	(Only for Combs. Including Wind)			l_{e2}/d_2 =	6.86			

Load Combination #1 D + L (formula 16-9)

C_f =	1.15	F_{bE} =	1946	f_b =	0.0 psi
C_D =	0.9	C_L =	1.000	F_c^* =	2259 psi
F_b^* =	2992 psi	F_b' =	2992 psi	F_{cE} =	607 psi
R_B =	25.0	S_x =	8.8 in³	K_f =	
f_c =	22 psi	F_c' =	569 psi	C_p =	0.252
Load at Base=	285 plf	C.S.R =	0.001 **O.K.**		

Load Combination #2 D + S (formula 16-10)

C_f =	1.15	F_{bE} =	1946	f_b =	0.0 psi
C_D =	1	C_L =	1.000	F_c^* =	2510 psi
F_b^* =	3325 psi	F_b' =	3325 psi	F_{cE} =	607 psi
R_B =	25.0	S_x =	8.8 in³	K_f =	
f_c =	49 psi	F_c' =	573 psi	C_p =	0.228
Load at Base=	479 plf	C.S.R =	0.007 **O.K.**		

Load Combination #3 D + 0.75L + 0.75S (formula 16-11)

C_f =	1.15	F_{bE} =	1946	f_b =	0.0 psi
C_D =	1	C_L =	1.000	F_c^* =	2510 psi
F_b^* =	3325 psi	F_b' =	3325 psi	F_{cE} =	607 psi
R_B =	25.0	S_x =	8.8 in³	K_f =	
f_c =	42 psi	F_c' =	573 psi	C_p =	0.228 in
Load at Base=	431 plf	C.S.R =	0.005 **O.K.**		

Load Combination #4 D + 0.6W (formula 16-12)

C_f =	1.35	F_{bE} =	1946	f_b =	1179.2 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	6245 psi	F_b' =	6245 psi	F_{cE} =	607 psi
R_B =	25.0	S_x =	8.8 in³	K_f =	
f_c =	22 psi	F_c' =	587 psi	C_p =	0.146 in
Load at Base=	30 plf	C.S.R =	0.197 **O.K.**		

Load Combination #5 D + 0.7E (formula 16-12)

C_f =	1.15	F_{bE} =	1946	f_b =	148.9 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	5319 psi	F_b' =	5319 psi	F_{cE} =	607 psi
R_B =	25.0	S_x =	8.8 in³	K_f =	
f_c =	22 psi	F_c' =	587 psi	C_p =	0.146 in
Load at Base=	30 plf	C.S.R =	0.030 **O.K.**		

Load Combination #6 D + 0.75L + 0.75S + 0.45W (formula 16-13)

C_f =	1.35	F_{bE} =	1946	f_b =	884.4 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	6245 psi	F_b' =	6245 psi	F_{cE} =	607 psi
R_B =	25.0	S_x =	8.8 in³	K_f =	
f_c =	42 psi	F_c' =	587 psi	C_p =	0.146 in
Load at Base=	431 plf	C.S.R =	0.157 **O.K.**		

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_f =	1.15	F_{bE} =	1946	f_b =	111.6 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	5319 psi	F_b' =	5319 psi	F_{cE} =	607 psi
R_B =	25.0	S_x =	8.8 in³	K_f =	
f_c =	28 psi	F_c' =	587 psi	C_p =	0.146 in
Load at Base=	285 plf	C.S.R =	0.024 **O.K.**		

based on NDS equation 15.4-1:

C6

ARW ENGINEERS

Wood Stud Design Based on IBC 2015 & 2015 NDS

Version Date: May 25, 2017

9-Aug-18
9:05 AM

JOB TITLE: NAC Rec Center
WALL LOCATION: Grid B & H

JOB #: 18121
ENGINEER: TMD

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:	970	plf
UNIFORM LIVE:	0	plf
UNIFORM DEAD:	150	plf
DESIGN ROOF SNOW LOAD:	97	psf
BUILDING ELEVATION:	5500	ft.
ALLOWABLE SOIL BEARING:	1500	psf

WALL WEIGHTS

SELF WEIGHT:	10	psf
FINISHES WEIGHT:	5	psf
S_{ds}:	0.541	g
I_e:	1	

APPLIED LATERAL LOADS

WIND (W):	30	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

REQUIRED FOOTING WIDTH: 2.0 ft (Footing sized for bearing only)

NOMINAL STUD SIZE: 1.75x5.5

STUD SIZE (actual): b(in.) 1.75 X d(in.) 5.5

STUD LENGTH: 21 ft.

SPACED AT: 16 inches o.c.

ECCENTRICITY: 0 inches (at top of wall)

MATERIAL PROPERTIES

WALL STUDS			BOTTOM PLATE		
Material:	LVL 2.0E		Material:	LVL 2.0E	
F_b	2891	psi	f_c =	177	psi
F_c	2510	psi	F_{cperp}' =	750	psi O.K.
E_{min}	1,016,535	psi	E =	2,000,000	psi

ANALYSIS

bending CF: 1 (2x4-1.5, 2x6-1.3, 2x8-1.2, 4x4-1.5) verify with table 4A - Not for engineered lumber

comp. CF: 1 (2x4-1.15, 2x6-1.1, 2x8-1.05, 4x4-1.15) verify with table 4A - Not for engineered lumber

Bending effective length					Compression effective length				
unsupported length, l_{u1}	252	l_{u1}/d_1 =	45.82	in.	unsupported length, l_{u1}	252	k_{e1} =	1.00	
unsupported length, l_{u2}	12	l_{u2}/d_2 =	6.86	in.	unsupported length, l_{u2}	12	k_{e2} =	1.00	
unbraced length, l_{e1}	427.26				unbraced length, l_{e1}	252.0	d_1 =	5.5	in.
unbraced length, l_{e2}	24.72				unbraced length, l_{e2}	12.0	d_2 =	1.75	in.
bending C_r	1.15	(Gravity Load Combs.)			l_{e1}/d_1 =	45.82			
Wind bending C_r	1.35	(Only for Combs. Including Wind)			l_{e2}/d_2 =	6.86			

Load Combination #1 D + L (formula 16-9)

C_r =	1.15	F_{bE} =	1590		f_b =	0.0	psi
C_D =	0.9	C_L =	1.000		F_c^* =	2259	psi
F_b^* =	2992 psi	F_b' =	2992	psi	F_{cE} =	398	psi
R_B =	27.7	S_x =	8.8	in^3	K_f =		
f_c =	43 psi	F_c' =	382	psi	C_p =	0.169	
Load at Base=	465 plf	C.S.R =	0.012	O.K.			

Load Combination #2 D + S (formula 16-10)

C_r =	1.15	F_{bE} =	1590		f_b =	0.0	psi
C_D =	1	C_L =	1.000		F_c^* =	2510	psi
F_b^* =	3325 psi	F_b' =	3325	psi	F_{cE} =	398	psi
R_B =	27.7	S_x =	8.8	in^3	K_f =		
f_c =	177 psi	F_c' =	384	psi	C_p =	0.153	
Load at Base=	1435 plf	C.S.R =	0.212	O.K.			

Load Combination #3 D + 0.75L + 0.75S (formula 16-11)

C_r =	1.15	F_{bE} =	1590		f_b =	0.0	psi
C_D =	1	C_L =	1.000		F_c^* =	2510	psi
F_b^* =	3325 psi	F_b' =	3325	psi	F_{cE} =	398	psi
R_B =	27.7	S_x =	8.8	in^3	K_f =		
f_c =	143 psi	F_c' =	384	psi	C_p =	0.153	in
Load at Base=	1193 plf	C.S.R =	0.139	O.K.			

Load Combination #4 D + 0.6W (formula 16-12)

C_r =	1.35	F_{bE} =	1590		f_b =	1799.4	psi
C_D =	1.6	C_L =	1		F_c^* =	4016	psi
F_b^* =	6245 psi	F_b' =	6245	psi	F_{cE} =	398	psi
R_B =	27.7	S_x =	8.8	in^3	K_f =		
f_c =	43 psi	F_c' =	390	psi	C_p =	0.097	in
Load at Base=	150 plf	C.S.R =	0.335	O.K.			

Load Combination #5 D + 0.7E (formula 16-12)

C_r =	1.15	F_{bE} =	1590		f_b =	227.1	psi
C_D =	1.6	C_L =	1		F_c^* =	4016	psi
F_b^* =	5319 psi	F_b' =	5319	psi	F_{cE} =	398	psi
R_B =	27.7	S_x =	8.8	in^3	K_f =		
f_c =	43 psi	F_c' =	390	psi	C_p =	0.097	in
Load at Base=	150 plf	C.S.R =	0.060	O.K.			

Load Combination #6 D + 0.75L + 0.75S + 0.45W (formula 16-13)

C_r =	1.35	F_{bE} =	1590		f_b =	1349.6	psi
C_D =	1.6	C_L =	1		F_c^* =	4016	psi
F_b^* =	6245 psi	F_b' =	6245	psi	F_{cE} =	398	psi
R_B =	27.7	S_x =	8.8	in^3	K_f =		
f_c =	143 psi	F_c' =	390	psi	C_p =	0.097	in
Load at Base=	1193 plf	C.S.R =	0.473	O.K.			

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_r =	1.15	F_{bE} =	1590		f_b =	170.4	psi
C_D =	1.6	C_L =	1		F_c^* =	4016	psi
F_b^* =	5319 psi	F_b' =	5319	psi	F_{cE} =	398	psi
R_B =	27.7	S_x =	8.8	in^3	K_f =		
f_c =	71 psi	F_c' =	390	psi	C_p =	0.097	in
Load at Base=	465 plf	C.S.R =	0.072	O.K.			

based on NDS equation 15.4-1:

C7

ENGINEERS

9-Aug-18

10:12 AM

Wood Stud Design Based on IBC 2015 & 2015 NDS

Version Date: May 25, 2017

JOB TITLE: NAC Rec Center
WALL LOCATION: Grid 2 & 4

JOB #: 18121
ENGINEER: TMD

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			WALL WEIGHTS		
UNIFORM SNOW:	2619	plf	SELF WEIGHT:	10	psf
UNIFORM LIVE:	0	plf	FINISHES WEIGHT:	5	psf
UNIFORM DEAD:	405	plf	S_{ds}:	0.541	g
DESIGN ROOF SNOW LOAD:	97	psf	I_e:	1	
BUILDING ELEVATION:	5500	ft.	**APPLIED LATERAL LOADS**		
ALLOWABLE SOIL BEARING:	1500	psf	WIND (W):	30	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

REQUIRED FOOTING WIDTH: 2.5 ft (Footing sized for bearing only)

NOMINAL STUD SIZE: 1.75x5.5

	b(in.)		d(in.)			
STUD SIZE (actual)	1.75	X	5.5	SPACED AT	16	inches o.c.
STUD LENGTH	11.5	ft.		ECCENTRICITY	0	inches (at top of wall)

MATERIAL PROPERTIES

WALL STUDS			BOTTOM PLATE			
Material:	LVL 2.0E		Material:	LVL 2.0E		
F_b	2891	psi	f_c =	431	psi	
F_c	2510	psi	F_{cperp}' =	750	psi	O.K.
E_{min}	1,016,535	psi	E =	2,000,000	psi	

ANALYSIS

bending CF	1	(2x4-1.5, 2x6-1.3, 2x8-1.2, 4x4-1.5) verify with table 4A - Not for engineered lumber
comp. CF	1	(2x4-1.15, 2x6-1.1, 2x8-1.05, 4x4-1.15) verify with table 4A - Not for engineered lumber

Bending effective length					Compression effective length				
unsupported length, l_{u1}	138	l_{u1}/d_1 =	25.09	in.	unsupported length, l_{u1}	138	k_{e1} =	1.00	
unsupported length, l_{u2}	12	l_{u2}/d_2 =	6.86	in.	unsupported length, l_{u2}	12	k_{e2} =	1.00	
unbraced length, l_{e1}	241.44				unbraced length, l_{e1}	138.0	d_1 =	5.5	in.
unbraced length, l_{e2}	24.72				unbraced length, l_{e2}	12.0	d_2 =	1.75	in.
bending C_r	1.15	(Gravity Load Combs.)			l_{e1}/d_1 =	25.09			
Wind bending C_r	1.35	(Only for Combs. Including Wind)			l_{e2}/d_2 =	6.86			

Load Combination #1 D + L (formula 16-9)

C_r =	1.15	F_{bE} =	2813	f_b =	0.0 psi
C_D =	0.9	C_L =	1.000	F_c^* =	2259 psi
F_b^* =	2992 psi	F_b' =	2992 psi	F_{cE} =	1327 psi
R_B =	20.8	S_x =	8.8 in^3	K_f =	
f_c =	68 psi	F_c' =	1112 psi	C_p =	0.492
Load at Base=	578 plf	C.S.R =	0.004 O.K.		

Load Combination #2 D + S (formula 16-10)

C_r =	1.15	F_{bE} =	2813	f_b =	0.0 psi
C_D =	1	C_L =	1.000	F_c^* =	2510 psi
F_b^* =	3325 psi	F_b' =	3325 psi	F_{cE} =	1327 psi
R_B =	20.8	S_x =	8.8 in^3	K_f =	
f_c =	431 psi	F_c' =	1138 psi	C_p =	0.454
Load at Base=	3197 plf	C.S.R =	0.143 O.K.		

Load Combination #3 D + 0.75L + 0.75S (formula 16-11)

C_r =	1.15	F_{bE} =	2813	f_b =	0.0 psi
C_D =	1	C_L =	1.000	F_c^* =	2510 psi
F_b^* =	3325 psi	F_b' =	3325 psi	F_{cE} =	1327 psi
R_B =	20.8	S_x =	8.8 in^3	K_f =	
f_c =	340 psi	F_c' =	1138 psi	C_p =	0.454 in
Load at Base=	2542 plf	C.S.R =	0.089 O.K.		

Load Combination #4 D + 0.6W (formula 16-12)

C_r =	1.35	F_{bE} =	2813	f_b =	539.6 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	6245 psi	F_b' =	6245 psi	F_{cE} =	1327 psi
R_B =	20.8	S_x =	8.8 in^3	K_f =	
f_c =	68 psi	F_c' =	1221 psi	C_p =	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_r =	1.15	F_{bE} =	2813	f_b =	68.1 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	5319 psi	F_b' =	5319 psi	F_{cE} =	1327 psi
R_B =	20.8	S_x =	8.8 in^3	K_f =	
f_c =	68 psi	F_c' =	1221 psi	C_p =	0.304 in
Load at Base=	405 plf	C.S.R =	0.017 O.K.		

Load Combination #6 D + 0.75L + 0.75S + 0.45W (formula 16-13)

C_r =	1.35	F_{bE} =	2813	f_b =	404.7 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	6245 psi	F_b' =	6245 psi	F_{cE} =	1327 psi
R_B =	20.8	S_x =	8.8 in^3	K_f =	
f_c =	340 psi	F_c' =	1221 psi	C_p =	0.304 in
Load at Base=	2542 plf	C.S.R =	0.165 O.K.		

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_r =	1.15	F_{bE} =	2813	f_b =	51.1 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	5319 psi	F_b' =	5319 psi	F_{cE} =	1327 psi
R_B =	20.8	S_x =	8.8 in^3	K_f =	
f_c =	145 psi	F_c' =	1221 psi	C_p =	0.304 in
Load at Base=	578 plf	C.S.R =	0.025 O.K.		

based on NDS equation 15.4-1:

C9

ARW ENGINEERS

9-Aug-18
9:05 AM

Wood Stud Design Based on IBC 2015 & 2015 NDS

Version Date: May 25, 2017

JOB TITLE: NAC Rec Center
WALL LOCATION: Grid J & K

JOB #: 18121
ENGINEER: TMD

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:	388	plf
UNIFORM LIVE:	0	plf
UNIFORM DEAD:	60	plf
DESIGN ROOF SNOW LOAD:	97	psf
BUILDING ELEVATION:	5500	ft.
ALLOWABLE SOIL BEARING:	1500	psf

WALL WEIGHTS

SELF WEIGHT:	10	psf
FINISHES WEIGHT:	5	psf
S_{ds}:	0.541	g
I_e:	1	

APPLIED LATERAL LOADS

WIND (W):	30	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

REQUIRED FOOTING WIDTH: 2.0 ft (Footing sized for bearing only)

NOMINAL STUD SIZE: 1.75x5.5

STUD SIZE (actual): b(in.) 1.75 X d(in.) 5.5

SPACED AT: 16 inches o.c.

STUD LENGTH: 23 ft.

ECCENTRICITY: 0 inches (at top of wall)

MATERIAL PROPERTIES

WALL STUDS

Material: LVL 2.0E

F_b 2891 psi

F_c 2510 psi

E_{min} 1,016,535 psi

BOTTOM PLATE

Material: LVL 2.0E

f_c = 86 psi

F_{cperp}' = 750 psi **O.K.**

E = 2,000,000 psi

ANALYSIS

bending CF: 1 (2x4-1.5, 2x6-1.3, 2x8-1.2, 4x4-1.5) verify with table 4A - Not for engineered lumber

comp. CF: 1 (2x4-1.15, 2x6-1.1, 2x8-1.05, 4x4-1.15) verify with table 4A - Not for engineered lumber

Bending effective length				Compression effective length				
unsupported length, l_{u1}	276	l_{u1}/d_1 =	50.18 in.	unsupported length, l_{u1}	276	k_{e1} =	1.00	
unsupported length, l_{u2}	12	l_{u2}/d_2 =	6.86 in.	unsupported length, l_{u2}	12	k_{e2} =	1.00	
unbraced length, l_{e1}	466.38			unbraced length, l_{e1}	276.0	d_1 =	5.5	in.
unbraced length, l_{e2}	24.72			unbraced length, l_{e2}	12.0	d_2 =	1.75	in.
bending C_r	1.15	(Gravity Load Combs.)		l_{e1}/d_1 =	50.18			
Wind bending C_r	1.35	(Only for Combs. Including Wind)		l_{e2}/d_2 =	6.86			

Load Combination #1 D + L (formula 16-9)

C_r =	1.15	F_{bE} =	1456	f_b =	0.0 psi
C_D =	0.9	C_L =	1.000	F_c^* =	2259 psi
F_b^* =	2992 psi	F_b' =	2992 psi	F_{cE} =	332 psi
R_B =	28.9	S_x =	8.8 in^3	K_f =	
f_c =	32 psi	F_c' =	321 psi	C_p =	0.142
Load at Base=	405 plf	**C.S.R =**	**0.010 O.K.**		

Load Combination #2 D + S (formula 16-10)

C_r =	1.15	F_{bE} =	1456	f_b =	0.0 psi
C_D =	1	C_L =	1.000	F_c^* =	2510 psi
F_b^* =	3325 psi	F_b' =	3325 psi	F_{cE} =	332 psi
R_B =	28.9	S_x =	8.8 in^3	K_f =	
f_c =	86 psi	F_c' =	322 psi	C_p =	0.128
Load at Base=	793 plf	**C.S.R =**	**0.071 O.K.**		

Load Combination #3 D + 0.75L + 0.75S (formula 16-11)

C_r =	1.15	F_{bE} =	1456	f_b =	0.0 psi
C_D =	1	C_L =	1.000	F_c^* =	2510 psi
F_b^* =	3325 psi	F_b' =	3325 psi	F_{cE} =	332 psi
R_B =	28.9	S_x =	8.8 in^3	K_f =	
f_c =	73 psi	F_c' =	322 psi	C_p =	0.128 in
Load at Base=	696 plf	**C.S.R =**	**0.051 O.K.**		

Load Combination #4 D + 0.6W (formula 16-12)

C_r =	1.35	F_{bE} =	1456	f_b =	2158.5 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	6245 psi	F_b' =	6245 psi	F_{cE} =	332 psi
R_B =	28.9	S_x =	8.8 in^3	K_f =	
f_c =	32 psi	F_c' =	326 psi	C_p =	0.081 in
Load at Base=	60 plf	**C.S.R =**	**0.393 O.K.**		

Load Combination #5 D + 0.7E (formula 16-12)

C_r =	1.15	F_{bE} =	1456	f_b =	272.5 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	5319 psi	F_b' =	5319 psi	F_{cE} =	332 psi
R_B =	28.9	S_x =	8.8 in^3	K_f =	
f_c =	32 psi	F_c' =	326 psi	C_p =	0.081 in
Load at Base=	60 plf	**C.S.R =**	**0.066 O.K.**		

Load Combination #6 D + 0.75L + 0.75S + 0.45W (formula 16-13)

C_r =	1.35	F_{bE} =	1456	f_b =	1618.9 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	6245 psi	F_b' =	6245 psi	F_{cE} =	332 psi
R_B =	28.9	S_x =	8.8 in^3	K_f =	
f_c =	73 psi	F_c' =	326 psi	C_p =	0.081 in
Load at Base=	696 plf	**C.S.R =**	**0.381 O.K.**		

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_r =	1.15	F_{bE} =	1456	f_b =	204.4 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	5319 psi	F_b' =	5319 psi	F_{cE} =	332 psi
R_B =	28.9	S_x =	8.8 in^3	K_f =	
f_c =	44 psi	F_c' =	326 psi	C_p =	0.081 in
Load at Base=	405 plf	**C.S.R =**	**0.062 O.K.**		

based on NDS equation 15.4-1:

C10

ARW ENGINEERS

Wood Stud Design Based on IBC 2015 & 2015 NDS

Version Date: May 25, 2017

9-Aug-18
9:05 AM

JOB TITLE: NAC Rec Center
WALL LOCATION: Grid A

JOB #: 18121
ENGINEER: TMD

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:	388	plf
UNIFORM LIVE:	0	plf
UNIFORM DEAD:	60	plf
DESIGN ROOF SNOW LOAD:	97	psf
BUILDING ELEVATION:	5500	ft.
ALLOWABLE SOIL BEARING:	1500	psf

WALL WEIGHTS

SELF WEIGHT:	10	psf
FINISHES WEIGHT:	5	psf
S_{ds}:	0.541	g
I_e:	1	

APPLIED LATERAL LOADS

WIND (W):	30	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: 1.75x5.5

STUD SIZE (actual): b(in.) 1.75 X d(in.) 5.5

STUD LENGTH: 20 ft.

REQUIRED FOOTING WIDTH: **2.0** ft (Footing sized for bearing only)

SPACED AT: 16 inches o.c.

ECCENTRICITY: 0 inches (at top of wall)

MATERIAL PROPERTIES

WALL STUDS		BOTTOM PLATE	
Material:	LVL 2.0E	Material:	LVL 2.0E
F_b	2891 psi	f_c =	83 psi
F_c	2510 psi	F_{cperp}' =	750 psi **O.K.**
E_{min}	1,016,535 psi	E =	2,000,000 psi

ANALYSIS

bending CF: 1 (2x4-1.5, 2x6-1.3, 2x8-1.2, 4x4-1.5) verify with table 4A - Not for engineered lumber

comp. CF: 1 (2x4-1.15, 2x6-1.1, 2x8-1.05, 4x4-1.15) verify with table 4A - Not for engineered lumber

Bending effective length				Compression effective length			
unsupported length, l_{u1}	240	l_{u1}/d_1 = 43.64	in.	unsupported length, l_{u1}	240	k_{e1} =	1.00
unsupported length, l_{u2}	12	l_{u2}/d_2 = 6.86	in.	unsupported length, l_{u2}	12	k_{e2} =	1.00
unbraced length, l_{e1}	407.70			unbraced length, l_{e1}	240.0	d_1 =	5.5 in.
unbraced length, l_{e2}	24.72			unbraced length, l_{e2}	12.0	d_2 =	1.75 in.
bending C_r	1.15	(Gravity Load Combs.)		l_{e1}/d_1 =	43.64		
Wind bending C_r	1.35	(Only for Combs. Including Wind)		l_{e2}/d_2 =	6.86		

Load Combination #1 D + L (formula 16-9)

C_r =	1.15	F_{bE} =	1666	f_b =	0.0 psi
C_D =	0.9	C_L =	1.000	F_c^* =	2259 psi
F_b^* =	2992 psi	F_b' =	2992 psi	F_{cE} =	439 psi
R_B =	27.1	S_x =	8.8 in^3	K_f =	
f_c =	29 psi	F_c' =	420 psi	C_p =	0.186
Load at Base=	360 plf	**C.S.R =**	**0.005 O.K.**		

Load Combination #2 D + S (formula 16-10)

C_r =	1.15	F_{bE} =	1666	f_b =	0.0 psi
C_D =	1	C_L =	1.000	F_c^* =	2510 psi
F_b^* =	3325 psi	F_b' =	3325 psi	F_{cE} =	439 psi
R_B =	27.1	S_x =	8.8 in^3	K_f =	
f_c =	83 psi	F_c' =	422 psi	C_p =	0.168
Load at Base=	748 plf	**C.S.R =**	**0.039 O.K.**		

Load Combination #3 D + 0.75L + 0.75S (formula 16-11)

C_r =	1.15	F_{bE} =	1666	f_b =	0.0 psi
C_D =	1	C_L =	1.000	F_c^* =	2510 psi
F_b^* =	3325 psi	F_b' =	3325 psi	F_{cE} =	439 psi
R_B =	27.1	S_x =	8.8 in^3	K_f =	
f_c =	69 psi	F_c' =	422 psi	C_p =	0.168 in
Load at Base=	651 plf	**C.S.R =**	**0.027 O.K.**		

Load Combination #4 D + 0.6W (formula 16-12)

C_r =	1.35	F_{bE} =	1666	f_b =	1632.1 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	6245 psi	F_b' =	6245 psi	F_{cE} =	439 psi
R_B =	27.1	S_x =	8.8 in^3	K_f =	
f_c =	29 psi	F_c' =	429 psi	C_p =	0.107 in
Load at Base=	60 plf	**C.S.R =**	**0.285 O.K.**		

Load Combination #5 D + 0.7E (formula 16-12)

C_r =	1.15	F_{bE} =	1666	f_b =	206.0 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	5319 psi	F_b' =	5319 psi	F_{cE} =	439 psi
R_B =	27.1	S_x =	8.8 in^3	K_f =	
f_c =	29 psi	F_c' =	429 psi	C_p =	0.107 in
Load at Base=	60 plf	**C.S.R =**	**0.046 O.K.**		

Load Combination #6 D + 0.75L + 0.75S + 0.45W (formula 16-13)

C_r =	1.35	F_{bE} =	1666	f_b =	1224.1 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	6245 psi	F_b' =	6245 psi	F_{cE} =	439 psi
R_B =	27.1	S_x =	8.8 in^3	K_f =	
f_c =	69 psi	F_c' =	429 psi	C_p =	0.107 in
Load at Base=	651 plf	**C.S.R =**	**0.259 O.K.**		

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_r =	1.15	F_{bE} =	1666	f_b =	154.5 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	5319 psi	F_b' =	5319 psi	F_{cE} =	439 psi
R_B =	27.1	S_x =	8.8 in^3	K_f =	
f_c =	41 psi	F_c' =	429 psi	C_p =	0.107 in
Load at Base=	360 plf	**C.S.R =**	**0.041 O.K.**		

based on NDS equation 15.4-1:

C11

ARW ENGINEERS

Wood Stud Design Based on IBC 2015 & 2015 NDS

Version Date: May 25, 2017

9-Aug-18
9:34 AM

JOB TITLE: NAC Rec Center
WALL LOCATION: 5ft opening trimmer

JOB #: 18121
ENGINEER: TMD

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			WALL WEIGHTS		
UNIFORM SNOW:	2910	plf	SELF WEIGHT:	10	psf
UNIFORM LIVE:	0	plf	FINISHES WEIGHT:	5	psf
UNIFORM DEAD:	450	plf	S_{ds}:	0.541	g
DESIGN ROOF SNOW LOAD:	97	psf	I_e:	1	
BUILDING ELEVATION:	5500	ft.	**APPLIED LATERAL LOADS**		
ALLOWABLE SOIL BEARING:	1500	psf	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" | X |

STUD CHARACTERISTICS

NOMINAL STUD SIZE: (2)1.75x5.5

REQUIRED SQUARE FOOTING SIZE: 3.0 ft (Footing sized for bearing only) — 2.5' x 3.5

	b(in.)		d(in.)			
STUD SIZE (actual)	3.5	X	5.5	TRIB. WIDTH	38	inches
STUD LENGTH	12.5 ft.			ECCENTRICITY	0	inches (at top of wall)

MATERIAL PROPERTIES

WALL STUDS			BOTTOM PLATE		
Material:	LVL 2.0E		Material:	LVL 2.0E	
F_b	2891	psi	f_c =	568	psi
F_c	2510	psi	F_{cperp}' =	750	psi — **O.K.**
E_{min}	1,016,535	psi	E =	2,000,000	psi

ANALYSIS

bending CF: 1 (2x4-1.5, 2x6-1.3, 2x8-1.2, 4x4-1.5) verify with table 4A - Not for engineered lumber
comp. CF: 1 (2x4-1.15, 2x6-1.1, 2x8-1.05, 4x4-1.15) verify with table 4A - Not for engineered lumber

Bending effective length					Compression effective length				
unsupported length, l_{u1}	150	l_{u1}/d_1 =	27.27	in.	unsupported length, l_{u1}	150	k_{e1} =	1.00	
unsupported length, l_{u2}	12	l_{u2}/d_2 =	3.43	in.	unsupported length, l_{u2}	12	k_{e2} =	1.00	
unbraced length, l_{e1}	261.00				unbraced length, l_{e1}	150.0	d_1 =	5.5	in.
unbraced length, l_{e2}	24.72				unbraced length, l_{e2}	12.0	d_2 =	3.5	in.
bending C_r	1	(Gravity Load Combs.)			l_{e1}/d_1 =	27.27			
Wind bending C_r	1	(Only for Combs. Including Wind)			l_{e2}/d_2 =	3.43			

Load Combination #1 D + L (formula 16-9)

C_r =	1	F_{bE} =	10410	f_b =	0.0 psi
C_D =	0.9	C_L =	1.000	F_c^* =	2259 psi
F_b^* =	2602 psi	F_b' =	2602 psi	F_{cE} =	1123 psi
R_B =	10.8	S_x =	17.6 in^3	K_f =	1
f_c =	89 psi	F_c' =	975 psi	C_p =	0.432
Load at Base=	2019 lb	C.S.R =	0.008 **O.K.**		

Load Combination #2 D + S (formula 16-10)

C_r =	1	F_{bE} =	10410	f_b =	0.0 psi
C_D =	1	C_L =	1.000	F_c^* =	2510 psi
F_b^* =	2891 psi	F_b' =	2891 psi	F_{cE} =	1123 psi
R_B =	10.8	S_x =	17.6 in^3	K_f =	1
f_c =	568 psi	F_c' =	993 psi	C_p =	0.396
Load at Base=	11234 lb	C.S.R =	0.327 **O.K.**		

Load Combination #3 D + 0.75L + 0.75S (formula 16-11)

C_r =	1	F_{bE} =	10410	f_b =	0.0 psi
C_D =	1	C_L =	1.000	F_c^* =	2510 psi
F_b^* =	2891 psi	F_b' =	2891 psi	F_{cE} =	1123 psi
R_B =	10.8	S_x =	17.6 in^3	K_f =	1
f_c =	448 psi	F_c' =	993 psi	C_p =	0.396 in
Load at Base=	8930 lb	C.S.R =	0.204 **O.K.**		

Load Combination #4 D + 0.6W (formula 16-12)

C_r =	1	F_{bE} =	10410	f_b =	0.0 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	4626 psi	F_b' =	4626 psi	F_{cE} =	1123 psi
R_B =	10.8	S_x =	17.6 in^3	K_f =	1
f_c =	89 psi	F_c' =	1049 psi	C_p =	0.261 in
Load at Base=	2019 lb	C.S.R =	0.085 **O.K.**		

Load Combination #5 D + 0.7E (formula 16-12)

C_r =	1	F_{bE} =	10410	f_b =	95.6 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	4626 psi	F_b' =	4626 psi	F_{cE} =	1123 psi
R_B =	10.8	S_x =	17.6 in^3	K_f =	1
f_c =	89 psi	F_c' =	1049 psi	C_p =	0.261 in
Load at Base=	2019 lb	C.S.R =	0.030 **O.K.**		

Load Combination #6 D + 0.75L + 0.75S + 0.45W (formula 16-13)

C_r =	1	F_{bE} =	10410	f_b =	0.0 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	4626 psi	F_b' =	4626 psi	F_{cE} =	1123 psi
R_B =	10.8	S_x =	17.6 in^3	K_f =	1
f_c =	448 psi	F_c' =	1049 psi	C_p =	0.261 in
Load at Base=	8930 lb	C.S.R =	0.427 **O.K.**		

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_r =	1	F_{bE} =	10410	f_b =	71.7 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	4626 psi	F_b' =	4626 psi	F_{cE} =	1123 psi
R_B =	10.8	S_x =	17.6 in^3	K_f =	1
f_c =	191 psi	F_c' =	1049 psi	C_p =	0.261 in
Load at Base=	2019 lb	C.S.R =	0.052 **O.K.**		

based on NDS equation 15.4-1:

C12

ARW ENGINEERS

9-Aug-18
10:13 AM

Wood Stud Design Based on IBC 2015 & 2015 NDS

Version Date: May 25, 2017

JOB TITLE: NAC Rec Center
WALL LOCATION: 5ft opening king

JOB #: 18121
ENGINEER: TMD

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			WALL WEIGHTS		
UNIFORM SNOW:	0	plf	SELF WEIGHT:	10	psf
UNIFORM LIVE:	0	plf	FINISHES WEIGHT:	5	psf
UNIFORM DEAD:	0	plf	S_{ds}:	0.541	g
DESIGN ROOF SNOW LOAD:	97	psf	I_e:	1	
BUILDING ELEVATION:	5500	ft.	**APPLIED LATERAL LOADS**		
ALLOWABLE SOIL BEARING:	1500	psf	WIND (W):	30	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" | X |

STUD CHARACTERISTICS | **REQUIRED SQUARE FOOTING SIZE:** 2.0 ft (Footing sized for bearing only)

NOMINAL STUD SIZE: 1.75x5.5

	b(in.)		d(in.)			
STUD SIZE (actual)	1.75	X	5.5	TRIB. WIDTH	38	inches
STUD LENGTH	20 ft.			ECCENTRICITY	0	inches (at top of wall)

MATERIAL PROPERTIES

WALL STUDS			BOTTOM PLATE			
Material:	LVL 2.0E		Material:	LVL 2.0E		
F_b	2891	psi	f_c =	49	psi	
F_c	2510	psi	F_{cperp}' =	750	psi	O.K.
E_{min}	1,016,535	psi	E =	2,000,000	psi	

ANALYSIS

bending CF: 1 (2x4-1.5, 2x6-1.3, 2x8-1.2, 4x4-1.5) verify with table 4A - Not for engineered lumber
comp. CF: 1 (2x4-1.15, 2x6-1.1, 2x8-1.05, 4x4-1.15) verify with table 4A - Not for engineered lumber

Bending effective length					Compression effective length				
unsupported length, l_{u1}	240	l_{u1}/d_1 =	43.64	in.	unsupported length, l_{u1}	240	k_{e1} =	1.00	
unsupported length, l_{u2}	12	l_{u2}/d_2 =	6.86	in.	unsupported length, l_{u2}	12	k_{e2} =	1.00	
unbraced length, l_{e1}	407.70				unbraced length, l_{e1}	240.0	d_1 =	5.5	in.
unbraced length, l_{e2}	24.72				unbraced length, l_{e2}	12.0	d_2 =	1.75	in.
bending C_r	1	(Gravity Load Combs.)			l_{e1}/d_1 =	43.64			
Wind bending C_r	1	(Only for Combs. Including Wind)			l_{e2}/d_2 =	6.86			

Load Combination #1 D + L (formula 16-9)

C_r =	1	F_{bE} =	1666	f_b =	0.0 psi
C_D =	0.9	C_L =	1.000	F_c^* =	2259 psi
F_b^* =	2602 psi	F_b' =	2602 psi	F_{cE} =	439 psi
R_B =	27.1	S_x =	8.8 in^3	K_f =	1
f_c =	49 psi	F_c' =	420 psi	C_p =	0.186
Load at Base=	950 lb	C.S.R =	0.014 O.K.		

Load Combination #2 D + S (formula 16-10)

C_r =	1	F_{bE} =	1666	f_b =	0.0 psi
C_D =	0.9	C_L =	1.000	F_c^* =	2259 psi
F_b^* =	2602 psi	F_b' =	2602 psi	F_{cE} =	439 psi
R_B =	27.1	S_x =	8.8 in^3	K_f =	1
f_c =	49 psi	F_c' =	420 psi	C_p =	0.186
Load at Base=	950 lb	C.S.R =	0.014 O.K.		

Load Combination #3 D + 0.75L + 0.75S (formula 16-11)

C_r =	1	F_{bE} =	1666	f_b =	0.0 psi
C_D =	0.9	C_L =	1.000	F_c^* =	2259 psi
F_b^* =	2602 psi	F_b' =	2602 psi	F_{cE} =	439 psi
R_B =	27.1	S_x =	8.8 in^3	K_f =	1
f_c =	49 psi	F_c' =	420 psi	C_p =	0.186 in
Load at Base=	950 lb	C.S.R =	0.014 O.K.		

Load Combination #4 D + 0.6W (formula 16-12)

C_r =	1	F_{bE} =	1666	f_b =	3876.3 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	4626 psi	F_b' =	4626 psi	F_{cE} =	439 psi
R_B =	27.1	S_x =	8.8 in^3	K_f =	1
f_c =	49 psi	F_c' =	429 psi	C_p =	0.107 in
Load at Base=	950 lb	C.S.R =	0.957 O.K.		

Load Combination #5 D + 0.7E (formula 16-12)

C_r =	1	F_{bE} =	1666	f_b =	489.3 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	4626 psi	F_b' =	4626 psi	F_{cE} =	439 psi
R_B =	27.1	S_x =	8.8 in^3	K_f =	1
f_c =	49 psi	F_c' =	429 psi	C_p =	0.107 in
Load at Base=	950 lb	C.S.R =	0.132 O.K.		

Load Combination #6 D + 0.75L + 0.75S + 0.45W (formula 16-13)

C_r =	1	F_{bE} =	1666	f_b =	2907.2 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	4626 psi	F_b' =	4626 psi	F_{cE} =	439 psi
R_B =	27.1	S_x =	8.8 in^3	K_f =	1
f_c =	49 psi	F_c' =	429 psi	C_p =	0.107 in
Load at Base=	950 lb	C.S.R =	0.721 O.K.		

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_r =	1	F_{bE} =	1666	f_b =	367.0 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	4626 psi	F_b' =	4626 psi	F_{cE} =	439 psi
R_B =	27.1	S_x =	8.8 in^3	K_f =	1
f_c =	49 psi	F_c' =	429 psi	C_p =	0.107 in
Load at Base=	950 lb	C.S.R =	0.103 O.K.		

based on NDS equation 15.4-1:

C13

ARW ENGINEERS

Wood Stud Design Based on IBC 2015 & 2015 NDS

Version Date: May 25, 2017

9-Aug-18
10:05 AM

JOB TITLE: NAC Rec Center
WALL LOCATION: 12ft opening trimmer

JOB #: 18121
ENGINEER: TMD

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			WALL WEIGHTS		
UNIFORM SNOW:	970	plf	SELF WEIGHT:	10	psf
UNIFORM LIVE:	0	plf	FINISHES WEIGHT:	5	psf
UNIFORM DEAD:	150	plf	S_{ds}:	0.541	g
DESIGN ROOF SNOW LOAD:	97	psf	I_e:	1	
BUILDING ELEVATION:	5500	ft.	APPLIED LATERAL LOADS		
ALLOWABLE SOIL BEARING:	1500	psf	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" — X

STUD CHARACTERISTICS

NOMINAL STUD SIZE: (2)1.75x5.5

REQUIRED SQUARE FOOTING SIZE: 3.0 ft (Footing sized for bearing only) 2.5' x 3.5'

	b(in.)		d(in.)			
STUD SIZE (actual)	3.5	X	5.5	TRIB. WIDTH	80	inches
STUD LENGTH	11 ft.			ECCENTRICITY	0	inches (at top of wall)

MATERIAL PROPERTIES

WALL STUDS			BOTTOM PLATE		
Material:	LVL 2.0E		Material:	LVL 2.0E	
F_b	2891	psi	f_c =	416	psi
F_c	2510	psi	F_{cperp}' =	750	psi O.K.
E_{min}	1,016,535	psi	E =	2,000,000	psi

ANALYSIS

bending CF: 1 (2x4-1.5, 2x6-1.3, 2x8-1.2, 4x4-1.5) verify with table 4A - Not for engineered lumber
comp. CF: 1 (2x4-1.15, 2x6-1.1, 2x8-1.05, 4x4-1.15) verify with table 4A - Not for engineered lumber

Bending effective length				Compression effective length				
unsupported length, l_{u1}	132	l_{u1}/d_1 = 24.00	in.	unsupported length, l_{u1}	132	k_{e1} =	1.00	
unsupported length, l_{u2}	12	l_{u2}/d_2 = 3.43	in.	unsupported length, l_{u2}	12	k_{e2} =	1.00	
unbraced length, l_{e1}	231.66			unbraced length, l_{e1}	132.0	d_1 =	5.5	in.
unbraced length, l_{e2}	24.72			unbraced length, l_{e2}	12.0	d_2 =	3.5	in.
bending C_r	1	(Gravity Load Combs.)		l_{e1}/d_1 =	24.00			
Wind bending C_r	1	(Only for Combs. Including Wind)		l_{e2}/d_2 =	3.43			

Load Combination #1 D + L (formula 16-9)

C_r =	1	F_{bE} =	11728	f_b =	0.0 psi
C_D =	0.9	C_L =	1.000	F_c^* =	2259 psi
F_b^* =	2602 psi	F_b' =	2602 psi	F_{cE} =	1451 psi
R_B =	10.2	S_x =	17.6 in³	K_f =	1
f_c =	81 psi	F_c' =	1187 psi	C_p =	0.526
Load at Base=	2100 lb	C.S.R =	0.005 O.K.		

Load Combination #2 D + S (formula 16-10)

C_r =	1	F_{bE} =	11728	f_b =	0.0 psi
C_D =	1	C_L =	1.000	F_c^* =	2510 psi
F_b^* =	2891 psi	F_b' =	2891 psi	F_{cE} =	1451 psi
R_B =	10.2	S_x =	17.6 in³	K_f =	1
f_c =	416 psi	F_c' =	1220 psi	C_p =	0.486
Load at Base=	8567 lb	C.S.R =	0.117 O.K.		

Load Combination #3 D + 0.75L + 0.75S (formula 16-11)

C_r =	1	F_{bE} =	11728	f_b =	0.0 psi
C_D =	1	C_L =	1.000	F_c^* =	2510 psi
F_b^* =	2891 psi	F_b' =	2891 psi	F_{cE} =	1451 psi
R_B =	10.2	S_x =	17.6 in³	K_f =	1
f_c =	332 psi	F_c' =	1220 psi	C_p =	0.486 in
Load at Base=	6950 lb	C.S.R =	0.074 O.K.		

Load Combination #4 D + 0.6W (formula 16-12)

C_r =	1	F_{bE} =	11728	f_b =	0.0 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	4626 psi	F_b' =	4626 psi	F_{cE} =	1451 psi
R_B =	10.2	S_x =	17.6 in³	K_f =	1
f_c =	81 psi	F_c' =	1321 psi	C_p =	0.329 in
Load at Base=	2100 lb	C.S.R =	0.061 O.K.		

Load Combination #5 D + 0.7E (formula 16-12)

C_r =	1	F_{bE} =	11728	f_b =	155.8 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	4626 psi	F_b' =	4626 psi	F_{cE} =	1451 psi
R_B =	10.2	S_x =	17.6 in³	K_f =	1
f_c =	81 psi	F_c' =	1321 psi	C_p =	0.329 in
Load at Base=	2100 lb	C.S.R =	0.039 O.K.		

Load Combination #6 D + 0.75L + 0.75S + 0.45W (formula 16-13)

C_r =	1	F_{bE} =	11728	f_b =	0.0 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	4626 psi	F_b' =	4626 psi	F_{cE} =	1451 psi
R_B =	10.2	S_x =	17.6 in³	K_f =	1
f_c =	332 psi	F_c' =	1321 psi	C_p =	0.329 in
Load at Base=	6950 lb	C.S.R =	0.252 O.K.		

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_r =	1	F_{bE} =	11728	f_b =	116.9 psi
C_D =	1.6	C_L =	1	F_c^* =	4016 psi
F_b^* =	4626 psi	F_b' =	4626 psi	F_{cE} =	1451 psi
R_B =	10.2	S_x =	17.6 in³	K_f =	1
f_c =	152 psi	F_c' =	1321 psi	C_p =	0.329 in
Load at Base=	2100 lb	C.S.R =	0.041 O.K.		

based on NDS equation 15.4-1:

C14

ENGINEERS

9-Aug-18
10:05 AM

Wood Stud Design Based on IBC 2015 & 2015 NDS

Version Date: May 25, 2017

JOB TITLE: NAC Rec Center
WALL LOCATION: 12ft opening king

JOB #: 18121
ENGINEER: TMD

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			WALL WEIGHTS		
UNIFORM SNOW:	0	plf	SELF WEIGHT:	10	psf
UNIFORM LIVE:	0	plf	FINISHES WEIGHT:	5	psf
UNIFORM DEAD:	0	plf	S_{ds}:	0.541	g
DESIGN ROOF SNOW LOAD:	97	psf	I_e:	1	
BUILDING ELEVATION:	5500	ft.	**APPLIED LATERAL LOADS**		
ALLOWABLE SOIL BEARING:	1500	psf	WIND (W):	30	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" | X |

STUD CHARACTERISTICS
NOMINAL STUD SIZE: (3)1.75x5.5
REQUIRED SQUARE FOOTING SIZE: 2.0 ft (Footing sized for bearing only)

STUD SIZE (actual): b(in.) 5.25 X d(in.) 5.5
TRIB. WIDTH: 80 inches
STUD LENGTH: 20 ft.
ECCENTRICITY: 0 inches (at top of wall)

MATERIAL PROPERTIES

WALL STUDS			BOTTOM PLATE			
Material:	LVL 2.0E		Material:	LVL 2.0E		
F_b	2891	psi	f_c =	35	psi	
F_c	2510	psi	F_{cperp}' =	750	psi	**O.K.**
E_{min}	1,016,535	psi	E =	2,000,000	psi	

ANALYSIS

bending CF: 1 (2x4-1.5, 2x6-1.3, 2x8-1.2, 4x4-1.5) verify with table 4A - Not for engineered lumber
comp. CF: 1 (2x4-1.15, 2x6-1.1, 2x8-1.05, 4x4-1.15) verify with table 4A - Not for engineered lumber

Bending effective length					Compression effective length				
unsupported length, l_{u1}	240	l_{u1}/d_1 =	43.64	in.	unsupported length, l_{u1}	240	k_{e1} =	1.00	
unsupported length, l_{u2}	12	l_{u2}/d_2 =	2.29	in.	unsupported length, l_{u2}	12	k_{e2} =	1.00	
unbraced length, l_{e1}	407.70				unbraced length, l_{e1}	240.0	d_1 =	5.5	in.
unbraced length, l_{e2}	24.72				unbraced length, l_{e2}	12.0	d_2 =	5.25	in.
bending C_r	1	(Gravity Load Combs.)			l_{e1}/d_1 =	43.64			
Wind bending C_r	1	(Only for Combs. Including Wind)			l_{e2}/d_2 =	2.29			

Load Combination #1 D + L (formula 16-9)

C_r =	1	F_{bE} =	14994		f_b =	0.0	psi
C_D =	0.9	C_L =	1.000		F_c^* =	2259	psi
F_b^* =	2602 psi	F_b' =	2602	psi	F_{cE} =	439	psi
R_B =	9.0	S_x =	26.5	in^3	K_f =	1	
f_c =	35 psi	F_c' =	420	psi	C_p =	0.186	
Load at Base=	2000 lb	**C.S.R =**	**0.007**	**O.K.**			

Load Combination #2 D + S (formula 16-10)

C_r =	1	F_{bE} =	14994		f_b =	0.0	psi
C_D =	0.9	C_L =	1.000		F_c^* =	2259	psi
F_b^* =	2602 psi	F_b' =	2602	psi	F_{cE} =	439	psi
R_B =	9.0	S_x =	26.5	in^3	K_f =	1	
f_c =	35 psi	F_c' =	420	psi	C_p =	0.186	
Load at Base=	2000 lb	**C.S.R =**	**0.007**	**O.K.**			

Load Combination #3 D + 0.75L + 0.75S (formula 16-11)

C_r =	1	F_{bE} =	14994		f_b =	0.0	psi
C_D =	0.9	C_L =	1.000		F_c^* =	2259	psi
F_b^* =	2602 psi	F_b' =	2602	psi	F_{cE} =	439	psi
R_B =	9.0	S_x =	26.5	in^3	K_f =	1	
f_c =	35 psi	F_c' =	420	psi	C_p =	0.186	in
Load at Base=	2000 lb	**C.S.R =**	**0.007**	**O.K.**			

Load Combination #4 D + 0.6W (formula 16-12)

C_r =	1	F_{bE} =	14994		f_b =	2720.2	psi
C_D =	1.6	C_L =	1		F_c^* =	4016	psi
F_b^* =	4626 psi	F_b' =	4626	psi	F_{cE} =	439	psi
R_B =	9.0	S_x =	26.5	in^3	K_f =	1	
f_c =	35 psi	F_c' =	429	psi	C_p =	0.107	in
Load at Base=	2000 lb	**C.S.R =**	**0.645**	**O.K.**			

Load Combination #5 D + 0.7E (formula 16-12)

C_r =	1	F_{bE} =	14994		f_b =	343.4	psi
C_D =	1.6	C_L =	1		F_c^* =	4016	psi
F_b^* =	4626 psi	F_b' =	4626	psi	F_{cE} =	439	psi
R_B =	9.0	S_x =	26.5	in^3	K_f =	1	
f_c =	35 psi	F_c' =	429	psi	C_p =	0.107	in
Load at Base=	2000 lb	**C.S.R =**	**0.087**	**O.K.**			

Load Combination #6 D + 0.75L + 0.75S + 0.45W (formula 16-13)

C_r =	1	F_{bE} =	14994		f_b =	2040.1	psi
C_D =	1.6	C_L =	1		F_c^* =	4016	psi
F_b^* =	4626 psi	F_b' =	4626	psi	F_{cE} =	439	psi
R_B =	9.0	S_x =	26.5	in^3	K_f =	1	
f_c =	35 psi	F_c' =	429	psi	C_p =	0.107	in
Load at Base=	2000 lb	**C.S.R =**	**0.485**	**O.K.**			

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_r =	1	F_{bE} =	14994		f_b =	257.5	psi
C_D =	1.6	C_L =	1		F_c^* =	4016	psi
F_b^* =	4626 psi	F_b' =	4626	psi	F_{cE} =	439	psi
R_B =	9.0	S_x =	26.5	in^3	K_f =	1	
f_c =	35 psi	F_c' =	429	psi	C_p =	0.107	in
Load at Base=	2000 lb	**C.S.R =**	**0.067**	**O.K.**			

based on NDS equation 15.4-1:

C15

ARW ENGINEERS

Simply Supported Wood Beam Design (IBC 2015 and NDS 2015)

Version Date: October 28, 2016

JOB TITLE: NAC Rec
DESCRIPTION: HEADERS

9-Aug-18
9:50 AM

JOB #: 18121
DESIGNER: TMD

SECTION PROPERTIES / **ADJUSTMENT FACTORS** / **CHOOSE MATERIAL PROPERTY: DF-L 2**

	b	d	A	S	I	C_F	L_u/d	L_e	R_B	F_b^*	E'	E'_{min}	F_{bE}	C_L	F_b'
2x4	1.5	3.5	5.25	3.063	5.359	1.5	3.43	24.72	6.20	1350	1600000	580000	18100	0.996	1345
2x6	1.5	5.5	8.25	7.563	20.797	1.3	2.18	24.72	7.77	1170	1600000	580000	11518	0.994	1163
2x8	1.5	7.25	10.875	13.141	47.635	1.2	1.66	24.72	8.92	1080	1600000	580000	8738	0.993	1072
2x10	1.5	9.25	13.875	21.391	98.932	1.1	1.30	24.72	10.08	990	1600000	580000	6849	0.992	982
2x12	1.5	11.25	16.875	31.641	177.979	1	1.07	24.72	11.12	900	1600000	580000	5631	0.991	892
(2) 2x4	3	3.5	10.5	6.125	10.719	1.5	3.43	24.72	3.10	1350	1600000	580000	72399	0.999	1349
(2) 2x6	3	5.5	16.5	15.125	41.594	1.3	2.18	24.72	3.89	1170	1600000	580000	46072	0.999	1168
(2) 2x8	3	7.25	21.75	26.281	95.270	1.2	1.66	24.72	4.46	1080	1600000	580000	34951	0.998	1078
(2) 2x10	3	9.25	27.75	42.781	197.863	1.1	1.30	24.72	5.04	990	1600000	580000	27394	0.998	988
(2) 2x12	3	11.25	33.75	63.281	355.957	1	1.07	24.72	5.56	900	1600000	580000	22524	0.998	898
(3) 2x4	4.5	3.5	15.75	9.188	16.078	1.5	3.43	24.72	2.07	1350	1600000	580000	162899	1.000	1349
(3) 2x6	4.5	5.5	24.75	22.688	62.391	1.3	2.18	24.72	2.59	1170	1600000	580000	103663	0.999	1169
(3) 2x8	4.5	7.25	32.625	39.422	142.904	1.2	1.66	24.72	2.97	1080	1600000	580000	78641	0.999	1079
(3) 2x10	4.5	9.25	41.625	64.172	296.795	1.1	1.30	24.72	3.36	990	1600000	580000	61637	0.999	989
(3) 2x12	4.5	11.25	50.625	94.922	533.936	1	1.07	24.72	3.71	900	1600000	580000	50680	0.999	899
						TABLE 4A	TABLE 3.3.3	TABLE 3.3.3	EQN 3.3-5	SECT 3.3.3	SECT 3.3.3	SECT 3.3.3	SECT 3.3.3	EQN 3.3-6	TABLE 2.3.1

UNBRACED LENGTH

L_u 1 ft

ADJUSTMENT FACTORS

C_D	1.00
C_M	1.00
C_t	1.00
C_{fu}	1.00
C_i	1.00
C_r	1.00
C_T	1.00
Density	33.00

DEFLECTION CRITERIA

L /	240	Total Load
L /	360	Live Load

MATERIAL PROPERTIES (NDS 2010)

	F_b	F_t	F_v	F_{cperp}	F_{cpar}	E	E_{min}	F_v'
DF-L S	1500	1000	180	625	1700	1900000	690000	180
DF-L 1 BTR	1200	800	180	625	1550	1800000	660000	180
DF-L 1	1000	675	180	625	1500	1700000	620000	180
DF-L 2	900	575	180	625	1350	1600000	580000	180
HEM-FIR S	1400	925	150	405	1500	1600000	580000	150
HEM-FIR 1 B	1100	725	150	405	1350	1500000	550000	150
HEM-FIR 1	975	625	150	405	1350	1500000	550000	150
HEM-FIR 2	850	525	150	405	1300	1300000	470000	150

INDICATES THE CONTROLLING ALLOWABLE UNIFORM LOAD

DF-L 2

ALLOWABLE UNIFORM TOTAL LOAD (PLF) Based on bending stress $\omega = 8*S*F_b*12/(L*12)^2$

SPAN (FT)	2x4	2x6	2x8	2x10	2x12	(2) 2x4	(2) 2x6	(2) 2x8	(2) 2x10	(2) 2x12	(3) 2x4	(3) 2x6	(3) 2x8	(3) 2x10	(3) 2x12
4	170	365	585	872	1172	342	733	1176	1755	2360	513	1100	1765	2635	3545
5	109	233	373	557	748	218	468	751	1121	1508	327	702	1127	1683	2264
6	75	161	258	386	519	151	324	520	776	1045	226	486	780	1166	1569
7	55	118	189	283	380	110	237	381	569	766	165	355	571	854	1150
8	42	90	144	216	290	84	180	290	434	584	126	271	436	652	877
9	33	71	114	170	228	66	142	228	342	460	98	213	343	513	691
10	26	57	91	137	184	53	114	184	275	371	79	171	276	414	557
11	21	47	75	113	152	43	94	151	227	305	65	140	227	340	459
12	18	39	63	94	127	36	78	126	189	255	54	117	189	284	384
13	15	33	53	80	107	30	66	107	160	216	45	99	160	241	325
14	13	28	45	68	92	26	56	91	137	186	39	85	137	206	279
15	11	24	39	59	80	22	49	79	119	161	33	73	119	179	241
16	10	21	34	52	70	19	42	69	104	140	29	63	103	156	211

TYP WORST CASE
1120 PLF
GRID B

DF-L 2

ALLOWABLE UNIFORM TOTAL LOAD (PLF) Based on deflection $\omega = 384*E*I*12/(TOTAL\ LOAD*5*(L*12)^3)$

SPAN (FT)	2x4	2x6	2x8	2x10	2x12	(2) 2x4	(2) 2x6	(2) 2x8	(2) 2x10	(2) 2x12	(3) 2x4	(3) 2x6	(3) 2x8	(3) 2x10	(3) 2x12
4	297	1153	2644	5493	9884	593	2307	5288	10986	19768	890	3460	7932	16479	29651
5	151	590	1352	2811	5059	302	1179	2705	5622	10117	454	1769	4057	8433	15176
6	87	340	782	1625	2926	174	681	1563	3251	5852	261	1021	2345	4876	8777
7	54	214	491	1022	1841	109	427	983	2045	3682	163	641	1474	3067	5523
8	36	143	328	684	1232	72	285	657	1368	2464	108	428	985	2052	3696
9	25	100	230	479	864	50	199	460	959	1728	75	299	690	1438	2593
10	18	72	167	349	629	36	144	334	697	1258	54	216	501	1046	1887
11	13	54	125	261	472	26	107	250	522	943	39	161	374	783	1415
12	10	41	96	200	362	20	82	191	401	725	29	123	287	601	1087
13	7	32	75	157	284	15	64	149	314	568	22	95	224	471	853
14	6	25	59	125	227	11	50	118	250	453	17	75	178	375	680
15	4	20	48	101	184	9	40	95	202	367	13	60	143	303	551
16	3	16	39	83	151	7	32	78	165	301	10	48	117	248	452

DF-L 2

ALLOWABLE UNIFORM LIVE LOAD (PLF) Based on deflection $\omega = 384*E*I*12/(LIVE\ LOAD*5*(L*12)^3)$

SPAN (FT)	2x4	2x6	2x8	2x10	2x12	(2) 2x4	(2) 2x6	(2) 2x8	(2) 2x10	(2) 2x12	(3) 2x4	(3) 2x6	(3) 2x8	(3) 2x10	(3) 2x12
4	197	768	1762	3661	6588	395	1537	3524	7322	13176	592	2305	5285	10983	19764
5	100	392	901	1873	3371	201	785	1802	3746	6742	301	1177	2702	5619	10113
6	58	226	520	1082	1949	115	453	1040	2165	3899	173	679	1561	3247	5848
7	36	142	327	681	1226	72	284	653	1361	2452	108	425	980	2042	3678
8	24	94	218	455	820	47	189	436	910	1640	71	283	654	1365	2460
9	16	66	152	319	575	32	131	305	637	1150	49	197	457	956	1725
10	12	47	110	231	418	23	95	221	463	836	35	142	331	694	1254
11	8	35	82	173	313	17	70	165	346	626	25	105	247	519	939
12	6	27	63	133	240	12	53	126	265	481	18	80	189	398	721
13	5	21	49	104	188	9	41	98	207	376	14	62	147	311	564
14	3	16	39	82	150	7	32	77	165	300	10	48	116	247	450
15	3	13	31	66	121	5	25	62	133	242	8	38	93	199	363
16	2	10	25	54	99	4	20	50	108	198	6	30	75	162	297

DF-L 2

ALLOWABLE UNIFORM TOTAL LOAD (PLF) Based on shear stress $\omega = 2*b*d*F_{v*12}/(3*((L*12)/2-d))$

SPAN (FT)	2x4	2x6	2x8	2x10	2x12	(2) 2x4	(2) 2x6	(2) 2x8	(2) 2x10	(2) 2x12	(3) 2x4	(3) 2x6	(3) 2x8	(3) 2x10	(3) 2x12
4	368	640	932	1351	1902	735	1281	1865	2703	3804	1103	1921	2797	4054	5706
5	284	483	686	960	1292	568	966	1372	1919	2584	852	1449	2058	2879	3876
6	231	388	542	744	978	463	775	1084	1487	1956	694	1163	1627	2231	2934
7	195	324	448	607	786	390	647	896	1214	1573	585	971	1344	1821	2359
8	169	278	382	512	657	337	555	764	1025	1315	506	833	1145	1537	1972
9	148	243	332	443	565	297	486	665	887	1129	445	729	997	1330	1694
10	133	216	294	391	495	265	432	589	781	989	398	648	883	1172	1484
11	120	194	264	349	440	240	389	528	698	880	359	583	792	1047	1320
12	109	177	239	315	396	218	354	479	630	792	327	530	718	946	1188
13	100	162	219	287	360	201	324	438	575	720	301	486	657	862	1081
14	93	149	202	264	330	185	299	403	528	660	278	448	605	792	990
15	86	139	187	244	305	172	277	374	489	609	259	416	560	733	914
16	81	129	174	227	283	161	259	348	454	566	242	388	522	681	849

C16

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.*

GRID K
448 PLF

GRID 3÷6
3360 PLF

GRID H
1120 PLF

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

Span	Condition	3½" Width (2-ply)				5¼" Width (3-ply)									
		14"	16"	18"	20"	5½"	7¼"	9¼"	9½"	11¼"	11⅞"	14"	16"	18"	20"
6'	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
	Live Load L/360	*	*	*	*	870	1,879	*	*	*	*	*	*	*	*
	Min. End/Int. Bearing (in.)	4.1/10.3	4.5/11.3	4.5/11.3	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
8'	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	4,399
	Live Load L/360	*	*	*	*	380	842	1,666	1,792	*	*	*	*	*	*
	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
9'-6"	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
	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	4.5/11.3
10'	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
	Live Load L/360	*	*	*	*	*	*	893	963	1,544	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	4.5/11.2
12'	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
	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
14'	Total Load	975	1,253	1,563	1,667		100	494	535	879	1,028	1,463	1,880	2,345	2,500
	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
16'-6"	Total Load	684	897	1,120	1,365			300	326	540	634	1,026	1,346	1,680	2,048
	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
18'-6"	Total Load	488	710	887	1,082			210	228	382	449	733	1,066	1,331	1,623
	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
20'	Total Load	387	573	756	922			164	178	300	354	580	860	1,134	1,384
	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
22'	Total Load	289	432	611	759			120	131	223	263	434	648	916	1,138
	Live Load L/360	202	298	419	566			89	97	160	187	304	448	629	850
	Min. End/Int. Bearing (in.)	1.5/3.5	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
24'	Total Load	221	332	471	634			89	98	168	199	332	498	707	951
	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
26'	Total Load	172	259	370	506			67	74	129	153	258	389	555	760
	Live Load L/360	124	183	259	351			54	59	97	114	186	275	388	527
	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
28'	Total Load	135	205	294	405			51	56	100	120	203	308	442	607
	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
30'	Total Load	108	164	237	327					78	94	162	247	356	491
	Live Load L/360	81	120	170	232					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

*Indicates **Total Load** value controls.

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Project Title:
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C17

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Steel Beam

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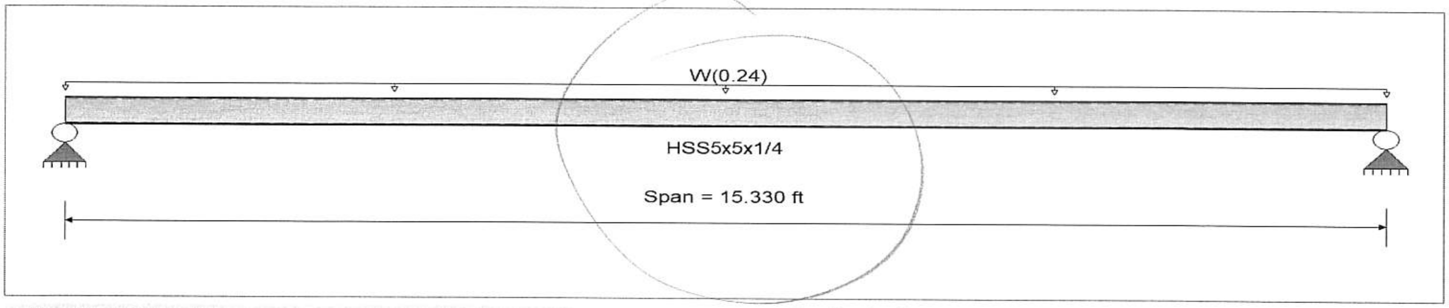
Description : HSS Girt @ Grid 1/D-E (out-of-plane)

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



Applied Loads

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

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

DESIGN SUMMARY

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	7.665 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 <360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.291 in	Ratio =	633 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
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	1		0.000				31.71	18.99	1.00	1.00	-0.00	60.13	36.00
+0.60W													
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													
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

Overall Maximum Deflections

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

Cantilevered Retaining Wall

Description : Screen Wall CMU

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.

Soil Data

Allow Soil Bearing	=	1,800.0 psf
Equivalent Fluid Pressure Method		
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	=	12.00 in

Calculations per ACI 318-14, ACI 530-11, IBC 2015, CBC 2016, ASCE 7-10

Surcharge Loads

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

Axial Load Applied to Stem

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

Lateral Load Applied to Stem

Lateral Load	=	0.0 plf
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft

Wind on Exposed Stem	=	30.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Cantilevered Retaining Wall

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Description : Screen Wall

Design Summary

Wall Stability Ratios

Overturning	=	3.56 OK
Sliding	=	11.39 OK
Total Bearing Load	=	2,271 lbs
...resultant ecc.	=	6.75 in
Soil Pressure @ Toe	=	1,047 psf OK
Soil Pressure @ Heel	=	89 psf OK
Allowable	=	1,800 psf

Soil Pressure Less Than Allowable

ACI Factored @ Toe	=	1,256 psf
ACI Factored @ Heel	=	106 psf
Footing Shear @ Toe	=	4.7 psi OK
Footing Shear @ Heel	=	7.5 psi OK
Allowable	=	75.0 psi

Sliding Calcs (Vertical Component NOT Used)

Lateral Sliding Force	=	271.9 lbs
less 100% Passive Force	= -	2,188.1 lbs
less 100% Friction Force	= -	908.3 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	0.0 lbs OK

Load Factors

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

Stem Construction

		Top Stem	2nd
		Stem OK	Stem OK
Design Height Above Ftg	ft =	2.50	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	=	0.475	0.146
Total Force @ Section	lbs =	180.0	255.0
Moment....Actual	ft-l =	540.0	1,052.5
Moment.....Allowable	ft-l =	1,136.9	7,221.8
Shear.....Actual	psi =	4.0	7.6
Shear.....Allowable	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			
f'm	psi =	1,500	1,500
Fy (	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			
f'c	psi =		2,500.0
Fy	psi =		

Footing Dimensions & Strengths

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
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	1,256	106 psf
Mu' : Upward	=	1,523	0 ft-lb
Mu' : Downward	=	708	708 ft-lb
Mu: Design	=	814	708 ft-lb
Actual 1-Way Shear	=	4.68	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 & Spacings

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

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Engineer:
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Project Descr:

C20

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Cantilevered Retaining Wall

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Description : Screen Wall

Summary of Overturning & Resisting Forces & Moments

Item		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	275.6	1.17	321.6
Surcharge over Heel	=			
Toe Active Pressure	=	-183.8	1.17	-214.4
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=	180.0	6.50	1,170.0
Total	=	271.9	**O.T.M.** =	1,277.2

Resisting/Overturning Ratio = **3.56**
Vertical Loads used for Soil Pressure = 2,270.7 lbs

		.....RESISTING..... Force lbs	Distance ft	Moment ft-lb
Soil Over Heel	=	458.3	3.17	1,451.4
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	458.3	0.83	381.9
Surcharge Over Toe	=			
Stem Weight(s)	=	754.0	2.00	1,508.0
Earth @ Stem Transitions	=			
Footing Weight	=	600.0	2.00	1,200.0
Key Weight	=		2.00	
Vert. Component	=			
Total =		2,270.7 lbs	**R.M. =**	4,541.3

* 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:

C21

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Cantilevered Retaining Wall

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Lic. # : KW-06002489

Licensee : ARW ENGINEERS

Description : Screen Wall CONC

Calculations per ACI 318-14, ACI 530-11, IBC 2015, CBC 2016, ASCE 7-10

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.

Soil Data

Allow Soil Bearing	=	1,800.0 psf
Equivalent Fluid Pressure Method		
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	=	12.00 in

Surcharge Loads

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

Lateral Load Applied to Stem

Lateral Load	=	0.0 plf
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft

Wind on Exposed Stem = 30.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

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

Design Summary

Wall Stability Ratios			
Overturning	=	3.71	OK
Sliding	=	11.53	OK
Total Bearing Load	=	2,367 lbs	
...resultant ecc.	=	6.48 in	
Soil Pressure @ Toe	=	1,071 psf	OK
Soil Pressure @ Heel	=	113 psf	OK
Allowable	=	1,800 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,285 psf	
ACI Factored @ Heel	=	135 psf	
Footing Shear @ Toe	=	5.0 psi	OK
Footing Shear @ Heel	=	7.5 psi	OK
Allowable	=	75.0 psi	
Sliding Calcs (Vertical Component NOT Used)			
Lateral Sliding Force	=	271.9 lbs	
less 100% Passive Force	= -	2,188.1 lbs	
less 100% Friction Force	= -	940.0 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	0.0 lbs	OK

Stem Construction

		Top Stem	2nd
		Stem OK	Stem OK
Design Height Above Ftg	ft =	2.50	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Thickness	in =	8.00	8.00
Rebar Size	=	# 5	# 5
Rebar Spacing	in =	16.00	16.00
Rebar Placed at	=	Center	Center
Design Data			
fb/FB + fa/Fa	=	0.137	0.270
Total Force @ Section	lbs =	180.0	255.0
Moment....Actual	ft-l =	540.0	1,052.5
Moment.....Allowable	ft-l =	3,945.8	3,898.0
Shear.....Actual	psi =	6.0	7.6
Shear.....Allowable	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			
f'c	psi =	3,000.0	2,500.0
Fy	psi =	60,000.0	60,000.0

Load Factors

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

Cantilevered Retaining Wall

Description : Screen Wall

Footing Dimensions & Strengths

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
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top 2.00	@ Btm.=	3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	1,285	135 psf
Mu' : Upward	=	1,563	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 & 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

Item		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	275.6	1.17	321.6
Surcharge over Heel	=			
Toe Active Pressure	=	-183.8	1.17	-214.4
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=	180.0	6.50	1,170.0
Total	=	271.9	**O.T.M.** =	1,277.2

Resisting/Overturning Ratio = **3.71**
Vertical Loads used for Soil Pressure = 2,366.7 lbs

		.....RESISTING..... Force lbs	Distance ft	Moment ft-lb
Soil Over Heel	=	458.3	3.17	1,451.4
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	458.3	0.83	381.9
Surcharge Over Toe	=			
Stem Weight(s)	=	850.0	2.00	1,700.0
Earth @ Stem Transitions	=			
Footing Weight	=	600.0	2.00	1,200.0
Key Weight	=		2.00	
Vert. Component	=			
Total =		2,366.7 lbs	**R.M. =**	4,733.3

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