LEIGH & O'KANE L.L.C. – STRUCTURAL CALCULATIONS FOR

ADVANCED AESTHETIC CENTER

6 SW 2ND ST., LEE'S SUMMIT, MO 64063

Wayne N. Hess, P.E. & Reagan Holden, E.I.

6-15-2022

Leigh & O'Kane
250 NE Mulberry St. | Suite 201
Lee's Summit, MO 64086

250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Table of Contents

GENERAL DESIGN CONSIDERATIONS

The calcuations provided are for a remodel of an existing building for Advanced Aesthetic Center in Lee's Summit, Missouri. The structural design is based on the existing building plans and architectural design given to Leigh + O'Kane by Guy Gronberg Architects. The remodel involves the removal of existing load bearing walls and is expected to affect the existing lateral resisting system. The structural design for the remodel consists of replacing two load bearing walls with LVL headers on steel columns and adding five shear walls to take the lateral load in the east/west direction. The lateral in the north/south direction is assumed to be resisted by the existing exterior masonry walls. Where the existing foundation system is not sufficient to support the new load bearing elements, new footings and sections of thickened slab will be added.

REFERENCED DESIGN STANDARDS

- International Building Code 2018 Edition – IBC 2018
- AISC Steel Construction Manual – Fifteenth Edition
- Minimum Design Loads for Building and Other Structures – ASCE 7-16
- Building Code Requirements for Structural Concrete – ACI 318-14
- National Design Specification for Wood Construction – NDS 2018

DESIGN CRITERIA

Building Classifications

Risk Category	II
Snow Importance Factor, I_s	1.0
Ice Importance Factor – Wind, I_w	1.0
Seismic Importance Factor, I_e	1.0

Slab on Grade Floor Loads

Live Load	100 psf

Roof Loads

Dead Load	20 psf
Live Load	20 psf

Snow Loads

Ground Snow Load, P_g	20 psf
Flat Roof Snow Load, P_f	14 psf
Minimum Snow Load, P_m	20 psf
Thermal Factor, C_t	1.0
Slope Factor, C_s	1.0
Drift	Per Code

LEOK
LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Made by: WNH/RAH	Date: 06/15/2022	Job No. 22-19
Revision:	Date:	Sheet No. 2
Project Name: Advanced Aesthetic Center		

Wind Loads

Basic Wind Speed	109 MPH
Exposure Category	C
Internal Pressure Coefficient, GC_{pi}	+/- 0.18

Seismic Loads

S_s	0.1
S_1	0.068
S_{DS}	0.106
S_{D1}	0.109
Site Class	D
Seismic Design Category	B
Seismic Analysis Procedure	Equivalent Lateral Force Procedure
Seismic Force Resisting System	Wood walls sheathed with shear panels of all other materials
Design Base Shear	C_sW
Design Response Coefficient, C_s	0.053
Response Modification Coefficient, R	2

Rain Loads

60-min Duration/100 Year Rain Intensity, i	3.52 in/h
15-min Duration/100 Year Rain Intensity, i	7.49 in/h

LEOK
LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

WIND LOADING (ASCE7-16)

DIRECTIONAL PROCEDURE FOR ENCLOSED MONOSLOPE BUILDINGS

Building Data

type of roof		Monoslope	
building length	L	121.3	ft
building width	W	40	ft
height to eaves	h_e	12	ft
roof pitch		0	°
mean roof height	h	12.00	ft
height to top of parapet	h_p	17	ft

General Wind Load Requirements — ASCE 7-16

Risk Category			II	
Wind Speed	(Figure 26.5-1B)		109	mph
Directionaity Factor, K_d	(Table 26.6-1)		0.85	
Exposure Category	(26.7.3)		C	
Topographic Factor, K_{zt}	(26.8.2)		1.00	
Ground Elevation, K_e	(26.9)		1.00	
Gust Effect Factor, G	(26.11.1)		0.85	
Internal pressure coef, GC_{pi}	(Table 26.13-1)	+/-	0.18	
Velocity pressure coef, K_h	(Table 26.10-1)		0.85	
Velocity pressure, q_h	$q_h = 0.00256 \times K_h \times K_{zt} \times K_d \times V^2$		21.98	psf
Velocity pressure coef, K_p	(Table 26.10-1)		0.87	
Velocity pressure, q_p	$q_p = 0.00256 \times K_p \times K_{zt} \times K_d \times V^2$		22.49	psf

External Pressure Coefficient — (Figure 27.3-1)

WIND ON NARROW FACE	L/B	C_p	$q\,G\,C_p$	$p_1 = q\,(GC_p - GC_{pi})$	$p_2 = q\,(GC_p - GC_{pi})$
Windward Wall	3.03	0.8	14.94	18.9	11.0
Leeward Wall	3.03	-0.25	-4.64	-0.7	-8.6
Sidewall	3.03	-0.70	-13.08	-9.1	-17.0
Total Lateral Pressure on Building = windward - leeward pressure =					19.6 psf

WIND ON WIDE FACE	L/B	C_p	$q\,G\,C_p$	$p_1 = q\,(GC_p - GC_{pi})$	$p_2 = q\,(GC_p - GC_{pi})$
Windward Wall	0.33	0.8	14.94	18.9	11.0
Leeward Wall	0.33	-0.50	-9.34	-5.4	-13.3
Sidewall	0.33	-0.70	-13.08	-9.1	-17.0
Total Lateral Pressure on Building = windward - leeward pressure =					24.3 psf

Parapet Load (27.3.4)

	GC_{pn}	$q_p\,(GC_{pn})$	
Windward Parapet	1.5	33.74	psf
Leeward Parapet	-1	-22.49	psf
		56.23	**psf**

Uplift (27.3.4)

Wind on Narrow Face	h/L < 0.5	C_p	$q\,G\,C_p$	$q_i\,(GC_{pi})$	p (psf)
0	6	-0.9	-16.81	3.96	-20.77
6	12	-0.9	-16.81	3.96	-20.77
12	24	-0.5	-9.34	3.96	-13.29
24	121	-0.3	-5.60	3.96	-9.56
	Maximum Uplift Pressure			**20.77**	**psf**

Wind on Wide Face	h/L < 0.5	C_p	$q\,G\,C_p$	$q_i\,(GC_{pi})$	p (psf)
0	6	-0.9	-16.81	3.96	-20.77
6	12	-0.9	-16.81	3.96	-20.77
12	24	-0.5	-9.34	3.96	-13.29
24	40	-0.3	-5.60	3.96	-9.56
	Maximum Uplift Pressure			**20.77**	**psf**

250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Made by: WNH/RAH	Date: 06/15/2022	Job No. 22-19
Revision:	Date:	Sheet No. 5
Project Name: Advanced Aesthetic Center		

SNOW LOADING (ASCE7-16)

SNOW LOAD SUMMARY

BALANCED

Sloped Roof Snow Load	14 psf
Snow Load + Rain Surcharge	19 psf
Minimum Snow Load	20 psf

UNBALANCED

Windward Load	0.00 psf
Leeward Load	0 psf
Leeward Surcharge	0.00 psf
Leeward Surcharge Length	0.00 ft

DRIFT

Maximum Drift Surcharge	24.17 psf
Width of Drift	5.82 ft

SNOW LOAD		**ASCE 7-16 Ch. 7**
Roof Type	Single Slope	
Roof Slope	0 °	
Eave to Ridge Distance, W	40 ft	
Ground Snow Load, p_g	20 psf	Figure 7.2-1
Snow Importance Factor, I_s	1.0	Table 1.5-2
Exposure Factor, C_e	1.0	Table 7.3-1
Thermal Factor, C_t	1.0	Table 7.3-2
Slope Factor, C_s	1.0	Figure 7.4-1
Upper Roof Length, $l_{u,l}$	0 ft	
Lower Roof Length, $l_{u,w}$	40 ft	
Roof Height Difference, h	5 ft	

BALANCED SNOW LOAD		
Flat Roof Snow Load, p_f	14 psf	Equation 7.3-1
Sloped Roof Snow Load, p_s	**14 psf**	Equation 7.4-1
Minimum Snow Load, p_m	**20 psf**	Section 7.3.4

DRIFT LOAD

Unit Weight of Snow, γ	16.6 pcf	Equation 7.7-1
Height of Balanced Snow, h_b	0.84 ft	p_s / γ
Height from h_b to Upper Roof, h_c	4.16 ft	
h_c / h_b	4.93	
Drift Load Required?	**YES**	Section 7.7.1
Height of Leeward Drift, $h_{d,l}$	-1.50 ft	Figure 7.6-1
Height of Windward Drift, $h_{d,w}$	1.46 ft	Figure 7.6-1
Maximum Drift Height, h_d	1.46 ft	
Max. Drift Surcharge Load, p_d	**24.17 psf**	$h_d * \gamma$
Drift Width, w	**5.82 ft**	Section 7.7.1

RAIN ON SNOW SURCHARGE Section 7.10

Required?	YES	
Surcharge Load	5 psf	
Adjusted Balanced Snow Load	**19 psf**	

250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Made by: WNH/RAH	Date: 06/15/2022	Job No. 22-19
Revision:	Date:	Sheet No. 8
Project Name: Advanced Aesthetic Center		

EXISTING ROOF JOIST CHECK

LOADS

Dead Load, *D*	10	psf
Live Load, *L*	0	psf
Roof Live Load, L_r	20	psf
Snow Load, *S*	36	psf
Rain Load, *R*	0	psf
Downward Wind Load, *W*	0	psf
Wind Uplift, *W*	21	psf
Downward Earthquake Load, *E*		psf
Earthquake Uplift, *E*		psf

ASD LOAD COMBINATIONS

D	10	psf
D + L	30	psf
D + (Lr or S or R)	46	psf
D + 0.75L + 0.75(Lr or S or R)	37.00	psf
D + (0.6W or 0.7E)	10	psf
D + 0.75L + 0.75(0.6W) + 0.75(Lr or S or R)	37.00	psf
D + 0.75L + 0.75(0.7E) + 0.75S	37.00	psf
0.6D + 0.6W	-6.6	psf
0.6D + 0.7E	6	psf

MAXIMUM LOADS

Maximum Downward Load	**46**	**psf**
Maximum Uplift	**-6.6**	**psf**

LEOK
LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

BEAM INFORMATION

SECTION PROPERTIES

Nominal Size		2x10	
Width	b	1.5	in
Depth	d	9.25	in
Cross Section Area	A	13.875	in^2
Section Modulus	S_{xx}	21.39	in^3
Moment of Inertia	I_{xx}	98.9	in^4

WOOD PROPERTIES (NDS 2018 SUPPLEMENT TABLE 4A)

Wood Type		SPF No 2	
Bending Stress	F_b	875	psi
Shear Stress	F_v	135	psi
Compression Stress Perpendicular to Grain	$F_{c\perp}$	425	psi
Modulus of Elasticity	E	1400000	psi
Modulus of Elasticity for Stability	E_{min}	510000	psi
Specific Gravity	G	0.42	

BEAM DIMENSIONS

Span Length	L	18.25	ft
Top Unbraced Length	l_u	0	ft
Top Effective Length	l_e	0.00	ft
Bottom Unbraced Length	l_u	18.25	ft
Bottom Effective Length	l_e	32.06	ft
Bearing Length	l_b	1.5	in
Tributary Width		1.33	ft

LEOK
LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Made by: WNH/RAH	Date: 06/15/2022	Job No. 22-19
Revision:	Date:	Sheet No. 10
Project Name: Advanced Aesthetic Center		

DOWNWARD LOAD

BEAM LOADS

Superimposed Load		61.3333	plf
Self Weight		2.53	plf
Total Load	w	**63.86**	**plf**

BENDING — NDS 2018

$F_b' = F_b C_D C_M C_t C_L C_F C_{fu} C_i C_r$				**Table 4.3.1**
Reference Bending Design Value	F_b	875	psi	
Load Duration Factor	C_D	1.15		Table 2.3.2
Wet Service Factor	C_M	1		Table 4A
Temperature Factor	C_t	1		Table 2.3.3
Beam Stability Factor	C_L	1.00		Section 3.3.3
Size Factor	C_F	1.3		Table 4A
Flat Use Factor	C_{fu}	1		Table 4A
Incising Factor	C_i	1		Table 4.3.8
Repetitive Member Factor	C_r	1.15		Table 4A
Adjusted Bending Design Value	F_b'	1504.3	psi	

Applied Loads

Moment on Beam	$M = wL^2 / 8$	2658.6	lb-ft
Actual Bending Stress	$f_b = M / S_{xx}$	1491.5	psi
Actual to Allowable Ratio	f_b / F_b'	**99%**	

SHEAR

$F_v' = F_v C_D C_M C_t C_i$				**Table 4.3.1**
Reference Shear Design Value	F_v	135	psi	
Load Duration Factor	C_D	1.15		
Wet Service Factor	C_M	1		Table 4A
Temperature Factor	C_t	1		
Incising Factor	C_i	1		
Adjusted Shear Design Value	F_v'	155.25	psi	

Applied Loads

Shear on Beam	$V = wL / 2$	582.71	lb-ft
Actual Shear Stress	$f_v = 1.5V / A$	63.00	psi
Actual to Allowable Ratio	f_v / F_v'	**41%**	

LEOK LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

BEARING

$F_{c\perp}' = F_{c\perp} * C_M * C_t * C_i * C_b$			Table 4.3.1
Reference Bearing Design Value	$F_{c\perp}$	425 psi	
Wet Service Factor	C_M	1	
Temperature Factor	C_t	1	
Incising Factor	C_i	1	
Bearing Length Factor	C_b	1.25	
Adjusted Bearing Design Value		531.25 psi	

Applied Loads

Actual Bearing Stress	$f_{c\perp} = V / (b * l_b)$	258.98 psi	
Actual to Allowable Ratio	f_v / F_v'	**49%**	

DEFLECTION — IBC 2018 Table 1604.3

$$\Delta_{max} = \frac{5wl^4}{384EI}$$

$E' = E * C_M * C_t * C_i$			Table 4.3.1
Reference Modulus of Elasticity	E	1400000 psi	
Wet Service Factor	C_M	1	
Temperature Factor	C_t	1	
Incising Factor	C_i	1	
Adjusted Modulus of Elasticity	E'	1400000 psi	

Live Load Deflection

Live Load	w_L	26.6667 plf
Live Load Deflection	Δ_L	0.481 in
Allowable Live Load Deflection	L / 180	1.217 in
		39%

Snow or Wind Load Deflection

S or W Load	w_S	48 plf
Wind/Snow Deflection	Δ_S	0.865 in
Allowable Wind/Snow Deflection	L / 180	1.217 in
		71%

Dead + Live Load Deflection

D + L Load	w_{D+L}	40 plf
Dead + Live Deflection	Δ_{D+L}	0.721 in
Allowable Dead + Live Deflection	L / 120	1.825 in
		39%

LEOK
LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

UPLIFT

BEAM LOADS

Uplift	-8.8 plf
Self Weight	2.53 plf
Total Load, w	**-6.27 plf**

BENDING — NDS 2018

$F_b' = F_b\,C_D\,C_M\,C_t\,C_L\,C_F\,C_{fu}\,C_i\,C_r$			Table 4.3.1
Reference Bending Design Value	F_b	875 psi	
Load Duration Factor	C_D	1.6	Table 2.3.2
Wet Service Factor	C_M	1	Table 4A
Temperature Factor	C_t	1	Table 2.3.3
Beam Stability Factor	C_L	0.18	Section 3.3.3
Size Factor	C_F	1.3	Table 4A
Flat Use Factor	C_{fu}	1	Table 4A
Incising Factor	C_i	1	Table 4.3.8
Repetitive Member Factor	C_r	1.15	Table 4A
Adjusted Bending Design Value	F_b'	382.7 psi	

Applied Loads

Moment on Beam	$M = wL^2 / 8$	261.2 lb-ft
Actual Bending Stress	$f_b = M / S_{xx}$	146.6 psi
Actual to Allowable Ratio	f_b / F_b'	**38%**

LEOK
LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

BEAM STABILITY FACTOR

DOWNWARD LOAD

Slenderness Ratio for Bending, R_B	R_B	0.00	OK
$E'_{min} = E_{min}\, C_M\, C_t\, C_i$			
Reference Modulus of Elasticity	E_{min}	510 ksi	
Wet Service Factor	C_M	1	
Temperature Factor	C_t	1	
Incising Factor	C_i	1	
Modulus of Elasticity for Stability	E'_{min}	510 ksi	
Euler Buckling for Bending	$F_{bE} = 1.20\, E'_{min} / R_B^2$	1000 ksi	
$F_b{*} = F_b\, C_D\, C_M\, C_t\, C_F\, C_i\, C_r$		1504.34 psi	
$F_{bE} / F_b{*}$		664.74	
Beam Stability Factor	C_L	**1.000**	

UPLIFT

Slenderness Ratio for Bending, R_B	R_B	39.77	OK
$E'_{min} = E_{min}\, C_M\, C_t\, C_i$			
Reference Modulus of Elasticity	E_{min}	510 ksi	
Wet Service Factor	C_M	1	
Temperature Factor	C_t	1	
Incising Factor	C_i	1	
Modulus of Elasticity for Stability	E'_{min}	510 ksi	
Euler Buckling for Bending	$F_{bE} = 1.20\, E'_{min} / R_B^2$	0.39 ksi	
$F_b{*} = F_b\, C_D\, C_M\, C_t\, C_F\, C_i\, C_r$		2093.00 psi	
$F_{bE} / F_b{*}$		0.1849	
Beam Stability Factor	C_L	**0.183**	

250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Made by: WNH/RAH	Date: 06/15/2022	Job No. 22-19
Revision:	Date:	Sheet No. 14
Project Name: Advanced Aesthetic Center		

HEADER TO REPLACE WALL

WOOD MEMBER ANALYSIS & DESIGN (NDS 2018)

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

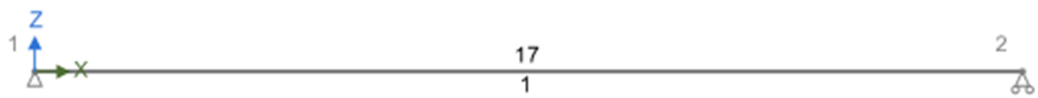
Tedds calculation version 2.2.13

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (ft) - Microllam(2.0E LVL) - 2/1.75"x16"

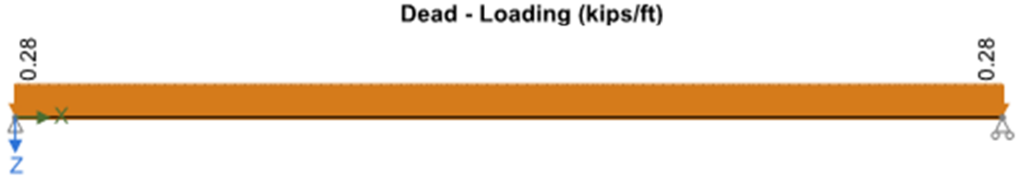


Span	Length (ft)	Section	Start Support	End Support
1	17	2/1.75"x16"	Pinned	Roller Pin X
2/1.75"x16": Area 56 in^2, Inertia Major 1195 in^4, Inertia Minor 57 in^4, Shear area parallel to Minor 47 in^2, Shear area parallel to Major 47 in^2				
Microllam(2.0E LVL): Density 42 lbm/ft^3, Youngs 2000 ksi, Shear 125 ksi, Thermal 0 °C^{-1}				

Loading

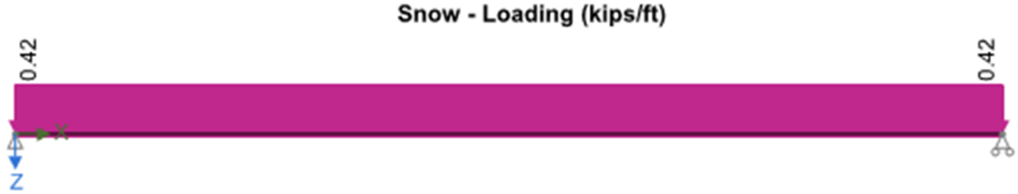
Self weight included

Dead - Loading (kips/ft)



Roof Live - Loading (kips/ft)

Snow - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Roof Live	Snow
1.0D (Strength)	1.00	1.00		
1.0D + 1.0L (Strength)	1.00	1.00		
1.0D + 1.0Lr (Strength)	1.00	1.00	1.00	
1.0D + 1.0S (Strength)	1.00	1.00		1.00
1.0D + 0.75L + 0.75Lr (Strength)	1.00	1.00	0.75	
1.0D + 0.75L + 0.75S (Strength)	1.00	1.00		0.75
1.0D + 0.6W (Strength)	1.00	1.00		
1.0D + 0.7E (Strength)	1.00	1.00		
1.0D + 0.75L + 0.75Lr + 0.45W (Strength)	1.00	1.00	0.75	
1.0D + 0.75L + 0.75S + 0.45W (Strength)	1.00	1.00		0.75
1.0D + 0.75L + 0.75S + 0.525E (Strength)	1.00	1.00		0.75
0.6D + 0.6W (Strength)	0.60	0.60		
0.6D + 0.7E (Strength)	0.60	0.60		

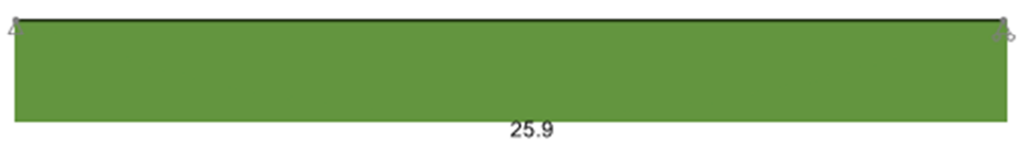
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	0.28 kips/ft
Beam	Roof Live	UDL	GlobalZ	0.28 kips/ft
Beam	Snow	UDL	GlobalZ	0.42 kips/ft

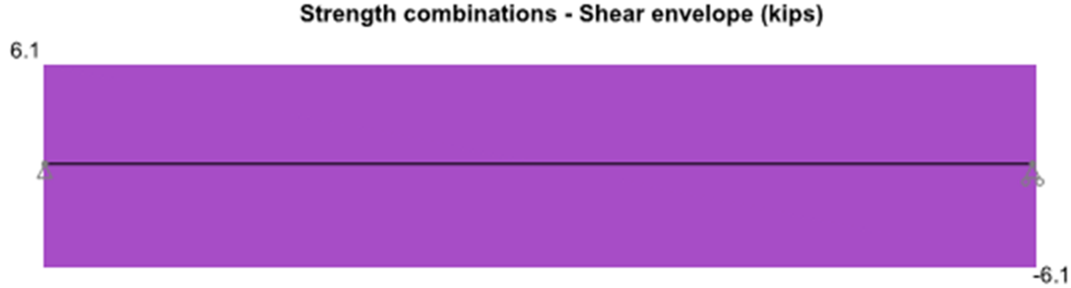
Results

Forces

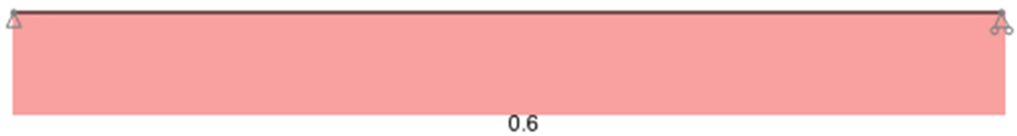
Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Strength combinations - Deflection envelope (in)



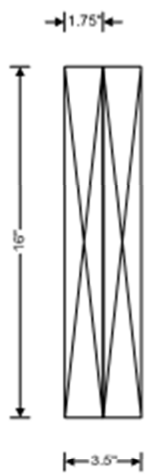
Beam - Span 1

Member details

Service condition	Dry
Load duration - Table 2.3.2	Ten years

Composite section details

Number of sawn lumber sections in member	N = **2**
Breadth of sections	b = **1.75** in
Depth of sections	d = **16** in
Material	**Microllam, 2.0E LVL grade**



2/1.75"x16" composite sections
Cross-sectional area, A, 56 in^2
Section modulus, S_x, 149.3 in^3
Section modulus, S_y, 32.7 in^3
Second moment of area, I_x, 1194.7 in^4
Second moment of area, I_y, 57.2 in^4
Radius of gyration, r_x, 4.619 in
Radius of gyration, r_y, 1.01 in
Microllam, x, 2.0E LVL grade
Bending about x-x axis, F_{bx}, 2600 psi
Bending about y-y axis, F_{by}, 2690 psi
Shear parallel to grain, bending about x-x axis, F_{vx}, 285 psi
Shear parallel to grain, bending about y-y axis, F_{vy}, 190 psi
Compression parallel to grain, F_c, 2510 psi
Compression perpendicular to grain, F_{c_perpx}, 750 psi
Compression perpendicular to grain, F_{c_perpy}, 480 psi
Tension parallel to grain, F_t, 1895 psi
Modulus of elasticity, E, 2000000 psi
Minimum modulus of elasticity, E_{min}, 1016535 psi
Density, ρ, 42 lbm/ft^3

Span details

Bearing length	L_b = **4** in

Consider Combination 4 - 1.0D + 1.0S (Strength)

Adjustment factors - Table 8.3.1

Load duration factor - Table 2.3.2	C_D = **1.15**
Volume factor maj.axis bending - Code evaluation	$C_{Vx} = (12 \text{ in} / d)^{0.136}$ = **0.962**
Time dependent deformation factor - cl.3.5.2	K_{cr} = **1.5**

Check design at start of span

Bending members - Shear - cl.3.4

Design shear force	V_x = **6089** lb
Design shear stress - Table 8.3.1	$F_{v,x}' = F_{vx} \times C_D$ = **328** lb/in^2

Actual shear stress - eq.3.4-2 — $f_{v,x} = 3 \times V_x / (2 \times N \times b \times d) = \mathbf{163}$ lb/in^2

$f_{v,x} / F_{v,x}' = \mathbf{0.498}$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression — $R_x = \mathbf{6089}$ lb

Design bearing stress - Table 8.3.1 — $F_{c\_perp,x}' = F_{c\_perpx} = \mathbf{750}$ lb/in^2

Actual bearing stress — $f_{c\_perp,x} = R_x / (N \times b \times L_b) = \mathbf{435}$ lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}' = \mathbf{0.580}$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

<u>**Check design 8ft 6in along span**</u>

Bending members - Flexure - cl.3.3

Design bending moment — $M_x = \mathbf{25878}$ lb_ft

Design bending stress - Table 8.3.1 — $F_{b,x}' = F_{bx} \times C_D \times C_{Vx} = \mathbf{2875}$ lb/in^2

Actual bending stress - eq.3.3-2 — $f_{b,x} = M_x / S_x = \mathbf{2079}$ lb/in^2

$f_{b,x} / F_{b,x}' = \mathbf{0.723}$

PASS - Design bending stress exceeds actual bending stress

<u>**Check design 8ft 6in along span**</u>

Bending members - Deflection - cl.3.5

Instantaneous deflection — $\delta_x = \mathbf{0.617}$ in

Final deflection — $\delta_{x,Final} = K_{cr} \times \delta_x = \mathbf{0.925}$ in

Allowable deflection — $\delta_{x,Allowable} = L_{m1\_s1} / 180 = \mathbf{1.133}$ in

$\delta_{x,Final} / \delta_{x,Allowable} = \mathbf{0.816}$

PASS - Allowable deflection exceeds final deflection

250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Made by: WNH/RAH	Date: 06/15/2022	Job No. 22-19
Revision:	Date:	Sheet No. 18
Project Name: Advanced Aesthetic Center		

HEADER OVER HALLWAY

WOOD MEMBER ANALYSIS & DESIGN (NDS 2018)

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

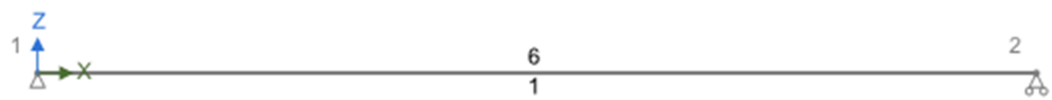
Tedds calculation version 2.2.13

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (ft) - Microllam(2.0E LVL) - 2/1.75"x16"

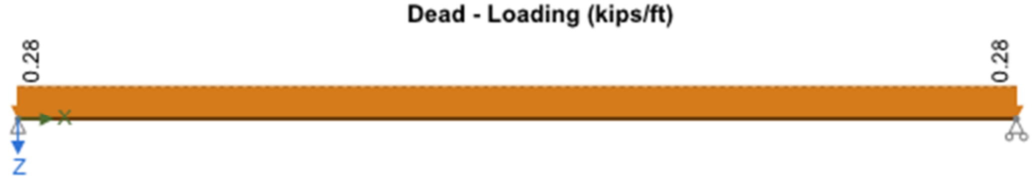


Span	Length (ft)	Section	Start Support	End Support
1	6	2/1.75"x16"	Pinned	Roller Pin X
2/1.75"x16": Area 56 in^2, Inertia Major 1195 in^4, Inertia Minor 57 in^4, Shear area parallel to Minor 47 in^2, Shear area parallel to Major 47 in^2				
Microllam(2.0E LVL): Density 42 lbm/ft^3, Youngs 2000 ksi, Shear 125 ksi, Thermal 0 °C^{-1}				

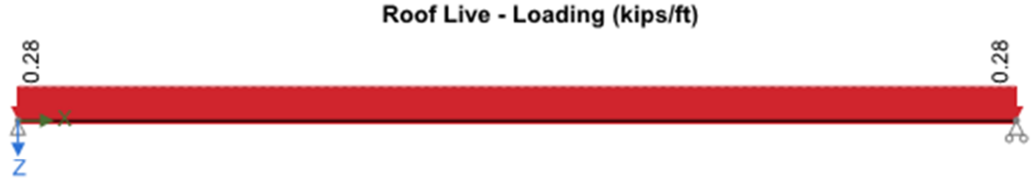
Loading

Self weight included

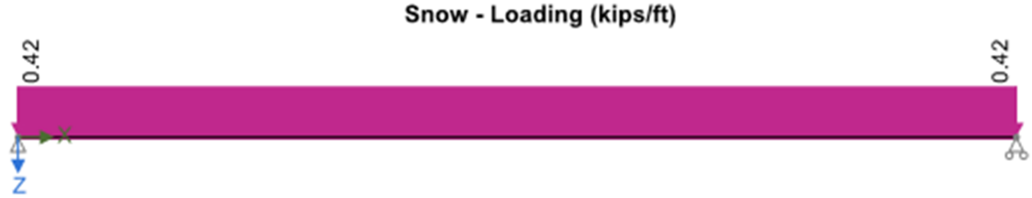
Dead - Loading (kips/ft)



Roof Live - Loading (kips/ft)



Snow - Loading (kips/ft)



LEOK
LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Load combination factors

Load combination	Self Weight	Dead	Roof Live	Snow
1.0D (Strength)	1.00	1.00		
1.0D + 1.0L (Strength)	1.00	1.00		
1.0D + 1.0Lr (Strength)	1.00	1.00	1.00	
1.0D + 1.0S (Strength)	1.00	1.00		1.00
1.0D + 0.75L + 0.75Lr (Strength)	1.00	1.00	0.75	
1.0D + 0.75L + 0.75S (Strength)	1.00	1.00		0.75
1.0D + 0.6W (Strength)	1.00	1.00		
1.0D + 0.7E (Strength)	1.00	1.00		
1.0D + 0.75L + 0.75Lr + 0.45W (Strength)	1.00	1.00	0.75	
1.0D + 0.75L + 0.75S + 0.45W (Strength)	1.00	1.00		0.75
1.0D + 0.75L + 0.75S + 0.525E (Strength)	1.00	1.00		0.75
0.6D + 0.6W (Strength)	0.60	0.60		
0.6D + 0.7E (Strength)	0.60	0.60		

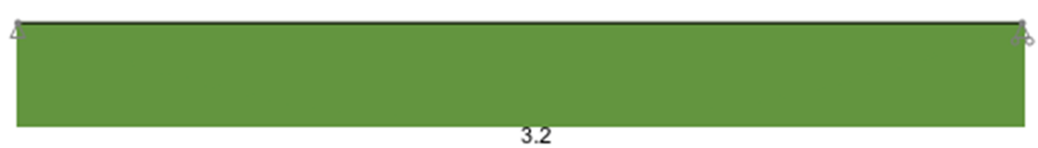
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	0.28 kips/ft
Beam	Roof Live	UDL	GlobalZ	0.28 kips/ft
Beam	Snow	UDL	GlobalZ	0.42 kips/ft

Results

Forces

Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)

Strength combinations - Deflection envelope (in)

Beam - Span 1

Member details

Service condition — Dry

Load duration - Table 2.3.2 — Ten years

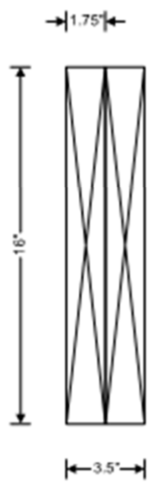
Composite section details

Number of sawn lumber sections in member — N = **2**

Breadth of sections — b = **1.75** in

Depth of sections — d = **16** in

Material — **Microllam, 2.0E LVL grade**



2/1.75"x16" composite sections
Cross-sectional area, A, 56 in^2
Section modulus, S_x, 149.3 in^3
Section modulus, S_y, 32.7 in^3
Second moment of area, I_x, 1194.7 in^4
Second moment of area, I_y, 57.2 in^4
Radius of gyration, r_x, 4.619 in
Radius of gyration, r_y, 1.01 in
Microllam, x, 2.0E LVL grade
Bending about x-x axis, F_{bx}, 2600 psi
Bending about y-y axis, F_{by}, 2690 psi
Shear parallel to grain, bending about x-x axis, F_{vx}, 285 psi
Shear parallel to grain, bending about y-y axis, F_{vy}, 190 psi
Compression parallel to grain, F_c, 2510 psi
Compression perpendicular to grain, F_{c_perpx}, 750 psi
Compression perpendicular to grain, F_{c_perpy}, 480 psi
Tension parallel to grain, F_t, 1895 psi
Modulus of elasticity, E, 2000000 psi
Minimum modulus of elasticity, E_{min}, 1016535 psi
Density, ρ, 42 lbm/ft^3

Span details

Bearing length — L_b = **4** in

Consider Combination 4 - 1.0D + 1.0S (Strength)

Adjustment factors - Table 8.3.1

Load duration factor - Table 2.3.2 — C_D = **1.15**

Volume factor maj.axis bending - Code evaluation — $C_{Vx} = (12 \text{ in} / d)^{0.136}$ = **0.962**

Time dependent deformation factor - cl.3.5.2 — K_{cr} = **1.5**

Check design at start of span

Bending members - Shear - cl.3.4

Design shear force — V_x = **2149** lb

Design shear stress - Table 8.3.1 — $F_{v,x}' = F_{vx} \times C_D$ = **328** lb/in^2

Actual shear stress - eq.3.4-2 $f_{v,x} = 3 \times V_x / (2 \times N \times b \times d) =$ **58** lb/in^2

$f_{v,x} / F_{v,x}' =$ **0.176**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression $R_x =$ **2149** lb

Design bearing stress - Table 8.3.1 $F_{c\_perp,x}' = F_{c\_perpx} =$ **750** lb/in^2

Actual bearing stress $f_{c\_perp,x} = R_x / (N \times b \times L_b) =$ **154** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}' =$ **0.205**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 3ft along span

Bending members - Flexure - cl.3.3

Design bending moment $M_x =$ **3224** lb_ft

Design bending stress - Table 8.3.1 $F_{b,x}' = F_{bx} \times C_D \times C_{Vx} =$ **2875** lb/in^2

Actual bending stress - eq.3.3-2 $f_{b,x} = M_x / S_x =$ **259** lb/in^2

$f_{b,x} / F_{b,x}' =$ **0.090**

PASS - Design bending stress exceeds actual bending stress

Check design at end of span

Bending members - Shear - cl.3.4

Design shear force $V_x =$ **2149** lb

Design shear stress - Table 8.3.1 $F_{v,x}' = F_{vx} \times C_D =$ **328** lb/in^2

Actual shear stress - eq.3.4-2 $f_{v,x} = 3 \times V_x / (2 \times N \times b \times d) =$ **58** lb/in^2

$f_{v,x} / F_{v,x}' =$ **0.176**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression $R_x =$ **2149** lb

Design bearing stress - Table 8.3.1 $F_{c\_perp,x}' = F_{c\_perpx} =$ **750** lb/in^2

Actual bearing stress $f_{c\_perp,x} = R_x / (N \times b \times L_b) =$ **154** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}' =$ **0.205**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 3ft along span

Bending members - Deflection - cl.3.5

Instantaneous deflection $\delta_x =$ **0.015** in

Final deflection $\delta_{x,Final} = K_{cr} \times \delta_x =$ **0.023** in

Allowable deflection $\delta_{x,Allowable} = L_{m1\_s1} / 250 =$ **0.288** in

$\delta_{x,Final} / \delta_{x,Allowable} =$ **0.08**

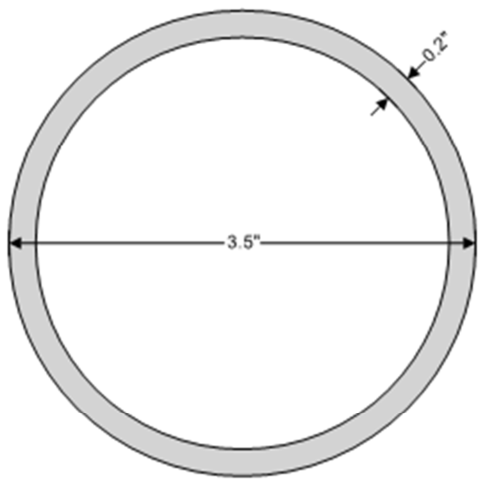
PASS - Allowable deflection exceeds final deflection

COLUMNS

STEEL COLUMN DESIGN

In accordance with AISC360-16 and the ASD method

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section **Pipe STD x3**

Design loading

Required axial strength P_r = **8** kips (Compression)

Moment about x axis at end 1 M_{x1} = **0.0** kips_ft

Moment about x axis at end 2 M_{x2} = **0.0** kips_ft

Maximum moment about x axis $M_x = \max(\mathrm{abs}(M_{x1}), \mathrm{abs}(M_{x2}))$ = **0.0** kips_ft

Moment about y axis at end 1 M_{y1} = **0.0** kips_ft

Moment about y axis at end 2 M_{y2} = **0.0** kips_ft

Maximum moment about y axis $M_y = \max(\mathrm{abs}(M_{y1}), \mathrm{abs}(M_{y2}))$ = **0.0** kips_ft

Maximum shear force parallel to y axis V_{ry} = **0.0** kips

Maximum shear force parallel to x axis V_{rx} = **0.0** kips

Material details

Steel grade **A53 Gr. B**

Yield strength F_y = **35** ksi

Ultimate strength F_u = **60** ksi

Modulus of elasticity E = **29000** ksi

Shear modulus of elasticity G = **11200** ksi

Unbraced lengths

For buckling about x axis L_x = **120** in

For buckling about y axis L_y = **120** in

For torsional buckling L_z = **120** in

Effective length factors

For buckling about x axis K_x = **1.00**

For buckling about y axis K_y = **1.00**

For torsional buckling K_z = **1.00**

Effective unbraced lengths

For buckling about x axis $L_{cx} = L_x \times K_x$ = **120** in

For buckling about y axis $L_{cy} = L_y \times K_y$ = **120** in

For torsional buckling $L_{cz} = L_z \times K_z$ = **120** in

Section classification

Section classification for local buckling (cl. B4)

Width to thickness ratio $\lambda = D_o / t$ = **17.413**

Compression

Limit for nonslender section $\lambda_{r\_c} = 0.11 \times E / F_y$ = **91.143**

The section is nonslender in compression

Slenderness

Member slenderness

Slenderness ratio about x axis $SR_x = L_{cx} / r_x$ = **102.6**

Slenderness ratio about y axis $SR_y = L_{cy} / r_y$ = **102.6**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 \times E / (SR_x)^2$ = **27.2** ksi

Flexural buckling stress $F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y$ = **20.4** ksi

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} \times A$ = **42.3** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 \times E / (SR_y)^2$ = **27.2** ksi

Flexural buckling stress $F_{cry} = (0.658^{F_y / F_{ey}}) \times F_y$ = **20.4** ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} \times A$ = **42.3** kips

Allowable compressive strength (cl. E1)

Safety factor for compression Ω_c = **1.67**

Allowable compressive strength $P_c = \min(P_{nx}, P_{ny}) / \Omega_c$ = **25.3** kips

PASS - The allowable compressive strength exceeds the required compressive strength

STUD WALL WOOD PROPERTIES

SECTION PROPERTIES

Nominal Size		2x6	
Width	b	1.5	in
Depth	d	5.5	in
Cross Section Area	A	8.25	in^2
Section Modulus	S_{xx}	7.56	in^3
Moment of Inertia	I_{xx}	20.8	in^4
Net Area w/ 1" Bolt Hole	A_n	6.8	in^2

WOOD PROPERTIES (NDS 2015 SUPPLEMENT TABLE 4A)

Wood Type		SPF No. 2	
Bending Stress	F_b	875	psi
Tension Stress Parallel to Grain	F_t	450	psi
Shear Stress	F_v	135	psi
Compression Stress Perpendicular to Grain	$F_{c\perp}$	425	psi
Compression Stress Parallel to Grain	F_c	1150	psi
Modulus of Elasticity	E	1400000	psi
Modulus of Elasticity for Stability	E_{min}	510000	psi
Specific Gravity	G	0.42	

ADJUSTED TENSION STRESS (NDS TABLE 4.3.1 AND SUPPLEMENT TABLE 4A)

$F'_t = F_t (C_D) (C_M) (C_t) (C_F) (C_i)$

Tension Stress Parallel to Grain	F_t	450	psi
Load Duration Factor	C_D	1.6	
Wet Service Factor	C_M	1	
Temperature Factor	C_t	1	
Size Factor	C_F	1.3	
Incising Factor	C_i	1	
Adjusted ASD Tension Design Value	**F'_t**	**936**	**psi**

ADJUSTED COMPRESSION STRESS (NDS TABLE 4.3.1 AND SUPPLEMENT TABLE 4A)

$F'_c = F_c (C_D) (C_M) (C_t) (C_F) (C_i) (C_P)$

Compression Stress Parallel to Grain	F_c	1150	psi
Load Duration Factor	C_D	1.15	
Wet Service Factor	C_M	1	
Temperature Factor	C_t	1	
Size Factor	C_F	1.1	
Incising Factor	C_i	1	
Column Stability Factor	C_P	0.503	
Adjusted ASD Compression Design Value	**F'_c**	**732.4**	**psi**

250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Column Stability Factor (NDS 3.7)

$C_p = \frac{1+(F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1+(F_{cE}/F_c^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}$		**0.503**
interaction coefficient	c	0.8
Euler Stress	$F_{cE} = \frac{0.822\ E_{min}'}{(\ell_e/d)^2}$	**880.9 psi**
Effective Length (major axis)	l_e	11.00 ft
Depth	d	5.5 in
Effective Length (minor axis)	l_e	1.00 ft
Width	b	1.5 in
Slenderness Ratio	l_e / d	24.00
Adjusted Modulus for Stability	$E'_{min} = E_{min}\ (C_M)\ (C_t)\ (C_i)\ (C_T)$	**617246 psi**
Modulus of Elasticity for Stability	E_{min}	510000 psi
Wet Service Factor	C_M	1
Temperature Factor	C_t	1
Incising Factor	C_i	1
Buckling Stiffness Factor	C_T	1.21
Upper Limit of Compression Design Value	$F^*_c = F_c\ (C_D)\ (C_M)\ (C_t)\ (C_F)\ (C_i)$	**1454.75 psi**

ADJUSTED COMPRESSION STRESS PERPENDICULAR TO GRAIN

$F'_{c\perp} = F_{c\perp}(C_M)\ (C_t)\ (C_i)\ (C_b)$

Reference Bearing Design Value	$F_{c\perp}$	425 psi
Wet Service Factor	C_M	1
Temperature Factor	C_t	1
Incising Factor	C_i	1
Bearing Length Factor	C_b	1
Adjusted ASD Compression Design Value	$F'_{c\perp}$	**425 psi**

ADJUSTED BENDING STRESS (NDS TABLE 4.3.1 AND SUPPLEMENT TABLE 4A)		
$F'_b = F_b (C_D) (C_M) (C_t) (C_L) (C_F) (C_{fu}) (C_i) (C_r)$		
Bending Stress	F_b	875 psi
Load Duration Factor	C_D	1.15
Wet Service Factor	C_M	1
Temperature Factor	C_t	1
Size Factor	C_F	1.3
Incising Factor	C_i	1
Repetitive Member Factor	C_r	1.15
Flat Use Factor	C_{fu}	1
Beam Stability Factor	C_L	0.730
Adjusted ASD Bending Design Value	F'_b	**1098.8 psi**

Beam Stability Factor (NDS 3.7)		
$C_L = \frac{1+(F_{bE}/F_b^*)}{1.9} - \sqrt{\left[\frac{1+(F_{bE}/F_b^*)}{1.9}\right]^2 - \frac{F_{bE}/F_b^*}{0.95}}$		**0.730**
Euler Bending Stress	$F_{bE} = \frac{1.20\, E_{min}'}{R_B^2}$	**1247.6 psi**
Slenderness Ratio	$R_B^2 = l_e\, d / b^2$	593.7
Unbraced Length (bending axis)	l_{ux}	132 in
Depth	d	5.5 in
Unbraced Length to Beam Depth Ratio	l_{ux} / d	24.00
Effective Length	l_e	242.88 in
Width	b	1.5 in
Partially Adjusted Bending Value	$F^*_b = F_b (C_D) (C_M) (C_t) (C_F) (C_i) (C_r)$	**1504.3 psi**

LOAD BEARING WALLS

LOAD BEARING WALL DESIGN

Wall Dimensions and Loads

stud size		2x6	
wood type		SPF No. 2	
stud spacing	s	24	in
trib width	B	18.25	ft
tributary area	$A_T = s*B$	36.5	ft^2
total roof load	TL	55	psf
lateral load on wall	p_{wall}	5	psf

Loads on Studs

axial load on stud	$P = TL*A_T$	2007.5	lb
distributed load on stud	$w_{stud} = p_{wall}*s$	10.00	plf
moment on stud	$M = w_{stud}\, h^2 / 8$	1815	lb-in
compression stress	$f_c = P / A$	243.3	psi
bending stress	$f_{bx} = M / S_{xx}$	240.0	psi

Stud Strength

allowable compression stress	F'_c	732.4	psi
euler buckling stress	F_{cEx}	880.9	psi
allowable bending stress	F'_{bx}	1098.8	psi

Interaction

compressive utilization ratio	f_c / F'_c	0.332
bending utilization ratio	f_{bx} / F'_{bx}	0.218
interaction	$\left(\frac{f_c}{F'_c}\right)^2 + \left(\frac{1}{1 - f_c/F_{cEx}}\right)\frac{f_{bx}}{F'_{bx}} \le 1.0$	0.412

LEOK
LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

SHEAR WALL 1

SHEAR WALL 1

Dimensions and Loads

Building Loads			
wind to eave load (LRFD)	W	427	plf
wind to eave load (ASD)	0.6W	256.2	plf
roof uplift (ASD)	0.6D + 0.6W	9.6	psf
max roof dead load	D_{roof}	20	psf
max Lr, S, or R load		35	psf
wall weight	D_{wall}	6	psf
gravity load trib width	B_v	15.6	ft
max dist. between stud walls in same line	d_w	5.5	ft
stud spacing	s	24	in
Wall Segments			
shear tributary width	B_h	23.5	ft
wall height	h	11	ft
length 1	b_1	8.00	ft
length 2	b_2	10.50	ft
length 3	b_3	8.00	ft
Segment Loads			
concentrated load	$P_{total} = 0.6\ W\ B_h$	6.02	k
induced unit shear	$v = P / b_{total}$	227.2	plf
wall 1 load	$P_1 = v\ b_1$	1.82	k
wall 2 load	$P_2 = v\ b_2$	2.39	k
wall 3 load	$P_3 = v\ b_3$	1.82	k
Tension in Chords	**0.6D + 0.6W**		
wall 1	$T_1 = P_1 h/b_1$ + Uplift - ½Wall Weight	3.43	k
wall 2	$T_2 = P_2 h/b_2$ + Uplift - ½Wall Weight	3.55	k
wall 3	$T_3 = P_3 h/b_3$ + Uplift - ½Wall Weight	3.43	k
Compression in Chords	**max of (D + 0.6W) and (D + 0.75S + 0.75*0.6W)**		
wall 1	$C_1 = P_1 h/b_1$ + Roof Load + ½Wall Weight	4.90	k
wall 2	$C_2 = P_2 h/b_2$ + Roof Load + ½Wall Weight	4.97	k
wall 3	$C_3 = P_3 h/b_3$ + Roof Load + ½Wall Weight	4.90	k

250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Shear Wall Sheathing Design (SDPWS2015 Table 4.3)

required unit shear capacity		**227.2**	**plf**
1/2" gypsum board, 4" fastener spacing, 16" stud spacing, blocked			
apparent stiffness	G_a	7.5	k/in
nominal capacity	v_w	300	plf
combined nominal capacity	v_{wc}	600	plf
ASD allowable capacity	v_{wca}	**300**	**plf**

Shear Wall Uplift Design (Simpson Strong-Tie)

Holdown Type		HDU5-SDS2.5	
allowable tension load		4.34	k
maximum uplift	T_{max}	3.55	k
required/allowable ratio		0.82	
deflection at allowable	Δ_a	0.115	in
wall 1 anchorage elongation	Δ_{a1}	0.091	in
wall 2 anchorage elongation	Δ_{a2}	0.094	in
wall 3 anchorage elongation	Δ_{a3}	0.091	in

Chord Design

tension stress in one stud	$f_t = T/A_n$	526.0	psi
allowable tension stress	F'_t	936	psi
number of studs required		1	
compression stress in one stud	$f_c = C/A$	602.7	psi
allowable compression stress	F'_c	425.0	psi
number of studs required		2	

Deflection

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$

induced unit shear	v	227.2	plf
wall height	h	11	ft
Modulus of Elasticity	E	1400000	psi
Cross Section Area	A	16.5	in^2
length	b_{min}	8.00	ft
apparent stiffness	G_a	7.5	k/in
deflection at allowable	Δ_a	0.094	in
shearwall deflection	δ_{sw}	**0.48**	**in**

DESIGN SUMMARY

Load Transfer

unit shear in diaphragm	**v**	**227.2 plf**
required uplift resistance	**T**	**3.55 k**

Shearwall

Sheathing	**1/2" gypsum board, 4" fastener spacing, 16" stud spacing, blocked**	
allowable unit shear		300 plf
induced unit shear		227 plf
required/allowable ratio		**0.76**

Holdown	**HDU5-SDS2.5**	
allowable tension load		4.34 k
maximum uplift		3.55 k
required/allowable ratio		**0.82**

Chords	**(2) 2x6 studs**	
allowable tension stress		936 psi
tension stress		263.0 psi
required/allowable ratio		**0.28**
allowable compression stress		425.0 psi
compression stress		301.3 psi
required/allowable ratio		**0.71**

Deflection		
shearwall deflection	δ_{sw}	**0.48 in**

LEOK
LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

SHEAR WALL 2

SHEAR WALL 2			
Dimensions and Loads			
Building Loads			
wind to eave load (LRFD)	W	427	plf
wind to eave load (ASD)	0.6W	256.2	plf
roof uplift (ASD)	0.6D + 0.6W	9.6	psf
max roof dead load	D_{roof}	20	psf
max Lr, S, or R load		35	psf
wall weight	D_{wall}	6	psf
gravity load trib width	B_v	17.1	ft
max dist. between stud walls in same line	d_w	5.5	ft
stud spacing	s	24	in
Wall Segments			
shear tributary width	B_h	21.6	ft
wall height	h	11	ft
length 1	b_1	13.00	ft
length 2	b_2	16.00	ft
length 3	b_3		ft
Segment Loads			
concentrated load	$P_{total} = 0.6\ W\ B_h$	5.53	k
induced unit shear	$v = P / b_{total}$	190.8	plf
wall 1 load	$P_1 = v\ b_1$	2.48	k
wall 2 load	$P_2 = v\ b_2$	3.05	k
wall 3 load	$P_3 = v\ b_3$	0.00	k
Tension in Chords	**0.6D + 0.6W**		
wall 1	$T_1 = P_1h/b_1$ + Uplift - ½Wall Weight	3.40	k
wall 2	$T_2 = P_2h/b_2$ + Uplift - ½Wall Weight	3.58	k
wall 3	$T_3 = P_3h/b_3$ + Uplift - ½Wall Weight	0.00	k
Compression in Chords	**max of (D + 0.6W) and (D + 0.75S + 0.75*0.6W)**		
wall 1	$C_1 = P_1h/b_1$ + Roof Load + ½Wall Weight	5.00	k
wall 2	$C_2 = P_2h/b_2$ + Roof Load + ½Wall Weight	5.09	k
wall 3	$C_3 = P_3h/b_3$ + Roof Load + ½Wall Weight	0.00	k

250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Shear Wall Sheathing Design (SDPWS2015 Table 4.3)

required unit shear capacity		**190.8**	**plf**
1/2" gypsum board, 4" fastener spacing, 16" stud spacing, blocked			
apparent stiffness	G_a	7.5	k/in
nominal capacity	v_w	300	plf
combined nominal capacity	v_{wc}	600	plf
ASD allowable capacity	v_{wca}	**300**	**plf**

Shear Wall Uplift Design (Simpson Strong-Tie)

Holdown Type		HDU5-SDS2.5	
allowable tension load		4.34	k
maximum uplift	T_{max}	3.58	k
required/allowable ratio		0.82	
deflection at allowable	Δ_a	0.115	in
wall 1 anchorage elongation	Δ_{a1}	0.090	in
wall 2 anchorage elongation	Δ_{a2}	0.095	in
wall 3 anchorage elongation	Δ_{a3}	0.000	in

Chord Design

tension stress in one stud	$f_t = T/A_n$	530.4	psi
allowable tension stress	F'_t	936	psi
number of studs required		1	
compression stress in one stud	$f_c = C/A$	617.3	psi
allowable compression stress	F'_c	425.0	psi
number of studs required		2	

Deflection

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$

induced unit shear	v	190.8	plf
wall height	h	11	ft
Modulus of Elasticity	E	1400000	psi
Cross Section Area	A	16.5	in^2
length	b_{min}	13.00	ft
apparent stiffness	G_a	7.5	k/in
deflection at allowable	Δ_a	0.095	in
shearwall deflection	δ_{sw}	**0.37**	**in**

LEOK
LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

DESIGN SUMMARY

Load Transfer

unit shear in diaphragm	**v**	**190.8 plf**
required uplift resistance	**T**	**3.58 k**

Shearwall

Sheathing	**1/2" gypsum board, 4" fastener spacing, 16" stud spacing, blocked**
allowable unit shear	300 plf
induced unit shear	191 plf
required/allowable ratio	**0.64**

Holdown	**HDU5-SDS2.5**
allowable tension load	4.34 k
maximum uplift	3.58 k
required/allowable ratio	**0.82**

Chords	**(2) 2x6 studs**
allowable tension stress	936 psi
tension stress	265.2 psi
required/allowable ratio	**0.28**
allowable compression stress	425.0 psi
compression stress	308.7 psi
required/allowable ratio	**0.73**

Deflection		
shearwall deflection	δ_{sw}	**0.37 in**

SHEAR WALL 3

SHEAR WALL 3

Dimensions and Loads

Building Loads			
wind to eave load (LRFD)	W	427	plf
wind to eave load (ASD)	0.6W	256.2	plf
roof uplift (ASD)	0.6D + 0.6W	9.6	psf
max roof dead load	D_{roof}	20	psf
max Lr, S, or R load		35	psf
wall weight	D_{wall}	6	psf
gravity load trib width	B_v	9	ft
max dist. between stud walls in same line	d_w	7	ft
stud spacing	s	24	in
Wall Segments			
shear tributary width	B_h	27	ft
wall height	h	11	ft
length 1	b_1	16.42	ft
length 2	b_2	9.00	ft
length 3	b_3		ft
Segment Loads			
concentrated load	$P_{total} = 0.6\ W\ B_h$	6.92	k
induced unit shear	$v = P / b_{total}$	272.1	plf
wall 1 load	$P_1 = v\ b_1$	4.47	k
wall 2 load	$P_2 = v\ b_2$	2.45	k
wall 3 load	$P_3 = v\ b_3$	0.00	k
Tension in Chords	**0.6D + 0.6W**		
wall 1	$T_1 = P_1h/b_1$ + Uplift - ½Wall Weight	3.73	k
wall 2	$T_2 = P_2h/b_2$ + Uplift - ½Wall Weight	3.59	k
wall 3	$T_3 = P_3h/b_3$ + Uplift - ½Wall Weight	0.00	k
Compression in Chords	**max of (D + 0.6W) and (D + 0.75S + 0.75*0.6W)**		
wall 1	$C_1 = P_1h/b_1$ + Roof Load + ½Wall Weight	4.69	k
wall 2	$C_2 = P_2h/b_2$ + Roof Load + ½Wall Weight	4.48	k
wall 3	$C_3 = P_3h/b_3$ + Roof Load + ½Wall Weight	0.00	k

LEOK
LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Made by: WNH/RAH	Date: 06/15/2022	Job No. 22-19
Revision:	Date:	Sheet No. 35
Project Name: Advanced Aesthetic Center		

Shear Wall Sheathing Design (SDPWS2015 Table 4.3)

required unit shear capacity		**272.1**	**plf**
1/2" gypsum board, 4" fastener spacing, 16" stud spacing, blocked			
apparent stiffness	G_a	7.5	k/in
nominal capacity	v_w	300	plf
combined nominal capacity	v_{wc}	600	plf
ASD allowable capacity	$\mathbf{v_{wca}}$	**300**	**plf**

Shear Wall Uplift Design (Simpson Strong-Tie)

Holdown Type		HDU5-SDS2.5	
allowable tension load		4.34	k
maximum uplift	T_{max}	3.73	k
required/allowable ratio		0.86	
deflection at allowable	Δ_a	0.115	in
wall 1 anchorage elongation	Δ_{a1}	0.099	in
wall 2 anchorage elongation	Δ_{a2}	0.095	in
wall 3 anchorage elongation	Δ_{a3}	0.000	in

Chord Design

tension stress in one stud	$f_t = T/A_n$	552.0	psi
allowable tension stress	F'_t	936	psi
number of studs required		1	
compression stress in one stud	$f_c = C/A$	569.1	psi
allowable compression stress	F'_c	425.0	psi
number of studs required		2	

Deflection

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$

induced unit shear	v	272.1	plf
wall height	h	11	ft
Modulus of Elasticity	E	1400000	psi
Cross Section Area	A	16.5	in^2
length	b_{min}	9.00	ft
apparent stiffness	G_a	7.5	k/in
deflection at allowable	Δ_a	0.099	in
shearwall deflection	$\boldsymbol{\delta_{sw}}$	**0.53**	**in**

LEOK
LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

DESIGN SUMMARY

Load Transfer

unit shear in diaphragm	**v**	**272.1**	**plf**
required uplift resistance	**T**	**3.73**	**k**

Shearwall

Sheathing	**1/2" gypsum board, 4" fastener spacing, 16" stud spacing, blocked**		
allowable unit shear		300	plf
induced unit shear		272	plf
required/allowable ratio		**0.91**	

Holdown	**HDU5-SDS2.5**		
allowable tension load		4.34	k
maximum uplift		3.73	k
required/allowable ratio		**0.86**	

Chords	**(2) 2x6 studs**		
allowable tension stress		936	psi
tension stress		276.0	psi
required/allowable ratio		**0.29**	
allowable compression stress		425.0	psi
compression stress		284.5	psi
required/allowable ratio		**0.67**	

Deflection			
shearwall deflection	δ_{sw}	**0.53**	**in**

250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Made by: WNH/RAH	Date: 06/15/2022	Job No. 22-19
Revision:	Date:	Sheet No. 37
Project Name: Advanced Aesthetic Center		

SHEAR WALL 4

SHEAR WALL 4

Dimensions and Loads

Building Loads			
wind to eave load (LRFD)	W	427	plf
wind to eave load (ASD)	0.6W	256.2	plf
roof uplift (ASD)	0.6D + 0.6W	9.6	psf
max roof dead load	D_{roof}	20	psf
max Lr, S, or R load		35	psf
wall weight	D_{wall}	6	psf
gravity load trib width	B_v	18	ft
max dist. between stud walls in same line	d_w	5.5	ft
stud spacing	s	24	in
Wall Segments			
shear tributary width	B_h	21	ft
wall height	h	11	ft
length 1	b_1	16.42	ft
length 2	b_2	16.00	ft
length 3	b_3		ft
Segment Loads			
concentrated load	$P_{total} = 0.6\ W\ B_h$	5.38	k
induced unit shear	$v = P / b_{total}$	166.0	plf
wall 1 load	$P_1 = v\ b_1$	2.72	k
wall 2 load	$P_2 = v\ b_2$	2.66	k
wall 3 load	$P_3 = v\ b_3$	0.00	k
Tension in Chords	**0.6D + 0.6W**		
wall 1	$T_1 = P_1h/b_1$ + Uplift - ½Wall Weight	3.42	k
wall 2	$T_2 = P_2h/b_2$ + Uplift - ½Wall Weight	3.40	k
wall 3	$T_3 = P_3h/b_3$ + Uplift - ½Wall Weight	0.00	k
Compression in Chords	**max of (D + 0.6W) and (D + 0.75S + 0.75*0.6W)**		
wall 1	$C_1 = P_1h/b_1$ + Roof Load + ½Wall Weight	5.05	k
wall 2	$C_2 = P_2h/b_2$ + Roof Load + ½Wall Weight	5.04	k
wall 3	$C_3 = P_3h/b_3$ + Roof Load + ½Wall Weight	0.00	k

250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Made by: WNH/RAH	Date: 06/15/2022	Job No. 22-19
Revision:	Date:	Sheet No. 38
Project Name: Advanced Aesthetic Center		

Shear Wall Sheathing Design (SDPWS2015 Table 4.3)

required unit shear capacity		**166.0**	**plf**
1/2" gypsum board, 4" fastener spacing, 16" stud spacing, blocked			
apparent stiffness	G_a	7.5	k/in
nominal capacity	v_w	300	plf
combined nominal capacity	v_{wc}	600	plf
ASD allowable capacity	v_{wca}	**300**	**plf**

Shear Wall Uplift Design (Simpson Strong-Tie)

Holdown Type		HDU5-SDS2.5	
allowable tension load		4.34	k
maximum uplift	T_{max}	3.42	k
required/allowable ratio		0.79	
deflection at allowable	Δ_a	0.115	in
wall 1 anchorage elongation	Δ_{a1}	0.091	in
wall 2 anchorage elongation	Δ_{a2}	0.090	in
wall 3 anchorage elongation	Δ_{a3}	0.000	in

Chord Design

tension stress in one stud	$f_t = T/A_n$	507.0	psi
allowable tension stress	F'_t	936	psi
number of studs required		1	
compression stress in one stud	$f_c = C/A$	612.6	psi
allowable compression stress	F'_c	425.0	psi
number of studs required		2	

Deflection

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$

induced unit shear	v	166.0	plf
wall height	h	11	ft
Modulus of Elasticity	E	1400000	psi
Cross Section Area	A	16.5	in^2
length	b_{min}	16.00	ft
apparent stiffness	G_a	7.5	k/in
deflection at allowable	Δ_a	0.091	in
shearwall deflection	δ_{sw}	**0.31**	**in**

DESIGN SUMMARY

Load Transfer

unit shear in diaphragm	**v**	**166.0 plf**
required uplift resistance	**T**	**3.42 k**

Shearwall

Sheathing	**1/2" gypsum board, 4" fastener spacing, 16" stud spacing, blocked**	
allowable unit shear		300 plf
induced unit shear		166 plf
required/allowable ratio		**0.55**

Holdown	**HDU5-SDS2.5**	
allowable tension load		4.34 k
maximum uplift		3.42 k
required/allowable ratio		**0.79**

Chords	**(2) 2x6 studs**	
allowable tension stress		936 psi
tension stress		253.5 psi
required/allowable ratio		**0.27**
allowable compression stress		425.0 psi
compression stress		306.3 psi
required/allowable ratio		**0.72**

Deflection		
shearwall deflection	δ_{sw}	**0.31 in**

SHEAR WALL 5

SHEAR WALL 5			
Dimensions and Loads			
Building Loads			
wind to eave load (LRFD)	W	427	plf
wind to eave load (ASD)	0.6W	256.2	plf
roof uplift (ASD)	0.6D + 0.6W	9.6	psf
max roof dead load	D_{roof}	20	psf
max Lr, S, or R load		35	psf
wall weight	D_{wall}	6	psf
gravity load trib width	B_v	9	ft
max dist. between stud walls in same line	d_w	5.5	ft
stud spacing	s	24	in
Wall Segments			
shear tributary width	B_h	29.5	ft
wall height	h	11	ft
length 1	b_1	16.42	ft
length 2	b_2	12.58	ft
length 3	b_3		ft
Segment Loads			
concentrated load	$P_{total} = 0.6\ W\ B_h$	7.56	k
induced unit shear	$v = P / b_{total}$	260.6	plf
wall 1 load	$P_1 = v\ b_1$	4.28	k
wall 2 load	$P_2 = v\ b_2$	3.28	k
wall 3 load	$P_3 = v\ b_3$	0.00	k
Tension in Chords	**0.6D + 0.6W**		
wall 1	$T_1 = P_1h/b_1$ + Uplift - ½Wall Weight	3.53	k
wall 2	$T_2 = P_2h/b_2$ + Uplift - ½Wall Weight	3.46	k
wall 3	$T_3 = P_3h/b_3$ + Uplift - ½Wall Weight	0.00	k
Compression in Chords	**max of (D + 0.6W) and (D + 0.75S + 0.75*0.6W)**		
wall 1	$C_1 = P_1h/b_1$ + Roof Load + ½Wall Weight	4.29	k
wall 2	$C_2 = P_2h/b_2$ + Roof Load + ½Wall Weight	4.17	k
wall 3	$C_3 = P_3h/b_3$ + Roof Load + ½Wall Weight	0.00	k

Shear Wall Sheathing Design (SDPWS2015 Table 4.3)

required unit shear capacity		**260.6**	**plf**
1/2" gypsum board, 4" fastener spacing, 16" stud spacing, blocked			
apparent stiffness	G_a	7.5	k/in
nominal capacity	v_w	300	plf
combined nominal capacity	v_{wc}	600	plf
ASD allowable capacity	$\mathbf{v_{wca}}$	**300**	**plf**

Shear Wall Uplift Design (Simpson Strong-Tie)

Holdown Type		HDU5-SDS2.5	
allowable tension load		4.34	k
maximum uplift	T_{max}	3.53	k
required/allowable ratio		0.81	
deflection at allowable	Δ_a	0.115	in
wall 1 anchorage elongation	Δ_{a1}	0.094	in
wall 2 anchorage elongation	Δ_{a2}	0.092	in
wall 3 anchorage elongation	Δ_{a3}	0.000	in

Chord Design

tension stress in one stud	$f_t = T/A_n$	523.4	psi
allowable tension stress	F'_t	936	psi
number of studs required		1	
compression stress in one stud	$f_c = C/A$	519.5	psi
allowable compression stress	F'_c	425.0	psi
number of studs required		2	

Deflection

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$

induced unit shear	v	260.6	plf
wall height	h	11	ft
Modulus of Elasticity	E	1400000	psi
Cross Section Area	A	16.5	in^2
length	b_{min}	12.58	ft
apparent stiffness	G_a	7.5	k/in
deflection at allowable	Δ_a	0.094	in
shearwall deflection	$\boldsymbol{\delta_{sw}}$	**0.47**	**in**

DESIGN SUMMARY

Load Transfer

unit shear in diaphragm	**v**	**260.6 plf**
required uplift resistance	**T**	**3.53 k**

Shearwall

Sheathing	**1/2" gypsum board, 4" fastener spacing, 16" stud spacing, blocked**	
allowable unit shear		300 plf
induced unit shear		261 plf
required/allowable ratio		**0.87**

Holdown	**HDU5-SDS2.5**	
allowable tension load		4.34 k
maximum uplift		3.53 k
required/allowable ratio		**0.81**

Chords	**(2) 2x6 studs**	
allowable tension stress		936 psi
tension stress		261.7 psi
required/allowable ratio		**0.28**
allowable compression stress		425.0 psi
compression stress		259.8 psi
required/allowable ratio		**0.61**

Deflection		
shearwall deflection	δ_{sw}	**0.47 in**

250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

BEARING AND UPLIFT RESISTANCE OF FOUNDATIONS

BEARING AND UPLIFT

Geometry and Applied Loads

max downward load		50	psf
max uplift		10.5	psf
soil bearing capacity	q	1500	psf
column tributary width		14	ft
wall tributary width		18	ft
max column trib length		11.5	ft
edge column trib length		8.75	ft
length of building		40	ft
total wall length		26	ft

Wall Loads

downward load	w_D	1384.6	plf
uplift	w_U	290.8	plf

Thickened Slab Under Wall

required bearing width	w_D / q	0.92	ft
required cross section area	w_U / 150 pcf	1.94	ft^2

Thickened Slab Design

Cross Sectional Dimensions

thickness	12	in
bearing width	24	in
footing bearing area	2	ft^2

24 in x 12 in

LEOK LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Column Loads

Interior Column			
tributary area	$A_{T,i}$	161	ft^2
downward load	$P_{D,i}$	8.1	k
uplift	$P_{U,i}$	1.7	k
Edge Column			
tributary area	$A_{T,e}$	122.5	ft^2
downward load	$P_{D,e}$	6.1	k
uplift	$P_{U,e}$	1.3	k

Column Footings

Interior Column			
required bearing area	$P_{D,i}$ / q	5.37	ft^2
required volume of concrete	$P_{U,i}$ / 150 pcf	11.3	ft^3
Edge Column			
required bearing area	$P_{D,e}$ / q	4.08	ft^2
required volume of concrete	$P_{U,e}$ / 150 pcf	8.6	ft^3

Foundation Design

Interior Column Footing		
length	3	ft
width	3	ft
depth	1.5	ft
footing bearing area	9	ft^2
footing volume	13.5	ft^3
3 ft x 3 ft x 1.5 ft		
Exterior Column Footing		
length	3	ft
width	2	ft
depth	1.5	ft
footing bearing area	6	ft^2
footing volume	9	ft^3
3 ft x 2 ft x 1.5 ft		

LEOK
LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

THICKENED SLAB REINFORCEMENT

THICKENED SLAB REINFORCEMENT

SLAB DIMENSIONS

span length	L	2	ft
slab depth	h	1	ft
effective depth	d	9.25	in

MATERIAL PROPERTIES

compressive strength	f'_c	4000	psi
maximum compressive strain	ε_{cu}	0.003	in/in
density		150	pcf
lightweight factor	λ	1	
β_1		0.85	
yield strength, f_y	f_y	60000	psi

LOADS

self weight, w_{sw}	w_{sw}	150	psf
wall load	P_{wall}	1400	plf
pressure from wall load	$w_{wall} = P_{wall} / L$	700	psf
total distributed load	$w_u = w_{sw} + w_{wall}$	850	psf
applied moment	$M_u = w_u L^2 / 8$	425	lb-ft/ft
applied shear	$V_u = w_u L / 2$	850	plf

SHRINKAGE AND TEMPERATURE

Long Direction

cross sectional area	A_g	288	in^2
required reinforcing	$0.0018\ A_g$	0.5184	in^2

bar #	A_s	# of bars required
4	0.20	3
5	0.31	2

Short Direction

slab depth	h	12	in
required reinforcing	0.0018 h	0.0216	in^2/in

bar #	A_s	required spacing
4	0.20	9.1 in
5	0.31	14.2 in

LEOK LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

FLEXURAL STRENGTH

Reinforcing

bar number		4	
spacing		8	in
provided reinforcement	$A_{s,prov}/s$	0.025	in^2/in
required minimum	0.0018 h	0.0216	in^2/in
provided > required reinforcement?		**YES**	

Moment Capacity

depth of stress block	$a = \dfrac{A_s f_y}{0.85 f'_c}$	0.433	in
nominal strength	$M_n = A_s f_y (d - a/2)$	13302.8	lb-ft/ft

Tension Control Check

distance to neutral axis	$c = a / \beta_1$	0.510	in
steel strain	$\varepsilon_s = \varepsilon_{cu} [(d-c)/c]$	0.0515	in/in
tension controlled?	$\varepsilon_s > 0.005$?	**YES**	
reduction factor	ϕ	0.9	
design strength	ϕM_n	11973	lb-ft/ft

Utilization Ratio	$M_u / \phi M_n$	**4%**	**OK**

CONCRETE SHEAR CAPACITY

$V_c = \phi\lambda\sqrt{f'_c}d$	5265.19	lb/ft
$V_c > V_u$?	16%	

no shear reinforcement needed

FINAL DESIGN

reinforcement in short direction	**(3) #4 bars**
reinforcement in long direction	**#4 bars @ 8 inches O.C.**

COLUMN FOOTING REINFORCEMENT

FOOTING REINFORCEMENT

FOOTING DIMENSIONS

footing width	b	1	ft
footing length	L	3	ft
footing thickness	h	1.5	ft
effective depth	d	14	in

MATERIAL PROPERTIES

compressive strength	f'_c	4000	psi
maximum compressive strain	ε_{cu}	0.003	in/in
density		150	pcf
lightweight factor	λ	1	
β_1		0.85	
yield strength	f_y	60000	psi

LOADS

self weight	w_{sw}	225	psf
column load	P_{col}	12000	lb
pressure from column load	$w_{col} = P_{col} / Lb$	4000	psf
total distributed load	$w_u = w_{sw} + w_{col}$	4225	psf
applied moment	$M_u = w_u L^2 / 8$	4753	lb-ft/ft
applied shear	$V_u = w_u L / 2$	6337.5	plf

LEOK
LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

FLEXURAL STRENGTH

Reinforcing

bar number		5	
spacing	s	8	in
provided reinforcement	$A_{s,prov}/s$	0.038	in^2/in
required minimum	0.0018h	0.0324	in^2/in
provided > required reinforcement?		**YES**	

Moment Capacity

depth of stress block	$a = \frac{A_s f_y}{0.85 f'_c}$	0.677	in
nominal strength	$M_n = A_s f_y (d - a/2)$	31435.0	lb-ft/ft

Tension Control Check

distance to neutral axis	$c = a / \beta_1$	0.796	in
steel strain	$\varepsilon_s = \varepsilon_{cu} [(d-c)/c]$	0.0498	in/in
tension controlled?	$\varepsilon_s > 0.005$?	**YES**	
reduction factor	ϕ	0.9	
design strength	ϕM_n	28291	lb-ft/ft
Utilization Ratio	$\mathbf{M_u / \phi M_n}$	**17%**	**OK**

CONCRETE SHEAR CAPACITY

$V_c = \phi\lambda\sqrt{f'_c}d$	7968.94	lb/ft
$V_c > V_u$?	80%	

no shear reinforcement needed

FINAL DESIGN

reinforcement in both directions **#5 bars @ 8 inches O.C.**

SHEAR WALL HOLDOWN ANCHOR

SIMPSON Strong-Tie® Anchor Designer™ Software
Version 2.9.7376.5

Company:		Date:	6/2/2022
Engineer:		Page:	1/6
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 8.000
Code report: ICC-ES ESR-2508
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 11.13
c_{ac} (inch): 19.24
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Periodic
Temperature range, Short/Long: 150/110°F
Ignore 6do requirement: Not applicable
Build-up grout pad: No

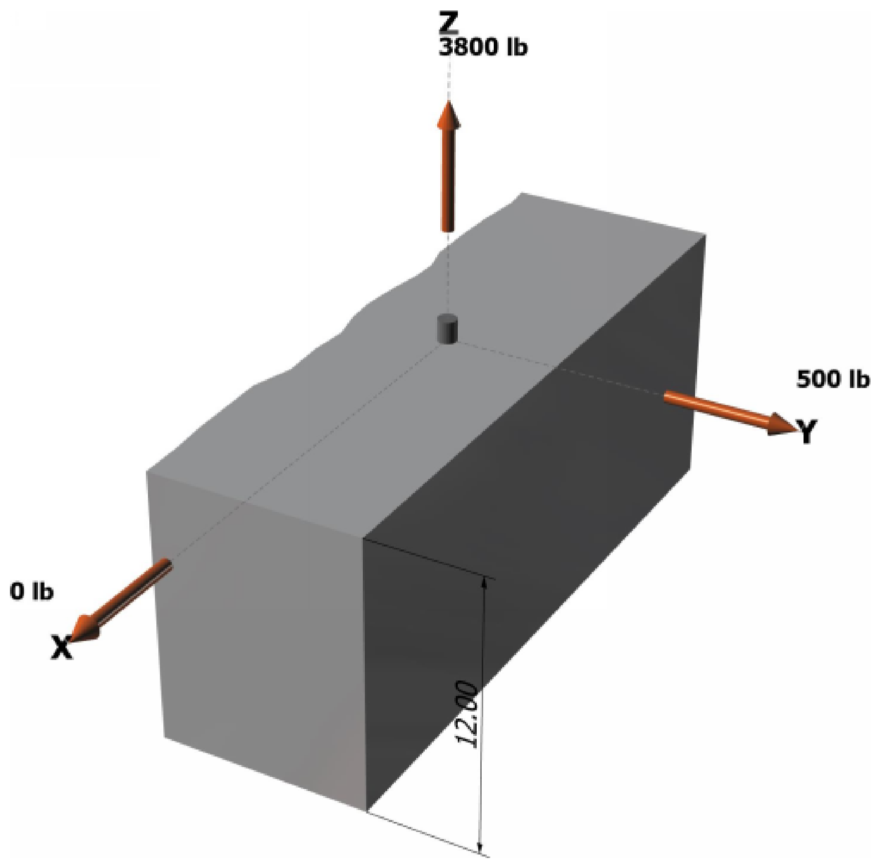
Recommended Anchor
Anchor Name: SET-XP® - SET-XP w/ 5/8"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-2508

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: No
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

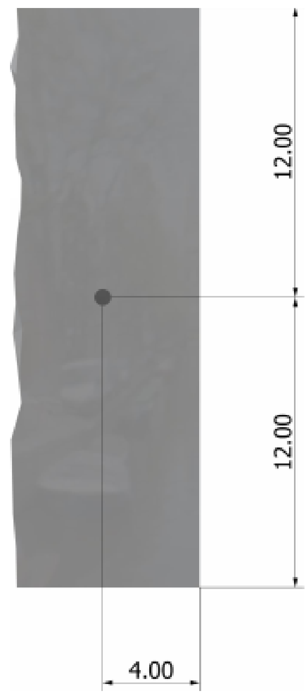
Strength level loads:

N_{ua} [lb]: 3800
V_{uax} [lb]: 0
V_{uay} [lb]: 500

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	3800.0	0.0	500.0	500.0
Sum	3800.0	0.0	500.0	500.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 3800
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13110	0.75	9833

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c\lambda_a\sqrt{f'_c}h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	8.000	19233

$\phi N_{cb} = \phi\ (A_{Nc} / A_{Nco})\ \Psi_{ed,N}\ \Psi_{c,N}\ \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
384.00	576.00	4.00	0.800	1.00	1.000	19233	0.65	6668

6. Adhesive Strength of Anchor in Tension (Sec. 17.4.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\tau_{k,cr}$ (psi)
435	1.72	1.00	748

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.4.5.2)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	748	0.63	8.000	11753

$\phi N_a = \phi\ (A_{Na} / A_{Na0})\ \Psi_{ed,Na}\ \Psi_{cp,Na} N_{ba}$ (Sec. 17.3.1 & Eq. 17.4.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	ϕ	ϕN_a (lb)
193.86	258.98	8.05	4.00	0.849	1.000	11753	0.55	4109

250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
7865	1.0	0.65	5112

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
5.00	0.625	1.00	4000	4.00	4244

$\phi V_{cby} = \phi\ (A_{Vc} / A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1 & Eq. 17.5.2.1a)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cby} (lb)
72.00	72.00	1.000	1.000	1.000	4244	0.70	2971

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
5.00	0.625	1.00	4000	12.00	22053

$\phi V_{cby} = \phi\ (2)(A_{Vc} / A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1a)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cby} (lb)
264.00	648.00	1.000	1.000	1.225	22053	0.70	15405

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cp} = \phi \min|k_{cp}N_a\ ;\ k_{cp}N_{cb}| = \phi \min|k_{cp}(A_{Na} / A_{Na0})\Psi_{ed,Na}\Psi_{cp,Na}N_{ba}\ ;\ k_{cp}(A_{Nc} / A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b|$ (Sec. 17.3.1 & Eq. 17.5.3.1a)

k_{cp}	A_{Na} (in^2)	A_{Na0} (in^2)	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	N_a (lb)
2.0	193.86	258.98	0.849	1.000	11753	7470

A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	N_{cb} (lb)	ϕ	ϕV_{cp} (lb)
384.00	576.00	0.800	1.000	1.000	19233	10258	0.70	10458

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	3800	9833	0.39	Pass
Concrete breakout	3800	6668	0.57	Pass
Adhesive	**3800**	**4109**	**0.92**	**Pass (Governs)**

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	500	5112	0.10	Pass
T Concrete breakout y+	**500**	**2971**	**0.17**	**Pass (Governs)**
|| Concrete breakout x-	**500**	**15405**	**0.03**	**Pass (Governs)**
Pryout	500	10458	0.05	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..1	0.92	0.00	92.5%	1.0	Pass

SET-XP w/ 5/8"Ø F1554 Gr. 36 with hef = 8.000 inch meets the selected design criteria.

12. Warnings

- When cracked concrete is selected, concrete compressive strength used in concrete breakout strength in tension, adhesive strength in tension and concrete pryout strength in shear for SET-XP adhesive anchor is limited to 2,500 psi per ICC-ES ESR-2508 Section 5.3.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

SILL ANCHORS

Anchor Designer™
Software
Version 2.9.7376.8

Company:		Date:	6/10/2022
Engineer:		Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

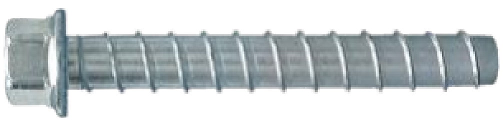
General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.375
Nominal Embedment depth (inch): 2.500
Effective Embedment depth, h_{ef} (inch): 1.770
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
h_{min} (inch): 4.00
c_{ac} (inch): 2.69
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 6.00 x 5.50 x 1.50

Recommended Anchor
Anchor Name: Titen HD® - 3/8"Ø Titen HD, hnom:2.5" (64mm)
Code Report: ICC-ES ESR-2713

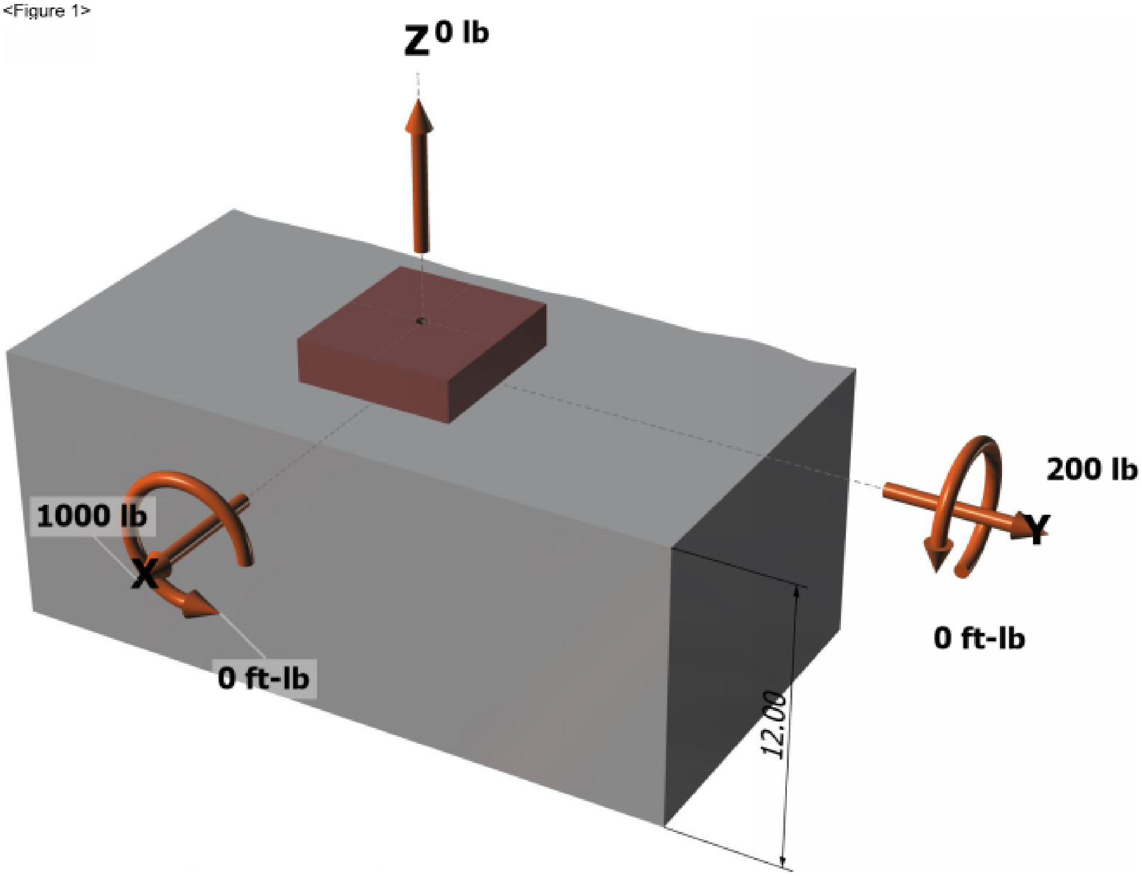


Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

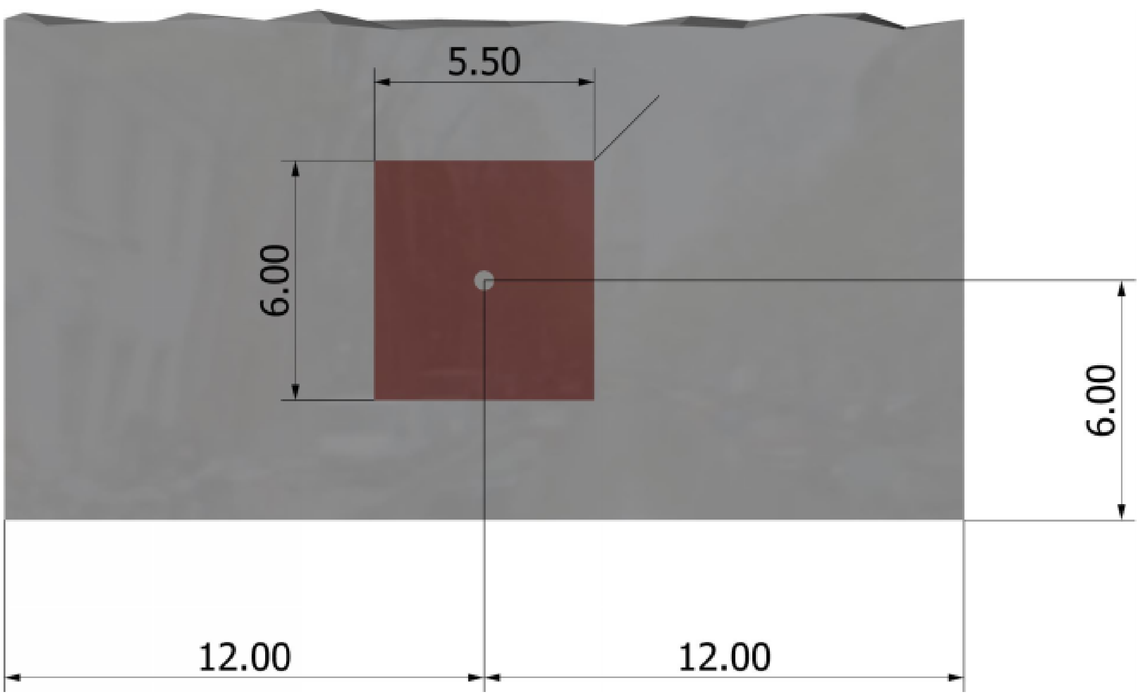
Strength level loads:

N_{ua} [lb]: 0
V_{uax} [lb]: 1000
V_{uay} [lb]: 200
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0

<Figure 1>



<Figure 2>



SIMPSON Strong-Tie

Anchor Designer™ Software
Version 2.9.7376.8

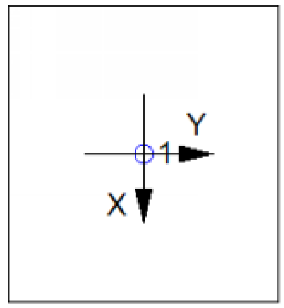
Company:		Date:	6/10/2022
Engineer:		Page:	4/5
Project:			
Address:			
Phone:			
E-mail:			

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	1000.0	200.0	1019.8
Sum	0.0	1000.0	200.0	1019.8

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 0
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
4460	1.0	0.60	2676

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
1.77	0.375	1.00	4000	12.00	15371

$\phi V_{cby} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1 & Eq. 17.5.2.1a)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cby} (lb)
288.00	648.00	0.800	1.000	1.225	15371	0.70	4685

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
1.77	0.375	1.00	4000	6.00	5434

$\phi V_{cbx} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1 & Eq. 17.5.2.1a)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbx} (lb)
162.00	162.00	1.000	1.000	1.000	5434	0.70	3804

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
1.77	0.375	1.00	4000	12.00	15371

$\phi V_{cbx} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1a)

LEOK
LEIGH + O'KANE
250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Made by: WNH/RAH	Date: 06/15/2022	Job No. 22-19
Revision:	Date:	Sheet No. 59
Project Name: Advanced Aesthetic Center		

SIMPSON Strong-Tie®

Anchor Designer™ Software
Version 2.9.7376.8

Company:		Date:	6/10/2022
Engineer:		Page:	5/5
Project:			
Address:			
Phone:			
E-mail:			

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbx} (lb)
288.00	648.00	1.000	1.000	1.225	15371	0.70	11714

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
1.77	0.375	1.00	4000	6.00	5434

$\phi V_{cby} = \phi\ (2)(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1a)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cby} (lb)
162.00	162.00	1.000	1.000	1.000	5434	0.70	7608

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cp} = \phi k_{cp}N_{cb} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1a)

k_{cp}	A_{Nc} (in²)	A_{Nco} (in²)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
1.0	28.20	28.20	1.000	1.000	1.000	2532	0.70	1772

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status
Steel	1020	2676	0.38	Pass
T Concrete breakout y+	200	4685	0.04	Pass
T Concrete breakout x+	1000	3804	0.26	Pass
|| Concrete breakout y+	1000	11714	0.09	Pass
|| Concrete breakout x+	200	7608	0.03	Pass
Concrete breakout, combined	-	-	0.27	Pass
Pryout	**1020**	**1772**	**0.58**	**Pass (Governs)**

3/8"Ø Titen HD, hnom:2.5" (64mm) meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

BASE PLATES

SIMPSON Strong-Tie®

Anchor Designer™
Software
Version 2.9.7376.7

Company:		Date:	6/10/2022
Engineer:		Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

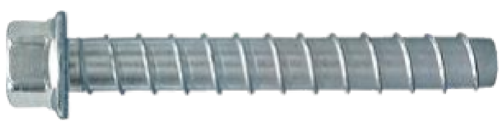
Anchor Information:
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.500
Nominal Embedment depth (inch): 4.000
Effective Embedment depth, h_{ef} (inch): 2.990
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
h_{min} (inch): 6.25
c_{ac} (inch): 4.50
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 8.00 x 8.00 x 0.50
Yield stress: 36000 psi

Profile type/size: Pipe3STD

Recommended Anchor
Anchor Name: Titen HD® - 1/2"Ø Titen HD, hnom:4" (102mm)
Code Report: ICC-ES ESR-2713

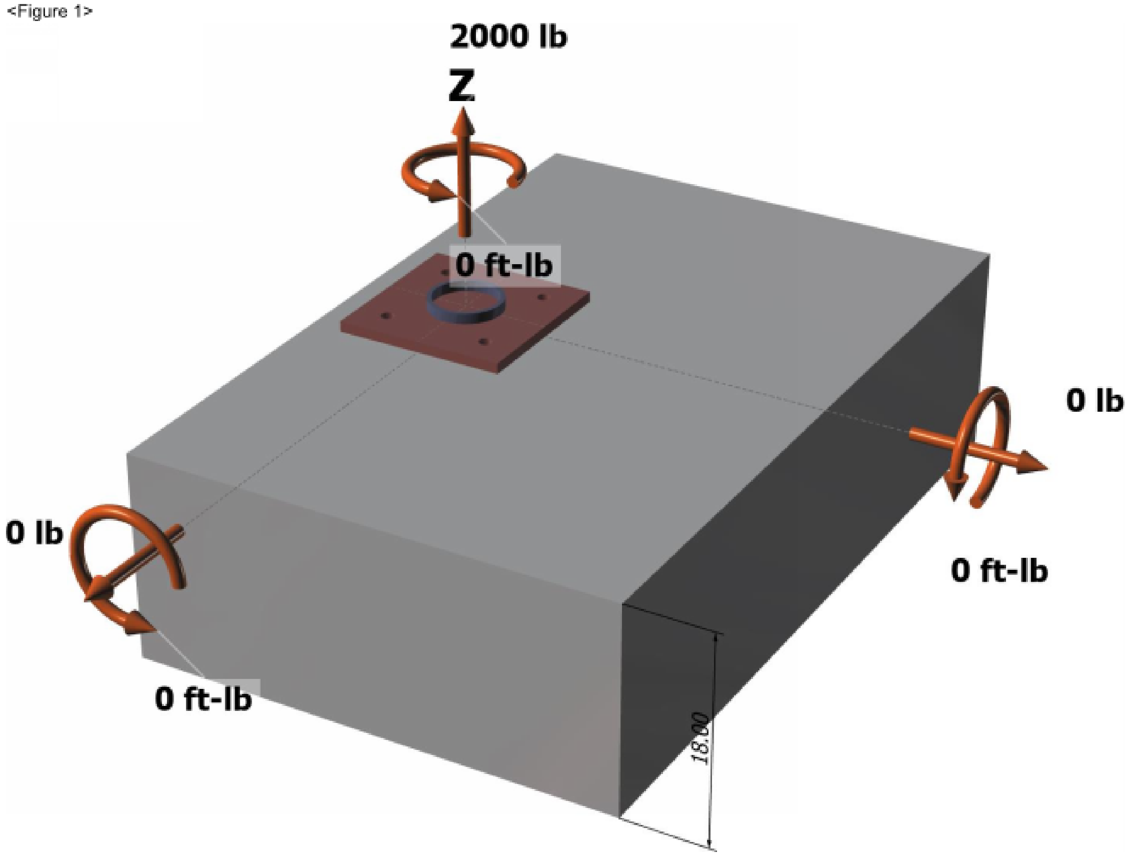


Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

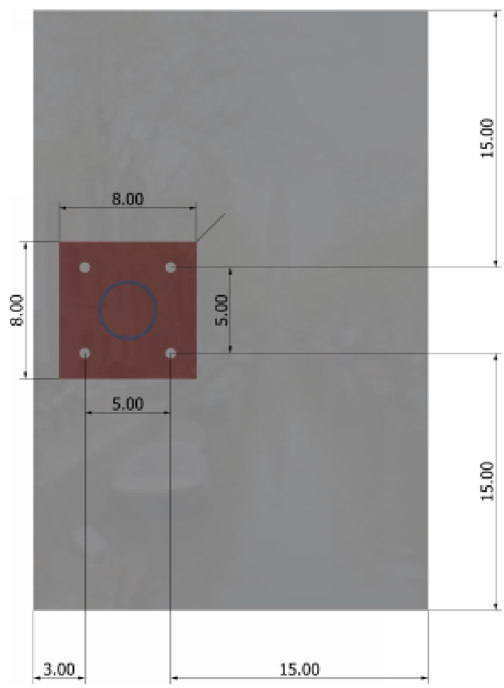
Strength level loads:

N_{ua} [lb]: 2000
V_{uax} [lb]: 0
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



<Figure 2>



250 NE Mulberry St., Suite 201
Lee's Summit, MO 64086
816.444.3144
www.leok.com

Made by: WNH/RAH	Date: 06/15/2022	Job No. 22-19
Revision:	Date:	Sheet No. 63
Project Name: Advanced Aesthetic Center		

SIMPSON Strong-Tie® Anchor Designer™ Software
Version 2.9.7376.7

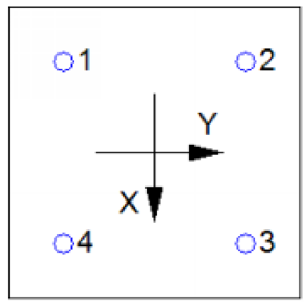
Company:		Date:	6/10/2022
Engineer:		Page:	4/5
Project:			
Address:			
Phone:			
E-mail:			

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	500.0	0.0	0.0	0.0
2	500.0	0.0	0.0	0.0
3	500.0	0.0	0.0	0.0
4	500.0	0.0	0.0	0.0
Sum	2000.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 2000
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
20130	0.65	13085

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	4000	2.990	5559

$\phi N_{cbg} = \phi\, (A_{Nc} / A_{Nco})\, \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
174.42	80.46	3.00	1.000	0.901	1.00	1.000	5559	0.65	7054

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	500	13085	0.04	Pass
Concrete breakout	**2000**	**7054**	**0.28**	**Pass (Governs)**

1/2"Ø Titen HD, hnom:4" (102mm) meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.192 inch

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.