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

ITAP – Lee's Summit

228 SW Main Street, Lee's Summit, Missouri 64063

STATE OF MISSOURI
ADAM
CHRISTOPHER
O'KANE
NUMBER
PE-2017000344
PROFESSIONAL ENGINEER

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GENERAL DESIGN CONSIDERATIONS

The calculations provided are for modifications to the existing building located at 228 SW Main Street in Lee's Summit, MO. The space is being modified for the tenant, International Tap House.

REFERENCED DESIGN STANDARDS

- 2018 IBC – International Building Code
- ASCE 7-16 – American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures
- AISC 360-16 – American Institute of Steel Construction Fifteenth Edition
- ACI 318-14 – American Concrete Institute Building Code Requirements for Structural Concrete
- NDS 2018 – National Design Specification for Wood Construction

NEW LINTEL OVER NEW WINDOW OPENING IN EXISTING WALL

SECTION PROPERITES: L 8x6x1/2

yield strength	F_y	36 ksi
modulus of elasticity	E	29000 ksi
vertical leg length	b_1	8 in
horizontal leg length	b_2	6 in
thickness	t	0.5 in
cross section area	A	6.8 in^2
major axis moment of inertia	I_x	44.4 in^4
elastic section modulus (major axis)	S_x	8.01 in^3
radius of gyration (major axis)	r_x	2.55 in
plastic section modulus (major axis)	Z_x	14.6 in^3
moment of inertia (minor axis)	I_y	21.7 in^4
elastic section modulus (minor axis)	S_y	4.79 in^3
radius of gyration (minor axis)	r_y	1.79 in
plastic section modulus (minor axis)	Z_y	8.52 in^3
moment of inertia (principal axis)	I_z	11.5 in^4
elastic section modulus (principal axis)	S_z	3.98 in^3
radius of gyration (principal axis)	r_z	1.3 in
width to thickness ratio	b/t	16.00
limiting compact/noncompact ratio	$\lambda_p = 0.54 \sqrt{(E/F_y)}$	15.33
limiting noncompact/slender ratio	$\lambda_r = 0.91 \sqrt{(E/F_y)}$	25.83

LOADING

span length	L	6 ft
trib length	T	19.5 ft
total wall height above window	H	18 ft
wall weight	W_w	140 psf
floor dead	D_f	20 psf
floor live	L_f	80 psf
roof dead	D_r	20 psf
roof snow	S	20 psf

service line load	$w = [D_f + D_r + L_f + S]*T + W_w H$	**5250 plf**
factored line load	$w_u = [1.2\ D + 1.6\ (L + S)]*T + 1.2\ W_w H$	**7080 plf**
factored moment	$M_u = w_u\ L^2 / 8$	**31.86 k-ft**

SINGLE LEG BENDING (F11)

thickness	t	0.5	in
elastic section modulus per foot	$S = [t^2 / 6]*12$	0.5	in^3/ft
plastic section modulus per foot	$Z = [t^2 / 4]*12$	0.75	in^3/ft
allowable moment	$\phi M_n = 0.9\,[F_y\,Z] < 0.9\,[1.6\,F_y\,S]$	24.30	k-in/ft
required moment per angle	$w_u\,b / 2$	21.24	k-in/ft
utilization		**87%**	

SINGLE ANGLE IN FLEXURE (F10)

PRINCIPAL AXIS YIELDING (F10.1)

yield moment	$M_y = F_y\,S_Z$	143.28	k-in
allowable moment	$\phi M_n = 0.9*1.5\,M_y$	16.12	k-ft
required moment per angle	$M_u / 2$	15.93	k-ft
utilization		**99%**	

LATERAL TORSIONAL BUCKLING ABOUT PRINCIPAL AXIS (F10.2)

lateral torsional buckling moment (F10-4)	M_{cr}	1239.01	k-in
section property from Table C-F10.1	β_w	-3.14	
lateral torsional modification factor	C_b	1	
laterally unbraced length	L_b	6	ft
allowable moment	$\phi M_n = 0.9*[1.92 - 1.17\,\sqrt{(M_y/M_{cr})}]M_y$	16.36	k-ft
required moment per angle	$M_u / 2$	15.93	k-ft
utilization		**97%**	

LEG LOCAL BUCKLING (F10.3)

(b) for sections with noncompact legs	$\mathbf{M_n = F_y\,S_c\,[2.43 - 1.72\,(b/t)\,\sqrt{(F_y/E)}]}$		
elastic section modulus of toe in compression	$S_c = 0.80\,S_x$	6.408	in^3
allowable moment	ϕM_n	25.27	k-ft
required moment per angle	$M_u / 2$	15.93	k-ft
utilization		**63%**	

DEFLECTION

deflection	$\Delta = 5\,w\,L^4 / 384\,E\,I_x$	0.119	in
deflection limit	$\Delta_{allow} = L/600$	0.12	in
utilization	$\mathbf{\Delta / \Delta_{allow}}$	**99%**	

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NEW LINTEL OVER NEW DUCT OPENING IN EXISTING WALL

SECTION PROPERITES: L 6x4x3/8 at Duct

yield strength	F_y	36 ksi
modulus of elasticity	E	29000 ksi
vertical leg length	b_1	6 in
horizontal leg length	b_2	4 in
thickness	t	0.375 in
cross section area	A	3.61 in^2
major axis moment of inertia	I_x	13.4 in^4
elastic section modulus (major axis)	S_x	3.3 in^3
radius of gyration (major axis)	r_x	1.93 in
plastic section modulus (major axis)	Z_x	5.89 in^3
moment of inertia (minor axis)	I_y	4.86 in^4
elastic section modulus (minor axis)	S_y	1.58 in^3
radius of gyration (minor axis)	r_y	1.16 in
plastic section modulus (minor axis)	Z_y	2.79 in^3
moment of inertia (principal axis)	I_z	2.73 in^4
elastic section modulus (principal axis)	S_z	1.31 in^3
radius of gyration (principal axis)	r_z	0.87 in
width to thickness ratio	b/t	16.00
limiting compact/noncompact ratio	$\lambda_p = 0.54 \sqrt{(E/F_y)}$	15.33
limiting noncompact/slender ratio	$\lambda_r = 0.91 \sqrt{(E/F_y)}$	25.83

LOADING

span length	L	2 ft
trib length	T	19.5 ft
total wall height above opening	H	14 ft
wall weight	W_w	140 psf
floor dead	D_f	20 psf
floor live	L_f	80 psf
roof dead	D_r	20 psf
roof snow	S	20 psf
service line load	$w = [D_f + D_r + L_f + S]*T + W_wH$	**4690 plf**
factored line load	$w_u = [1.2\ D + 1.6\ (L + S)]*T + 1.2\ W_wH$	**6408 plf**
factored moment	$M_u = w_u\ L^2 / 8$	**3.204 k-ft**

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SINGLE LEG BENDING (F11)

thickness	t	0.375	in
elastic section modulus per foot	$S = [t^2 / 6]*12$	0.28125	in^3/ft
plastic section modulus per foot	$Z = [t^2 / 4]*12$	0.42	in^3/ft
allowable moment	$\phi M_n = 0.9\,[F_y\,Z] < 0.9\,[1.6\,F_y\,S]$	13.67	k-in/ft
required moment per angle	$w_u\,b / 2$	12.82	k-in/ft
utilization		**94%**	

SINGLE ANGLE IN FLEXURE (F10)

PRINCIPAL AXIS YIELDING (F10.1)

yield moment	$M_y = F_y\,S_Z$	47.16	k-in
allowable moment	$\phi M_n = 0.9*1.5\,M_y$	5.31	k-ft
required moment per angle	$M_u / 2$	1.602	k-ft
utilization		**30%**	

LATERAL TORSIONAL BUCKLING ABOUT PRINCIPAL AXIS (F10.2)

lateral torsional buckling moment (F10-4)	M_{cr}	346.59	k-in
section property from Table C-F10.1	β_w	-3.14	
lateral torsional modification factor	C_b	1	
laterally unbraced length	L_b	6	ft
allowable moment	$\phi M_n = 0.9*[1.92 - 1.17\,\sqrt{(M_y/M_{cr})}]M_y$	5.26	k-ft
required moment per angle	$M_u / 2$	1.602	k-ft
utilization		**30%**	

LEG LOCAL BUCKLING (F10.3)

(b) for sections with noncompact legs	$\mathbf{M_n = F_y\,S_c\,[2.43 - 1.72\,(b/t)\,\sqrt{(F_y/E)}]}$		
elastic section modulus of toe in compression	$S_c = 0.80\,S_x$	2.64	in^3
allowable moment	ϕM_n	10.41	k-ft
required moment per angle	$M_u / 2$	1.602	k-ft
utilization		**15%**	

DEFLECTION

deflection	$\Delta = 5\,w\,L^4 / 384\,E\,I_x$	0.004	in
deflection limit	$\Delta_{allow} = L/600$	0.04	in
utilization	$\mathbf{\Delta / \Delta_{allow}}$	**11%**	

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NEW LINTEL OVER ENLARGED DOOR OPENING IN EXISTING WALL

SECTION PROPERITES: L 6x4x3/8 at Door

yield strength	F_y	36	ksi
modulus of elasticity	E	29000	ksi
vertical leg length	b_1	6	in
horizontal leg length	b_2	4	in
thickness	t	0.375	in
cross section area	A	3.61	in^2
major axis moment of inertia	I_x	13.4	in^4
elastic section modulus (major axis)	S_x	3.3	in^3
radius of gyration (major axis)	r_x	1.93	in
plastic section modulus (major axis)	Z_x	5.89	in^3
moment of inertia (minor axis)	I_y	4.86	in^4
elastic section modulus (minor axis)	S_y	1.58	in^3
radius of gyration (minor axis)	r_y	1.16	in
plastic section modulus (minor axis)	Z_y	2.79	in^3
moment of inertia (principal axis)	I_z	2.73	in^4
elastic section modulus (principal axis)	S_z	1.31	in^3
radius of gyration (principal axis)	r_z	0.87	in
width to thickness ratio	b/t	16.00	
limiting compact/noncompact ratio	$\lambda_p = 0.54\sqrt{(E/F_y)}$	15.33	
limiting noncompact/slender ratio	$\lambda_r = 0.91\sqrt{(E/F_y)}$	25.83	

LOADING

span length	L	4	ft
trib length	T	10	ft
total wall height above opening	H	11	ft
wall weight	W_w	140	psf
floor dead	D_f	20	psf
floor live	L_f	80	psf
roof dead	D_r	20	psf
roof snow	S	20	psf

service line load	$w = [D_f + D_r + L_f + S]*T + W_wH$	**2940**	**plf**
factored line load	$w_u = [1.2\ D + 1.6\ (L + S)]*T + 1.2\ W_wH$	**3928**	**plf**
factored moment	$M_u = w_u\ L^2 / 8$	**7.856**	**k-ft**

SINGLE LEG BENDING (F11)

thickness	t	0.375 in
elastic section modulus per foot	$S = [t^2 / 6]*12$	0.28125 in^3/ft
plastic section modulus per foot	$Z = [t^2 / 4]*12$	0.42 in^3/ft
allowable moment	$\phi M_n = 0.9\ [F_y Z] < 0.9\ [1.6\ F_y S]$	13.67 k-in/ft
required moment per angle	$w_u\ b / 2$	7.86 k-in/ft
utilization		**57%**

SINGLE ANGLE IN FLEXURE (F10)

PRINCIPAL AXIS YIELDING (F10.1)

yield moment	$M_y = F_y S_Z$	47.16 k-in
allowable moment	$\phi M_n = 0.9*1.5\ M_y$	5.31 k-ft
required moment per angle	$M_u / 2$	3.928 k-ft
utilization		**74%**

LATERAL TORSIONAL BUCKLING ABOUT PRINCIPAL AXIS (F10.2)

lateral torsional buckling moment (F10-4)	M_{cr}	346.59 k-in
section property from Table C-F10.1	β_w	-3.14
lateral torsional modification factor	C_b	1
laterally unbraced length	L_b	6 ft
allowable moment	$\phi M_n = 0.9*[1.92 - 1.17\ \sqrt{(M_y/M_{cr})}]M_y$	5.26 k-ft
required moment per angle	$M_u / 2$	3.928 k-ft
utilization		**75%**

LEG LOCAL BUCKLING (F10.3)

(b) for sections with noncompact legs	$\mathbf{M_n = F_y S_c [2.43 - 1.72 (b/t) \sqrt{(F_y/E)}]}$	
elastic section modulus of toe in compression	$S_c = 0.80\ S_x$	2.64 in^3
allowable moment	ϕM_n	10.41 k-ft
required moment per angle	$M_u / 2$	3.928 k-ft
utilization		**38%**

DEFLECTION

deflection	$\Delta = 5\ w\ L^4 / 384\ E\ I_x$	0.044 in
deflection limit	$\Delta_{allow} = L/600$	0.08 in
utilization	$\mathbf{\Delta / \Delta_{allow}}$	**54%**

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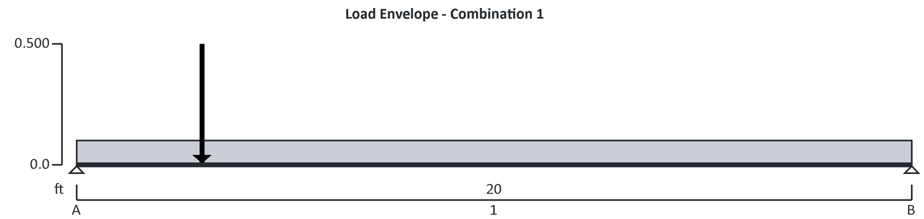
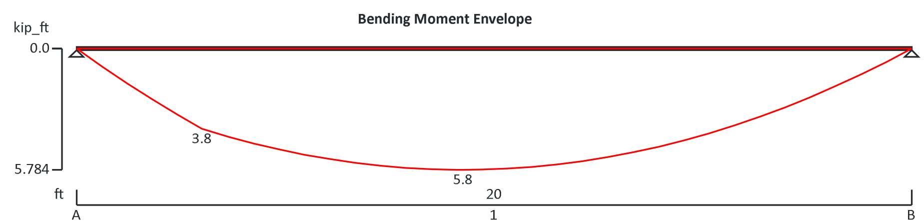
EXISTING WOOD JOIST MODIFICATION FOR NEW RTU

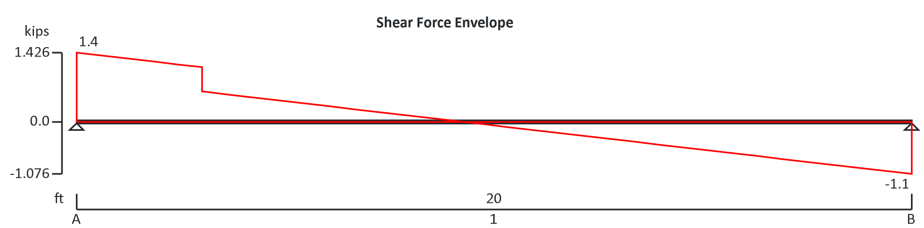
STRUCTURAL WOOD MEMBER ANALYSIS & DESIGN (NDS)

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

Tedds calculation version 1.7.10







Applied loading

Beam loads

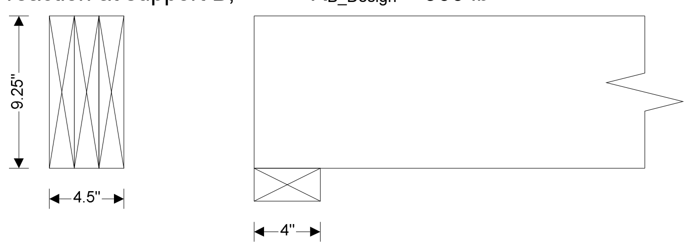
Dead self weight of beam × 1
Design full UDL 90 lb/ft
Dead point load 500 lb at 36.00 in

Load combinations

Load combination 1	Support A	Dead × 1.00
		Design × 1.00
	Span 1	Dead × 1.00
		Design × 1.00
	Support B	Dead × 1.00
		Design × 1.00

Analysis results

Maximum moment;	M_{max} = **5784** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **5784** lb_ft	
Maximum shear;	F_{max} = **1426** lb;	F_{min} = **-1076** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **1426** lb	
Total load on member;	W_{tot} = **2502** lb	
Reaction at support A;	R_{A_max} = **1426** lb;	R_{A_min} = **1426** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **526** lb	
Unfactored design load reaction at support A;	R_{A_Design} = **900** lb	
Reaction at support B;	R_{B_max} = **1076** lb;	R_{B_min} = **1076** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **176** lb	
Unfactored design load reaction at support B;	R_{B_Design} = **900** lb	



Sawn lumber section details

Nominal breadth of sections;	b_{nom} = **2** in
Dressed breadth of sections;	b = **1.5** in
Nominal depth of sections;	d_{nom} = **10** in
Dressed depth of sections;	d = **9.25** in
Number of sections in member;	N = **3**
Overall breadth of member;	$b_b = N \times b$ = **4.5** in
Species, grade and size classification;	Spruce-Pine-Fir, No.2 grade, 2" & wider
Bending parallel to grain;	F_b = **875** lb/in^2
Tension parallel to grain;	F_t = **450** lb/in^2
Compression parallel to grain;	F_c = **1150** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **425** lb/in^2
Shear parallel to grain;	F_v = **135** lb/in^2
Modulus of elasticity;	E = **1400000** lb/in^2
Modulus of elasticity, stability calculations;	E_{min} = **510000** lb/in^2
Mean shear modulus;	G_{def} = E / 16 = **87500** lb/in^2

Member details

Service condition;	**Dry**
Length of span;	L_{s1} = **20** ft
Length of bearing;	L_b = **4** in
Load duration;	**Ten years**

Section properties

Cross sectional area of member;	$A = N \times b \times d$ = **41.62** in^2
Section modulus;	$S_x = N \times b \times d^2 / 6$ = **64.17** in^3

	$S_y = d \times (N \times b)^2 / 6 =$ **31.22** in^3
Second moment of area;	$I_x = N \times b \times d^3 / 12 =$ **296.79** in^4
	$I_y = d \times (N \times b)^3 / 12 =$ **70.24** in^4

Adjustment factors

Load duration factor - Table 2.3.2;	C_D = **1.00**
Temperature factor - Table 2.3.3;	C_t = **1.00**
Size factor for bending - Table 4A;	C_{Fb} = **1.10**
Size factor for tension - Table 4A;	C_{Ft} = **1.10**
Size factor for compression - Table 4A;	C_{Fc} = **1.00**
Flat use factor - Table 4A;	C_{fu} = **1.20**
Incising factor for modulus of elasticity - Table 4.3.8	
	C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8	
	C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8	
	C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9;	C_r = **1.15**
Bearing area factor - cl.3.10.4;	C_b = **1.00**
Depth-to-breadth ratio;	$d_{nom} / (N \times b_{nom})$ = **1.67**
- Beam is fully restrained	
Beam stability factor - cl.3.3.3;	C_L = **1.00**

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain;	$F_{c\_perp}' = F_{c\_perp} \times C_t \times C_{ic\_perp} \times C_b$ = **425** lb/in^2
Applied compression stress perpendicular to grain;	$f_{c\_perp} = R_{A\_max} / (N \times b \times L_b)$ = **79** lb/in^2
	f_{c_perp} / F_{c_perp}' = **0.186**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress;	$F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1107** lb/in^2
Actual bending stress;	$f_b = M / S_x$ = **1082** lb/in^2
	f_b / F_b' = **0.977**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress;	$F_v' = F_v \times C_D \times C_t \times C_i$ = **135** lb/in^2
Actual shear stress - eq.3.4-2;	$f_v = 3 \times F / (2 \times A)$ = **51** lb/in^2
	f_v / F_v' = **0.381**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection;	$E' = E \times C_{ME} \times C_t \times C_{iE}$ = **1400000** lb/in^2
Design deflection;	$\delta_{adm} = 0.0056 \times L_{s1}$ = **1.344** in
Total deflection;	δ_{b_s1} = **1.021** in
	$\delta_{b\_s1} / \delta_{adm}$ = **0.760**

PASS - Total deflection is less than design deflection