

Leigh & O'Kane - Structural Calculations

JKV Salon Remodel



05/24/2022

Adam C. O'Kane
5-24-2022



Leigh + O'Kane
 250 NE Mulberry
 Suite 201
 Lee's Summit MO 64086

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JKV - Salon					
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GENERAL DESIGN CONSIDERATIONS

The calculations provided are for a remodel of the existing residential units that were expanded to create the existing thrift store. The area is being repurposed as a salon. New openings in the existing demising walls and new widening of exterior wall openings are the scope of services provided for the structural documents.

REFERENCED DESIGN STANDARDS

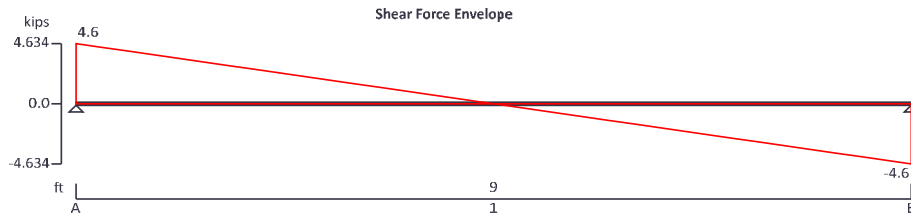
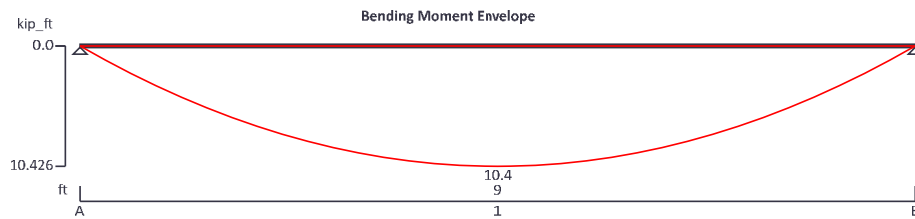
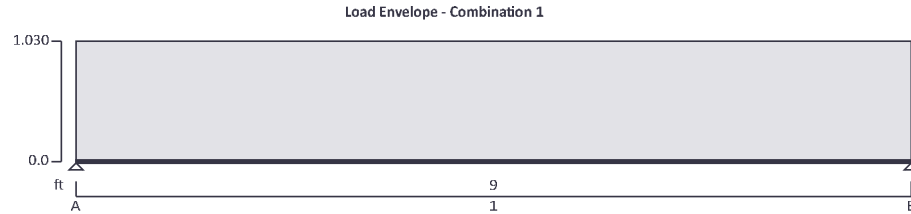
- 2018 IBC – International Building Code
- ASCE 7-16 – American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures
- NDS 2018 – National Design Specification for Wood Construction

LVL DESIGN

STRUCTURAL COMPOSITE LUMBER MEMBER ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2015 using the ASD method

Tedds calculation version 1.7.10



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Applied loading

Beam loads

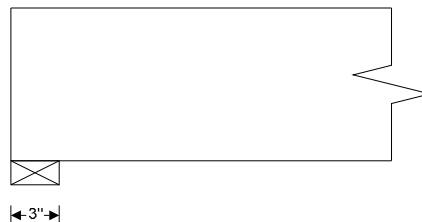
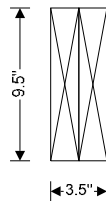
Dead self weight of beam $\times 1$
Dead full UDL 340 lb/ft
Live full UDL 680 lb/ft

Load combinations

Load combination 1	Support A	Dead $\times 1.00$
		Live $\times 1.00$
	Span 1	Dead $\times 1.00$
		Live $\times 1.00$
	Support B	Dead $\times 1.00$
		Live $\times 1.00$

Analysis results

Maximum moment	$M_{max} = 10426 \text{ lb_ft}$	$M_{min} = 0 \text{ lb_ft}$
Design moment	$M = \max(\text{abs}(M_{max}), \text{abs}(M_{min})) = 10426 \text{ lb_ft}$	
Maximum shear	$F_{max} = 4634 \text{ lb}$	$F_{min} = -4634 \text{ lb}$
Design shear	$F = \max(\text{abs}(F_{max}), \text{abs}(F_{min})) = 4634 \text{ lb}$	
Total load on member	$W_{tot} = 9267 \text{ lb}$	
Reaction at support A	$R_{A_max} = 4634 \text{ lb}$	$R_{A_min} = 4634 \text{ lb}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 1574 \text{ lb}$	
Unfactored live load reaction at support A	$R_{A_Live} = 3060 \text{ lb}$	
Reaction at support B	$R_{B_max} = 4634 \text{ lb}$	$R_{B_min} = 4634 \text{ lb}$
Unfactored dead load reaction at support B	$R_{B_Dead} = 1574 \text{ lb}$	
Unfactored live load reaction at support B	$R_{B_Live} = 3060 \text{ lb}$	



Composite section details

Breadth of composite section	$b = 1.75 \text{ in}$
Depth of composite section	$d = 9.5 \text{ in}$
Number of composite sections in member	$N = 2$
Overall breadth of composite member	$b_b = N \times b = 3.5 \text{ in}$
Composite type and grade	Microllam LVL, 2.0E-2600Fb grade
Bending parallel to grain	$F_b = 2600 \text{ lb/in}^2$
Tension parallel to grain	$F_t = 1555 \text{ lb/in}^2$
Compression parallel to grain	$F_c = 2510 \text{ lb/in}^2$
Compression perpendicular to grain	$F_{c_perp} = 750 \text{ lb/in}^2$
Shear parallel to grain	$F_v = 285 \text{ lb/in}^2$

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Modulus of elasticity $E = 2000000 \text{ lb/in}^2$
 Modulus of elasticity, stability calculations $E_{\min} = 1017000 \text{ lb/in}^2$
 Mean shear modulus $G_{\text{def}} = E / 16 = 125000 \text{ lb/in}^2$
 Average density $\rho = 42 \text{ lb/ft}^3$

Member details

Service condition **Dry**
 Length of span $L_{s1} = 9 \text{ ft}$
 Length of bearing $L_b = 3 \text{ in}$
 Load duration **Ten years**

Section properties

Cross sectional area of member $A = N \times b \times d = 33.25 \text{ in}^2$
 Section modulus $S_x = N \times b \times d^2 / 6 = 52.65 \text{ in}^3$
 $S_y = d \times (N \times b)^2 / 6 = 19.40 \text{ in}^3$
 Second moment of area $I_x = N \times b \times d^3 / 12 = 250.07 \text{ in}^4$
 $I_y = d \times (N \times b)^3 / 12 = 33.94 \text{ in}^4$

Adjustment factors

Load duration factor - Table 2.3.2 $C_D = 1.00$
 Temperature factor - Table 2.3.3 $C_t = 1.00$
 Volume factor $C_V = (12 \text{ in} / \max(d, 3.5 \text{ in}))^{0.136} = 1.03$
 Repetitive member factor - cl.8.3.7 $C_r = 1.00$
 Length factor $C_{L_{\text{en}}} = (4 \text{ ft} / L_{s1})^{0.085} = 0.93$
 Bearing area factor - cl.3.10.4 $C_b = 1.00$
 Depth-to-breadth ratio $d / (N \times b) = 2.71$
 Effective laterally unsupported span length $l_e = 9 \text{ ft}$
 Slenderness ratio for bending members - eq.3.3-5 $R_b = \sqrt{[l_e \times d / (N \times b)^2]} = 9.152$
 Adjusted bending design value for bending $F_b^* = F_b \times C_D \times C_M \times C_t \times C_r = 2600 \text{ lb/in}^2$
 Adjusted modulus of elasticity for member stability $E_{\min}' = E_{\min} \times C_M \times C_t = 1017000 \text{ lb/in}^2$
 Critical buckling design value for bending $F_{bE} = 1.2 \times E_{\min}' / R_b^2 = 14571 \text{ lb/in}^2$
 Beam stability factor - eq.3.3-6 $C_L = [1 + (F_{bE} / F_b^*)] / 1.9 - \sqrt{([1 + (F_{bE} / F_b^*)] / 1.9)^2 - (F_{bE} / F_b^*) / 0.95} = 0.99$

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c_{\text{perp}}}' = F_{c_{\text{perp}}} \times C_t \times C_b = 750 \text{ lb/in}^2$
 Applied compression stress perpendicular to grain $f_{c_{\text{perp}}} = R_{A_{\text{max}}} / (N \times b \times L_b) = 441 \text{ lb/in}^2$
 $f_{c_{\text{perp}}} / F_{c_{\text{perp}}}' = 0.588$

PASS - Design compressive stress exceeds applied compressive stress at bearing


Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_V \times C_r = 2655 \text{ lb/in}^2$
 Actual bending stress $f_b = M / S_x = 2376 \text{ lb/in}^2$
 $f_b / F_b' = 0.895$

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_t = 285 \text{ lb/in}^2$

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Actual shear stress - eq.3.4-2

$$f_v = 3 \times F / (2 \times A) = 209 \text{ lb/in}^2$$

$$f_v / F_v' = 0.733$$

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection

$$E' = E \times C_M \times C_t = 2000000 \text{ lb/in}^2$$

Design deflection

$$\delta_{adm} = 0.003 \times L_{s1} = 0.324 \text{ in}$$

Total deflection

$$\delta_{b_{s1}} = 0.304 \text{ in}$$

$$\delta_{b_{s1}} / \delta_{adm} = 0.938$$

PASS - Total deflection is less than design deflection

COLUMN DESIGN

WOOD MEMBER DESIGN (NDS 2018)

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

Tedds calculation version 2.2.13

Design section 1

Member details

Service condition

Dry

Load duration - Table 2.3.2

Ten years

Sawn lumber section details

Number of sections in member

N = 2

Nominal breadth of sections

b_{nom} = 2 in

Breadth of sections

b = 1.5 in

Nominal depth of sections

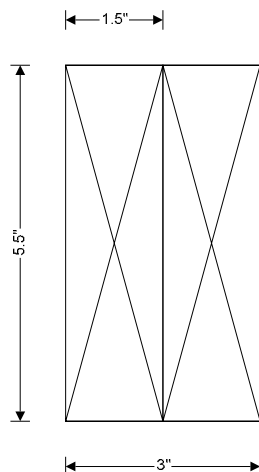
d_{nom} = 6 in

Depth of sections

d = 5.5 in

Material

Spruce-Pine-Fir, 2" && wider, No.2 grade



2/2"x6" sawn lumber sections

Cross-sectional area, A, 16.5 in²

Section modulus, S_x, 15.1 in³

Section modulus, S_y, 8.3 in³

Second moment of area, I_x, 41.6 in⁴

Second moment of area, I_y, 12.4 in⁴

Radius of gyration, r_x, 1.588 in

Radius of gyration, r_y, 0.866 in

Spruce-Pine-Fir, 2" && wider, No.2 grade

Bending, F_b, 875 psi

Shear parallel to grain, F_v, 135 psi

Compression parallel to grain, F_c, 1150 psi

Compression perpendicular to grain, F_{c_perp}, 425 psi


Tension parallel to grain, F_t, 450 psi

Modulus of elasticity, E, 1400000 psi

Minimum modulus of elasticity, E_{min}, 510000 psi

Density, ρ, 29.098 lbm/ft³

Specific gravity, G, 0.42

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Span details

Unbraced length - Major axis $L_x = 10$ ft
 Effective bending length - Major axis $L_{e,x} = 1.63 \times L_x + 3 \times N \times b = 17.05$ ft
 Column buckling length - Major axis $L_{b,x} = L_x = 10$ ft
 Unbraced length - Minor axis $L_y = 0$ ft

Analysis results

Design axial compression force $P = 5000$ lb

Section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Compressive stress	lb/in ²	704	303	0.430	PASS

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2 $C_D = 1$
 Size factor for compression - Table 4A $C_{F_c} = 1.1$
 Reference compression design value $F_c^* = F_c \times C_D \times C_{F_c} = 1265$ lb/in²
 Adjusted modulus of elasticity $E_{min}^* = E_{min} = 510000$ lb/in²
 Critical buckling design value $F_{cE} = 0.822 \times E_{min}^* / (L_{b,x} / d)^2 = 881$ lb/in²
 Column stability factor - eq.3.7-1

$$C_P = (1 + (F_{cE} / F_c^*)) / 1.6 - \sqrt{((1 + (F_{cE} / F_c^*)) / 1.6)^2 - (F_{cE} / F_c^*) / 0.8} = 0.557$$

Compression members - General - cl.3.6

Design axial compression force $P = 5000$ lb
 Design compression parallel to grain - Table 4.3.1 $F_c' = F_c \times C_D \times C_{F_c} \times C_P = 704$ lb/in²
 Actual compression parallel to grain $f_c = P / (N \times b \times d) = 303$ lb/in²
 $f_c / F_c' = 0.430$

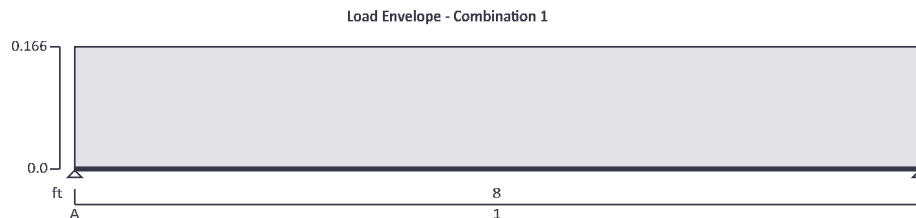
PASS - Design compression stress exceeds actual compression stress

WOOD BEAM ANALYSIS & DESIGN (NDS)

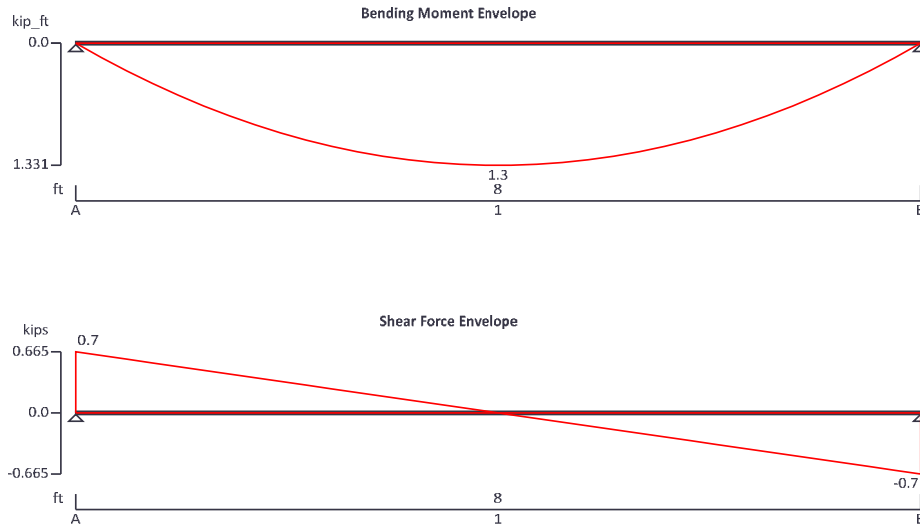
STRUCTURAL WOOD MEMBER ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2015 using the ASD method

Tedds calculation version 1.7.10



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Applied loading

Beam loads

Dead self weight of beam $\times 1$
 Dead full UDL 40 lb/ft
 Live full UDL 80 lb/ft
 Snow full UDL 40 lb/ft

Load combinations

Load combination 1

Support A	Dead $\times 1.00$
	Live $\times 1.00$
	Snow $\times 1.00$
Span 1	Dead $\times 1.00$
	Live $\times 1.00$
	Snow $\times 1.00$
Support B	Dead $\times 1.00$
	Live $\times 1.00$
	Snow $\times 1.00$

Analysis results

Maximum moment	$M_{max} = 1331 \text{ lb_ft}$	$M_{min} = 0 \text{ lb_ft}$
Design moment	$M = \max(\text{abs}(M_{max}), \text{abs}(M_{min})) = 1331 \text{ lb_ft}$	
Maximum shear	$F_{max} = 665 \text{ lb}$	$F_{min} = -665 \text{ lb}$
Design shear	$F = \max(\text{abs}(F_{max}), \text{abs}(F_{min})) = 665 \text{ lb}$	
Total load on member	$W_{tot} = 1331 \text{ lb}$	
Reaction at support A	$R_{A_max} = 665 \text{ lb}$	$R_{A_min} = 665 \text{ lb}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 185 \text{ lb}$	
Unfactored live load reaction at support A	$R_{A_Live} = 320 \text{ lb}$	
Unfactored snow load reaction at support A	$R_{A_Snow} = 160 \text{ lb}$	

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Reaction at support B

$R_{B_max} = 665 \text{ lb}$

$R_{B_min} = 665 \text{ lb}$

Unfactored dead load reaction at support B

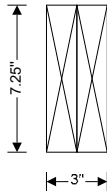
$R_{B_Dead} = 185 \text{ lb}$

Unfactored live load reaction at support B

$R_{B_Live} = 320 \text{ lb}$

Unfactored snow load reaction at support B

$R_{B_Snow} = 160 \text{ lb}$



Sawn lumber section details

Nominal breadth of sections	$b_{nom} = 2 \text{ in}$
Dressed breadth of sections	$b = 1.5 \text{ in}$
Nominal depth of sections	$d_{nom} = 8 \text{ in}$
Dressed depth of sections	$d = 7.25 \text{ in}$
Number of sections in member	$N = 2$
Overall breadth of member	$b_b = N \times b = 3 \text{ in}$
Species, grade and size classification	Spruce-Pine-Fir, No.2 grade, 2" & wider
Bending parallel to grain	$F_b = 875 \text{ lb/in}^2$
Tension parallel to grain	$F_t = 450 \text{ lb/in}^2$
Compression parallel to grain	$F_c = 1150 \text{ lb/in}^2$
Compression perpendicular to grain	$F_{c_perp} = 425 \text{ lb/in}^2$
Shear parallel to grain	$F_v = 135 \text{ lb/in}^2$
Modulus of elasticity	$E = 1400000 \text{ lb/in}^2$
Modulus of elasticity, stability calculations	$E_{min} = 510000 \text{ lb/in}^2$
Mean shear modulus	$G_{def} = E / 16 = 87500 \text{ lb/in}^2$

Member details

Service condition	Dry
Length of span	$L_{s1} = 8 \text{ ft}$
Length of bearing	$L_b = 4 \text{ in}$
Load duration	Ten years

Section properties

Cross sectional area of member	$A = N \times b \times d = 21.75 \text{ in}^2$
Section modulus	$S_x = N \times b \times d^2 / 6 = 26.28 \text{ in}^3$
	$S_y = d \times (N \times b)^2 / 6 = 10.87 \text{ in}^3$
Second moment of area	$I_x = N \times b \times d^3 / 12 = 95.27 \text{ in}^4$
	$I_y = d \times (N \times b)^3 / 12 = 16.31 \text{ in}^4$

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	$C_{Fb} = 1.20$
Size factor for tension - Table 4A	$C_{Ft} = 1.20$

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Size factor for compression - Table 4A $C_{Fc} = 1.05$

Flat use factor - Table 4A $C_{fu} = 1.15$

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

$$C_r = 1.00$$

Bearing area factor - cl.3.10.4

$$C_b = 1.00$$

Depth-to-breadth ratio

$$d_{nom} / (N \times b_{nom}) = 2.00$$

- 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 $F_{c_perp}' = F_{c_perp} \times C_t \times C_{ic_perp} \times C_b = 425 \text{ lb/in}^2$

Applied compression stress perpendicular to grain $f_{c_perp} = R_{A_max} / (N \times b \times L_b) = 55 \text{ lb/in}^2$

$$f_{c_perp} / F_{c_perp}' = 0.130$$

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress

$$F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = 1050 \text{ lb/in}^2$$

Actual bending stress

$$f_b = M / S_x = 608 \text{ lb/in}^2$$

$$f_b / F_b' = 0.579$$

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress

$$F_v' = F_v \times C_D \times C_t \times C_i = 135 \text{ lb/in}^2$$

Actual shear stress - eq.3.4-2

$$f_v = 3 \times F / (2 \times A) = 46 \text{ lb/in}^2$$

$$f_v / F_v' = 0.340$$

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection

$$E' = E \times C_{ME} \times C_t \times C_{iE} = 1400000 \text{ lb/in}^2$$

Design deflection

$$\delta_{adm} = 0.003 \times L_{s1} = 0.288 \text{ in}$$

Total deflection

$$\delta_{b_s1} = 0.115 \text{ in}$$

$$\delta_{b_s1} / \delta_{adm} = 0.399$$

PASS - Total deflection is less than design deflection