GL Architectural Engr
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Project				Job Ref.	
Bravern Residence				1932	
Section				Sheet no./rev.	
				1	
Calc. by	Date	Chk'd by	Date	App'd by	Date
GL	06.18.19				

STRUCTURAL CALCULATIONS

FOR

THE

Bravern Residence

Located at

12607 14th Ave S

Burien, Washington 98146

TABLE OF CONTENTS

DEAD LOAD CONSTRUCTION

Roof Assembly

Material	Thickness (in)	γ (lb/ft^3)	Weight (lb/ft^2)
Asphalt Shingles;	0.250;	135;	2.8
1/2" Plywood or OSB;	0.500;	45;	1.9
Insulation;	12.000;	1;	1.0
2x Rafters;	1.000;	35;	2.9
Beams;	0.500;	35;	1.5
Gypsum Board;	0.625;	60;	3.1
Miscellaneous;	1.000;	;	1.8
Totals;			**15.0**

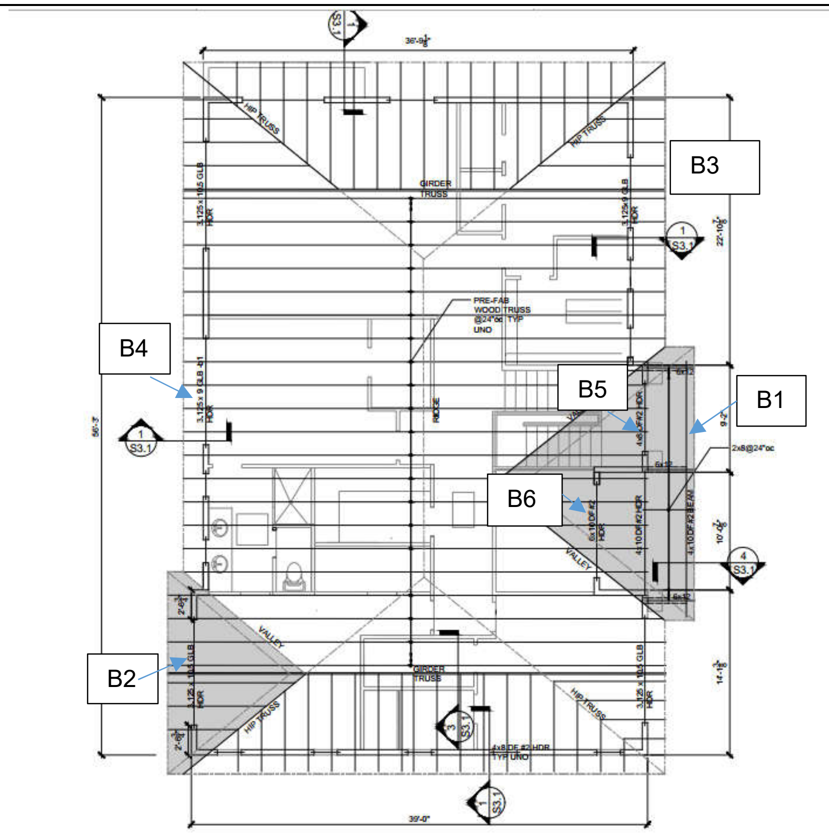
DESIGN CRITERIA FLOOR ASSEMBLY

DEAD LOAD CONSTRUCTION

Floor Assembly

Material	Thickness (in)	γ (lb/ft^3)	Weight (lb/ft^2)
Flooring;	0.125;	95;	1.0
3/4" Plywood or OSB;	0.750;	45;	2.8
2x Joists;	0.625;	35;	1.8
Beams;	0.600;	35;	1.7
Gypsum Board;	0.625;	60;	3.1
Miscellaneous;	1.000;	;	1.5
Totals;	**3.725;**		**12.0**

;Live Load: 40 PSF
WIND SPEED: 110 MPH, EXPOSURE B.
SEISMIC : SDC D, RISK CATEGORY II.

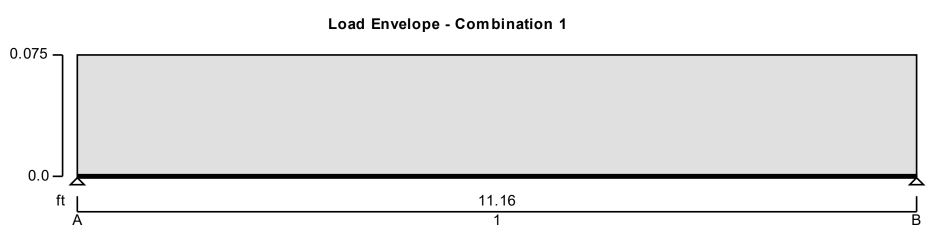


B1

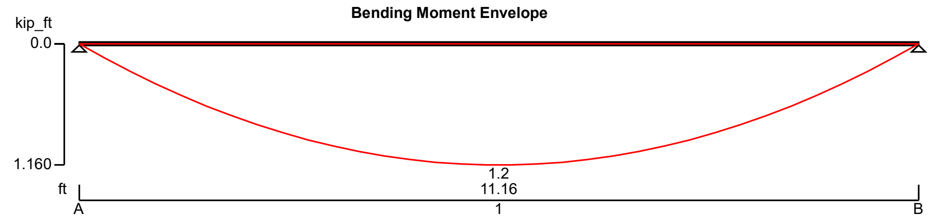
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS)

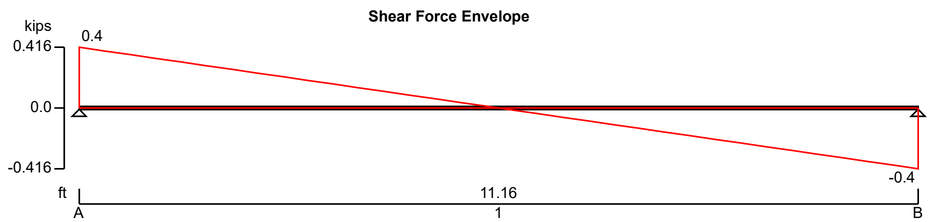
In accordance with the ANSI/AF&PA NDS-2015 using the ASD method

TEDDS calculation version 1.7.03



GUIBIN LU, PE PO Box 1040, Tacoma, WA 98401-1040 Email: akegl2002@gmail.com Ph: (360)747-7509	Project Sun Residence Mercer Island			Job Ref. 1956	
	Section			Sheet no./rev. 6	
	Calc. by GL	Date 01-05-2020	Chk'd by	Date	App'd by	Date





Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 25 lb/ft

Snow full UDL 42 lb/ft

Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00
		Snow × 1.00
	Span 1	Dead × 1.00
		Live × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Live × 1.00
		Snow × 1.00

Analysis results

Maximum moment;	M_{max} = **1160** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **1160** lb_ft	
Maximum shear;	F_{max} = **416** lb;	F_{min} = **-416** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **416** lb	
Total load on member;	W_{tot} = **832** lb	
Reaction at support A;	R_{A_max} = **416** lb;	R_{A_min} = **416** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **184** lb	
Unfactored snow load reaction at support A;	R_{A_Snow} = **232** lb	
Reaction at support B;	R_{B_max} = **416** lb;	R_{B_min} = **416** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **184** lb	

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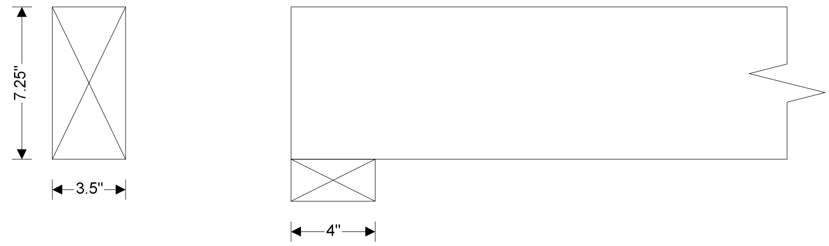
Unfactored snow load reaction at support B; R_{B_Snow} = **232** lb



Sawn lumber section details

Nominal breadth of sections; b_{nom} = **4** in
Dressed breadth of sections; b = **3.5** in
Nominal depth of sections; d_{nom} = **8** in
Dressed depth of sections; d = **7.25** in
Number of sections in member; N = **1**
Overall breadth of member; $b_b = N \times b$ = **3.5** in
Species, grade and size classification; Hem-Fir, No.2 grade, 2" & wider
Bending parallel to grain; F_b = **850** lb/in^2
Tension parallel to grain; F_t = **525** lb/in^2
Compression parallel to grain; F_c = **1300** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **405** lb/in^2
Shear parallel to grain; F_v = **150** lb/in^2
Modulus of elasticity; E = **1300000** lb/in^2
Modulus of elasticity, stability calculations; E_{min} = **470000** lb/in^2
Mean shear modulus; G_{def} = E / 16 = **81250** lb/in^2

Member details

Service condition; **Dry**
Length of span; L_{s1} = **11.16** ft
Length of bearing; L_b = **4** in
Load duration; **Two months**

Section properties

Cross sectional area of member; $A = N \times b \times d$ = **25.37** in^2
Section modulus; $S_x = N \times b \times d^2 / 6$ = **30.66** in^3
$S_y = d \times (N \times b)^2 / 6$ = **14.80** in^3
Second moment of area; $I_x = N \times b \times d^3 / 12$ = **111.15** in^4
$I_y = d \times (N \times b)^3 / 12$ = **25.90** in^4

Adjustment factors

Load duration factor - Table 2.3.2; C_D = **1.15**
Temperature factor - Table 2.3.3; C_t = **1.00**
Size factor for bending - Table 4A; C_{Fb} = **1.30**
Size factor for tension - Table 4A; C_{Ft} = **1.20**
Size factor for compression - Table 4A; C_{Fc} = **1.05**
Flat use factor - Table 4A; C_{fu} = **1.05**
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; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b$ = **405** lb/in^2

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b)$ = **30** lb/in^2

f_{c_perp} / F_{c_perp}' = **0.073**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1271** lb/in^2

Actual bending stress; $f_b = M / S_x$ = **454** lb/in^2

f_b / F_b' = **0.357**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_i$ = **173** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A)$ = **25** lb/in^2

f_v / F_v' = **0.143**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection; $E' = E \times C_{ME} \times C_t \times C_{iE}$ = **1300000** lb/in^2

Design deflection; $\delta_{adm} = 0.0042 \times L_{s1}$ = **0.562** in

Total deflection; δ_{b_s1} = **0.180** in

$\delta_{b\_s1} / \delta_{adm}$ = **0.320**

PASS - Total deflection is less than design deflection

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B2

STRUCTURAL GLUED LAMINATED TIMBER (GLULAM) BEAM ANALYSIS & DESIGN (NDS)

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

TEDDS calculation version 1.7.03

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Project				Job Ref.	
Sun Residence Mercer Island				1956	
Section				Sheet no./rev.	
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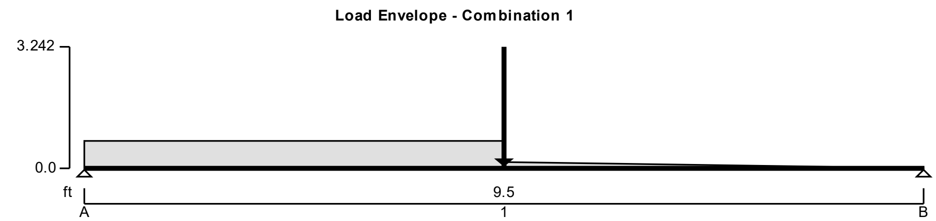
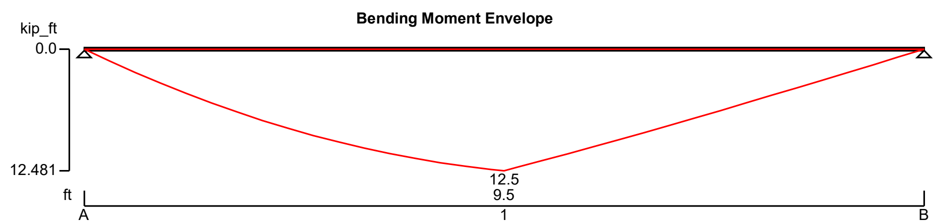
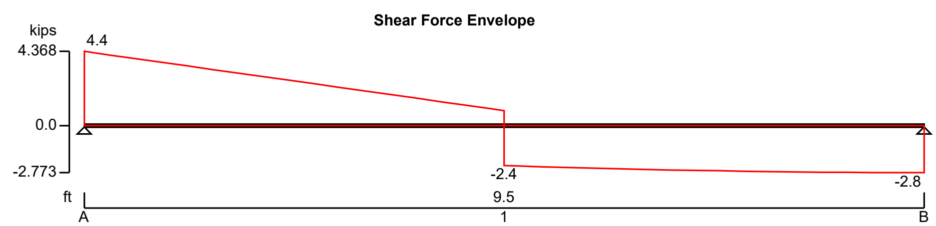
Calc. by	Date	Chk'd by	Date	App'd by	Date
GL	01-05-2020				







Applied loading

Beam loads

Dead self weight of beam × 1
Dead partial UDL 272 lb/ft from 0.00 in to 57.00 in
Snow partial UDL 453 lb/ft from 0.00 in to 57.00 in
Dead partial VDL 60 lb/ft at 57.00 in to 0 lb/ft at 114.00 in
Snow partial VDL 100 lb/ft at 57.00 in to 0 lb/ft at 114.00 in
Dead point load 1389 lb at 57.00 in
Snow point load 1853 lb at 57.00 in

Load combinations

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

Analysis results

Maximum moment; M_{max} = **12481** lb_ft; M_{min} = **0** lb_ft

Design moment; $M = \max(\mathrm{abs}(M_{max}),\mathrm{abs}(M_{min}))$ = **12481** lb_ft

Maximum shear; F_{max} = **4368** lb; F_{min} = **-2773** lb

Design shear; $F = \max(\mathrm{abs}(F_{max}),\mathrm{abs}(F_{min}))$ = **4368** lb

Total load on member; W_{tot} = **7141** lb

Reaction at support A; R_{A_max} = **4368** lb; R_{A_min} = **4368** lb

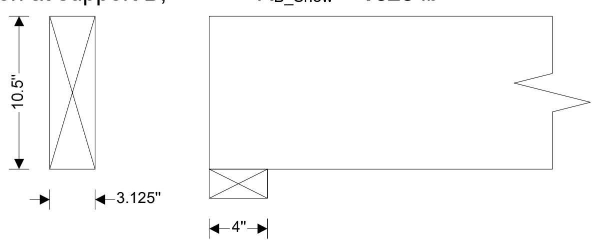
Unfactored dead load reaction at support A; R_{A_Dead} = **1749** lb

Unfactored snow load reaction at support A; R_{A_Snow} = **2620** lb

Reaction at support B; R_{B_max} = **2773** lb; R_{B_min} = **2773** lb

Unfactored dead load reaction at support B; R_{B_Dead} = **1150** lb

Unfactored snow load reaction at support B; R_{B_Snow} = **1623** lb



Glulam section details

Net finished breadth of sections; b = **3.125** in

Net finished depth of sections; d = **10.5** in

Number of sections in member; N = **1**

Overall breadth of member; $b_b = N \times b$ = **3.125** in

Alignment of laminations; **Horizontal**

Stress class; 24F-V4 DF/DF

Tension parallel to grain; F_t = **1100** lb/in^2

Compression parallel to grain; F_c = **1650** lb/in^2

Bending about X-X axis properties (loaded perpendicular to wide faces of laminations):

Positive bending; F_{bx_pos} = **2400** lb/in^2

Negative bending; F_{bx_neg} = **1850** lb/in^2

Compression perpendicular to grain; F_{c_perp} = **650** lb/in^2

Shear parallel to grain; F_v = **265** lb/in^2

Modulus of elasticity; E = **1800000** lb/in^2

Modulus of elasticity, stability calculations; E_{min} = **950000** lb/in^2

Mean shear modulus; $G_{def} = E / 16$ = **112500** lb/in^2

Bending about Y-Y axis properties (loaded parallel to wide faces of laminations):

Bending; F_{by} = **1450** lb/in^2

Modulus of elasticity; stability calculations; E_{ymin} = **850000** lb/in^2

Member details

Service condition; **Dry**

Length of span; L_{s1} = **9.5** ft

Length of bearing; L_b = **4** in

Load duration; **Two months**

Section properties

Cross sectional area of member; $A = N \times b \times d =$ **32.81** in^2

Section modulus; $S_x = N \times b \times d^2 / 6 =$ **57.42** in^3

$S_y = d \times (N \times b)^2 / 6 =$ **17.09** in^3

Second moment of area; $I_x = N \times b \times d^3 / 12 =$ **301.46** in^4

$I_y = d \times (N \times b)^3 / 12 =$ **26.70** in^4

Adjustment factors

Load duration factor - Table 2.3.2; $C_D =$ **1.15**

Temperature factor - Table 2.3.3; $C_t =$ **1.00**

Flat use factor - Table 5A; $C_{fu} =$ **1.16**

Bearing area factor - cl.3.10.4; $C_b =$ **1.00**

Length of beam between points of zero moment; $L_0 =$ **9.5** ft

For species other than Southern Pine; x = **10**

Volume factor - eq.5.3-1; $C_V = \min((21\text{ ft} / L_0)^{1/x} \times (12\text{ in} / d)^{1/x} \times (5.125\text{ in} / b)^{1/x}, 1) =$ **1.00**

Depth-to-breadth ratio; $d / (N \times b) =$ **3.36**

- 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; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_b =$ **650** lb/in^2

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b) =$ **349** lb/in^2

$f_{c\_perp} / F_{c\_perp}' =$ **0.538**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_{bx\_pos} \times C_D \times C_t \times \min(C_L, C_V) \times C_c =$ **2760** lb/in^2

Actual bending stress; $f_b = M_{max} / S_x =$ **2608** lb/in^2

$f_b / F_b' =$ **0.945**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t =$ **305** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A) =$ **200** lb/in^2

$f_v / F_v' =$ **0.655**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection; $E' = E_x \times C_{ME} \times C_t =$ **1800000** lb/in^2

Design deflection; $\delta_{adm} = 0.0042 \times L_{s1} =$ **0.479** in

Total deflection; $\delta_{b\_s1} =$ **0.327** in

$\delta_{b\_s1} / \delta_{adm} =$ **0.683**

PASS - Total deflection is less than design deflection

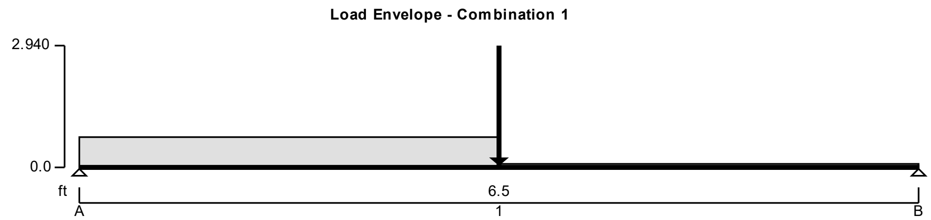
;

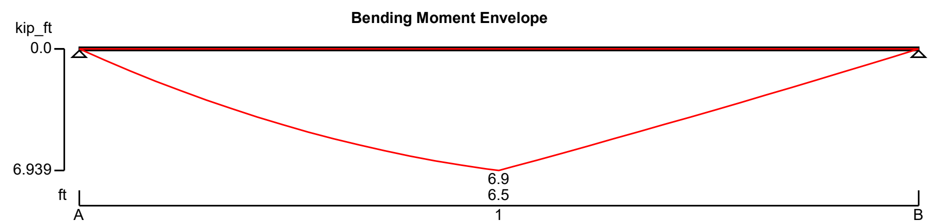
B3

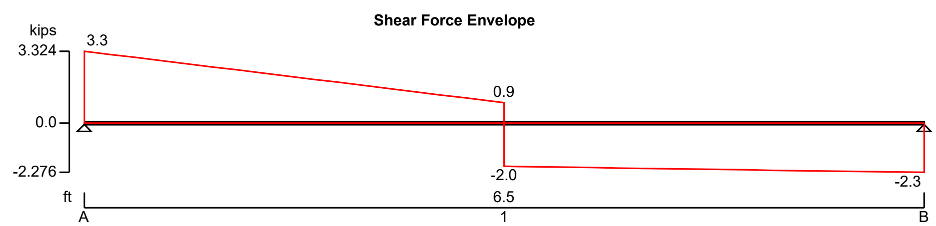
STRUCTURAL GLUED LAMINATED TIMBER (GLULAM) BEAM ANALYSIS & DESIGN (NDS)

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

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1
Dead partial UDL 272 lb/ft from 0.00 in to 39.00 in
Snow partial UDL 453 lb/ft from 0.00 in to 39.00 in
Dead partial UDL 30 lb/ft from 39.00 in to 78.00 in
Snow partial UDL 50 lb/ft from 39.00 in to 78.00 in
Dead point load 1103 lb at 39.00 in
Snow point load 1838 lb at 39.00 in

Load combinations

Load combination 1	Support A	Dead × 1.00
		Snow × 1.00
	Span 1	Dead × 1.00

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		Snow × 1.00
	Support B	Dead × 1.00
		Snow × 1.00

Analysis results

Maximum moment; M_{max} = **6939** lb_ft; M_{min} = **0** lb_ft

Design moment; M = max(abs(M_{max}),abs(M_{min})) = **6939** lb_ft

Maximum shear; F_{max} = **3324** lb; F_{min} = **-2276** lb

Design shear; F = max(abs(F_{max}),abs(F_{min})) = **3324** lb

Total load on member; W_{tot} = **5601** lb

Reaction at support A; R_{A_max} = **3324** lb; R_{A_min} = **3324** lb

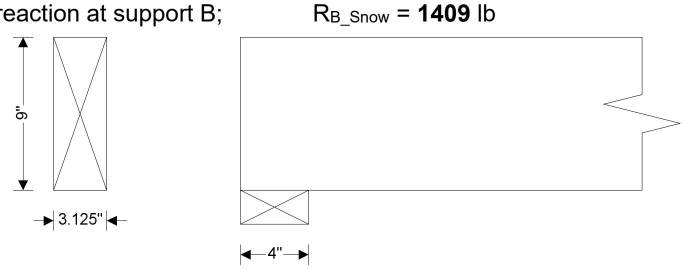
Unfactored dead load reaction at support A; R_{A_Dead} = **1261** lb

Unfactored snow load reaction at support A; R_{A_Snow} = **2064** lb

Reaction at support B; R_{B_max} = **2276** lb; R_{B_min} = **2276** lb

Unfactored dead load reaction at support B; R_{B_Dead} = **867** lb

Unfactored snow load reaction at support B; R_{B_Snow} = **1409** lb



Glulam section details

Net finished breadth of sections; b = **3.125** in

Net finished depth of sections; d = **9** in

Number of sections in member; N = **1**

Overall breadth of member; b_b = N × b = **3.125** in

Alignment of laminations; **Horizontal**

Stress class; 24F-V4 DF/DF

Tension parallel to grain; F_t = **1100** lb/in^2

Compression parallel to grain; F_c = **1650** lb/in^2

Bending about X-X axis properties (loaded perpendicular to wide faces of laminations):

Positive bending; F_{bx_pos} = **2400** lb/in^2

Negative bending; F_{bx_neg} = **1850** lb/in^2

Compression perpendicular to grain; F_{c_perp} = **650** lb/in^2

Shear parallel to grain; F_v = **265** lb/in^2

Modulus of elasticity; E = **1800000** lb/in^2

Modulus of elasticity, stability calculations; E_{min} = **950000** lb/in^2

Mean shear modulus; G_{def} = E / 16 = **112500** lb/in^2

Bending about Y-Y axis properties (loaded parallel to wide faces of laminations):

Bending; F_{by} = **1450** lb/in^2

Modulus of elasticity; stability calculations; E_{ymin} = **850000** lb/in^2

Member details

Service condition;	**Dry**
Length of span;	L_{s1} = **6.5** ft
Length of bearing;	L_b = **4** in
Load duration;	**Two months**

Section properties

Cross sectional area of member;	$A = N \times b \times d$ = **28.13** in^2
Section modulus;	$S_x = N \times b \times d^2 / 6$ = **42.19** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **14.65** in^3
Second moment of area;	$I_x = N \times b \times d^3 / 12$ = **189.84** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **22.89** in^4

Adjustment factors

Load duration factor - Table 2.3.2;	C_D = **1.15**
Temperature factor - Table 2.3.3;	C_t = **1.00**
Flat use factor - Table 5A;	C_{fu} = **1.16**
Bearing area factor - cl.3.10.4;	C_b = **1.00**
Length of beam between points of zero moment;	L_0 = **6.5** ft
For species other than Southern Pine;	x = **10**
Volume factor - eq.5.3-1;	$C_V = \min((21\text{ ft} / L_0)^{1/x} \times (12\text{ in} / d)^{1/x} \times (5.125\text{ in} / b)^{1/x}, 1)$ = **1.00**
Depth-to-breadth ratio;	$d / (N \times b)$ = **2.88**
- 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;	$F_{c\_perp}' = F_{c\_perp} \times C_t \times C_b$ = **650** lb/in^2
Applied compression stress perpendicular to grain;	$f_{c\_perp} = R_{A\_max} / (N \times b \times L_b)$ = **266** lb/in^2
	f_{c_perp} / F_{c_perp}' = **0.409**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress;	$F_b' = F_{bx\_pos} \times C_D \times C_t \times \min(C_L, C_V) \times C_c$ = **2760** lb/in^2
Actual bending stress;	$f_b = M_{max} / S_x$ = **1974** lb/in^2
	f_b / F_b' = **0.715**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress;	$F_v' = F_v \times C_D \times C_t$ = **305** lb/in^2
Actual shear stress - eq.3.4-2;	$f_v = 3 \times F / (2 \times A)$ = **177** lb/in^2
	f_v / F_v' = **0.582**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection;	$E' = E_x \times C_{ME} \times C_t$ = **1800000** lb/in^2
Design deflection;	$\delta_{adm} = 0.0042 \times L_{s1}$ = **0.328** in
Total deflection;	δ_{b_s1} = **0.133** in
	$\delta_{b\_s1} / \delta_{adm}$ = **0.407**

PASS - Total deflection is less than design deflection

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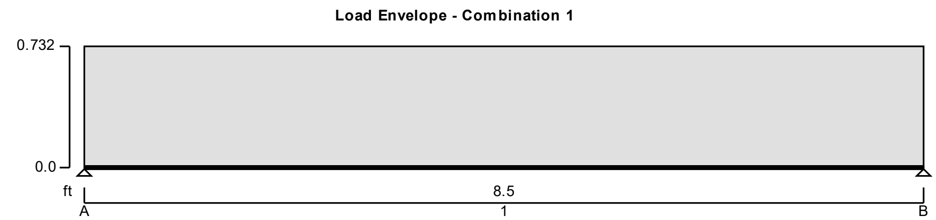
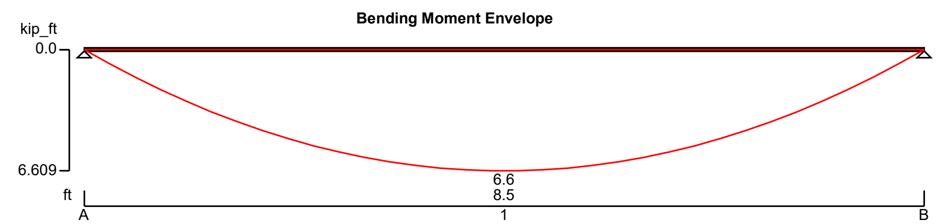
B4

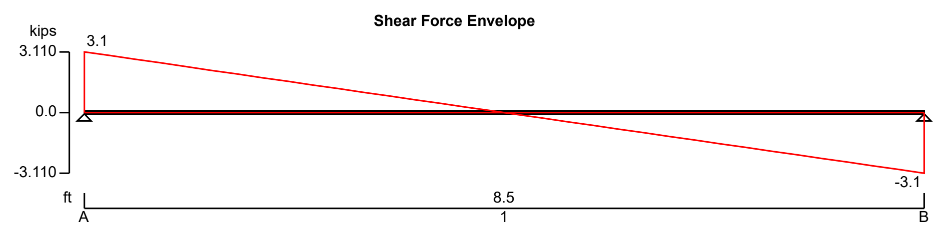
STRUCTURAL GLUED LAMINATED TIMBER (GLULAM) BEAM ANALYSIS & DESIGN (NDS)

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

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 272 lb/ft

Snow full UDL 453 lb/ft

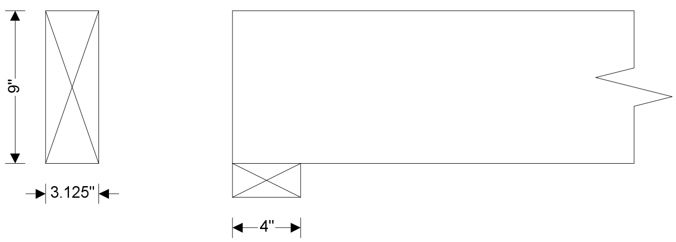
Load combinations

Load combination 1	Support A	Dead × 1.00
		Snow × 1.00
	Span 1	Dead × 1.00

		Snow × 1.00
	Support B	Dead × 1.00
		Snow × 1.00

Analysis results

Maximum moment;	M_{max} = **6609** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **6609** lb_ft	
Maximum shear;	F_{max} = **3110** lb;	F_{min} = **-3110** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **3110** lb	
Total load on member;	W_{tot} = **6221** lb	
Reaction at support A;	R_{A_max} = **3110** lb;	R_{A_min} = **3110** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **1185** lb	
Unfactored snow load reaction at support A;	R_{A_Snow} = **1926** lb	
Reaction at support B;	R_{B_max} = **3110** lb;	R_{B_min} = **3110** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **1185** lb	
Unfactored snow load reaction at support B;	R_{B_Snow} = **1926** lb	



Glulam section details

Net finished breadth of sections;	b = **3.125** in
Net finished depth of sections;	d = **9** in
Number of sections in member;	N = **1**
Overall breadth of member;	b_b = N × b = **3.125** in
Alignment of laminations;	**Horizontal**
Stress class;	24F-V4 DF/DF
Tension parallel to grain;	F_t = **1100** lb/in^2
Compression parallel to grain;	F_c = **1650** lb/in^2

Bending about X-X axis properties (loaded perpendicular to wide faces of laminations):

Positive bending;	F_{bx_pos} = **2400** lb/in^2
Negative bending;	F_{bx_neg} = **1850** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **650** lb/in^2
Shear parallel to grain;	F_v = **265** lb/in^2
Modulus of elasticity;	E = **1800000** lb/in^2
Modulus of elasticity, stability calculations;	E_{min} = **950000** lb/in^2
Mean shear modulus;	G_{def} = E / 16 = **112500** lb/in^2

Bending about Y-Y axis properties (loaded parallel to wide faces of laminations):

Bending;	F_{by} = **1450** lb/in^2
Modulus of elasticity; stability calculations;	E_{ymin} = **850000** lb/in^2

Member details

Service condition;	**Dry**
Length of span;	L_{s1} = **8.5** ft
Length of bearing;	L_b = **4** in
Load duration;	**Two months**

Section properties

Cross sectional area of member;	$A = N \times b \times d$ = **28.13** in^2
Section modulus;	$S_x = N \times b \times d^2 / 6$ = **42.19** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **14.65** in^3
Second moment of area;	$I_x = N \times b \times d^3 / 12$ = **189.84** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **22.89** in^4

Adjustment factors

Load duration factor - Table 2.3.2;	C_D = **1.15**
Temperature factor - Table 2.3.3;	C_t = **1.00**
Flat use factor - Table 5A;	C_{fu} = **1.16**
Bearing area factor - cl.3.10.4;	C_b = **1.00**
Length of beam between points of zero moment;	L_0 = **8.5** ft
For species other than Southern Pine;	x = **10**
Volume factor - eq.5.3-1;	$C_V = \min((21 \text{ ft} / L_0)^{1/x} \times (12 \text{ in} / d)^{1/x} \times (5.125 \text{ in} / b)^{1/x}, 1)$ = **1.00**
Depth-to-breadth ratio;	$d / (N \times b)$ = **2.88**
- 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;	$F_{c\_perp}' = F_{c\_perp} \times C_t \times C_b$ = **650** lb/in^2
Applied compression stress perpendicular to grain;	$f_{c\_perp} = R_{A\_max} / (N \times b \times L_b)$ = **249** lb/in^2
	f_{c_perp} / F_{c_perp}' = **0.383**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress;	$F_b' = F_{bx\_pos} \times C_D \times C_t \times \min(C_L, C_V) \times C_c$ = **2760** lb/in^2
Actual bending stress;	$f_b = M_{max} / S_x$ = **1880** lb/in^2
	f_b / F_b' = **0.681**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress;	$F_v' = F_v \times C_D \times C_t$ = **305** lb/in^2
Actual shear stress - eq.3.4-2;	$f_v = 3 \times F / (2 \times A)$ = **166** lb/in^2
	f_v / F_v' = **0.544**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection;	$E' = E_x \times C_{ME} \times C_t$ = **1800000** lb/in^2
Design deflection;	$\delta_{adm} = 0.0042 \times L_{s1}$ = **0.428** in
Total deflection;	δ_{b_s1} = **0.252** in
	$\delta_{b\_s1} / \delta_{adm}$ = **0.587**

PASS - Total deflection is less than design deflection

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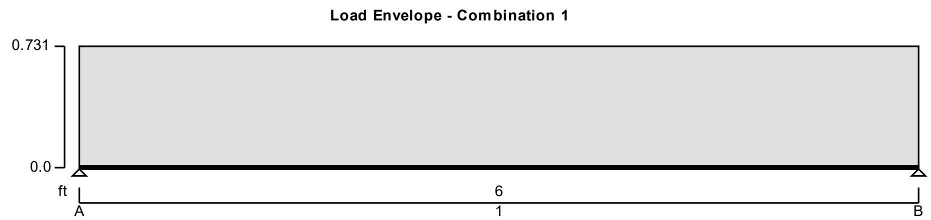
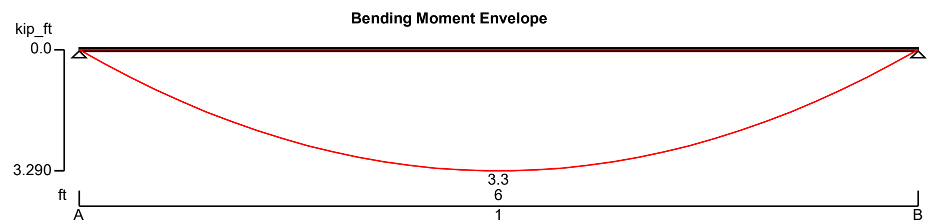
B5

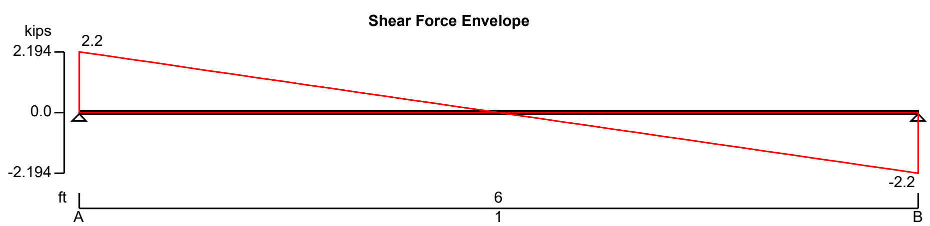
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS)

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

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 272 lb/ft
Snow full UDL 453 lb/ft

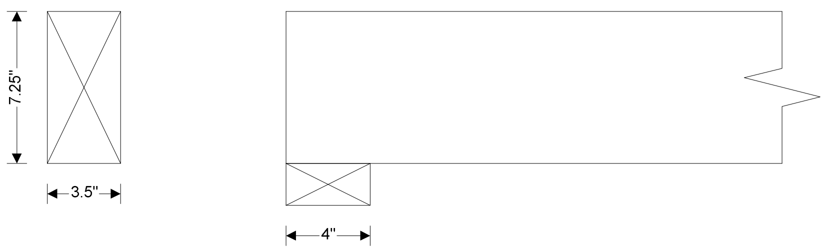
Load combinations

Load combination 1	Support A	Dead × 1.00
		Snow × 1.00
	Span 1	Dead × 1.00

		Snow × 1.00
	Support B	Dead × 1.00
		Snow × 1.00

Analysis results

Maximum moment;	M_{max} = **3290** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **3290** lb_ft	
Maximum shear;	F_{max} = **2194** lb;	F_{min} = **-2194** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **2194** lb	
Total load on member;	W_{tot} = **4387** lb	
Reaction at support A;	R_{A_max} = **2194** lb;	R_{A_min} = **2194** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **834** lb	
Unfactored snow load reaction at support A;	R_{A_Snow} = **1359** lb	
Reaction at support B;	R_{B_max} = **2194** lb;	R_{B_min} = **2194** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **834** lb	
Unfactored snow load reaction at support B;	R_{B_Snow} = **1359** lb	



Sawn lumber section details

Nominal breadth of sections;	b_{nom} = **4** in
Dressed breadth of sections;	b = **3.5** in
Nominal depth of sections;	d_{nom} = **8** in
Dressed depth of sections;	d = **7.25** in
Number of sections in member;	N = **1**
Overall breadth of member;	b_b = N × b = **3.5** in
Species, grade and size classification;	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain;	F_b = **900** lb/in^2
Tension parallel to grain;	F_t = **575** lb/in^2
Compression parallel to grain;	F_c = **1350** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **625** lb/in^2
Shear parallel to grain;	F_v = **180** lb/in^2
Modulus of elasticity;	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations;	E_{min} = **580000** lb/in^2
Mean shear modulus;	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition;	**Dry**
Length of span;	L_{s1} = **6** ft
Length of bearing;	L_b = **4** in
Load duration;	**Two months**

Section properties

Cross sectional area of member; $A = N \times b \times d = \mathbf{25.37}$ in^2

Section modulus; $S_x = N \times b \times d^2 / 6 = \mathbf{30.66}$ in^3

$S_y = d \times (N \times b)^2 / 6 = \mathbf{14.80}$ in^3

Second moment of area; $I_x = N \times b \times d^3 / 12 = \mathbf{111.15}$ in^4

$I_y = d \times (N \times b)^3 / 12 = \mathbf{25.90}$ in^4

Adjustment factors

Load duration factor - Table 2.3.2; $C_D = \mathbf{1.15}$

Temperature factor - Table 2.3.3; $C_t = \mathbf{1.00}$

Size factor for bending - Table 4A; $C_{Fb} = \mathbf{1.30}$

Size factor for tension - Table 4A; $C_{Ft} = \mathbf{1.20}$

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

Flat use factor - Table 4A; $C_{fu} = \mathbf{1.05}$

Incising factor for modulus of elasticity - Table 4.3.8

$C_{iE} = \mathbf{1.00}$

Incising factor for bending, shear, tension & compression - Table 4.3.8

$C_i = \mathbf{1.00}$

Incising factor for perpendicular compression - Table 4.3.8

$C_{ic\_perp} = \mathbf{1.00}$

Repetitive member factor - cl.4.3.9; $C_r = \mathbf{1.00}$

Bearing area factor - cl.3.10.4; $C_b = \mathbf{1.00}$

Depth-to-breadth ratio; $d_{nom} / (N \times b_{nom}) = \mathbf{2.00}$

- Beam is fully restrained

Beam stability factor - cl.3.3.3; $C_L = \mathbf{1.00}$

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b = \mathbf{625}$ lb/in^2

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b) = \mathbf{157}$ lb/in^2

$f_{c\_perp} / F_{c\_perp}' = \mathbf{0.251}$

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = \mathbf{1346}$ lb/in^2

Actual bending stress; $f_b = M / S_x = \mathbf{1288}$ lb/in^2

$f_b / F_b' = \mathbf{0.957}$

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_i = \mathbf{207}$ lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A) = \mathbf{130}$ lb/in^2

$f_v / F_v' = \mathbf{0.626}$

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection; $E' = E \times C_{ME} \times C_t \times C_{iE} = \mathbf{1600000}$ lb/in^2

Design deflection; $\delta_{adm} = 0.0042 \times L_{s1} = \mathbf{0.302}$ in

Total deflection; δ_{b_s1} = **0.120** in

δ_{b_s1} / δ_{adm} = **0.396**

PASS - Total deflection is less than design deflection

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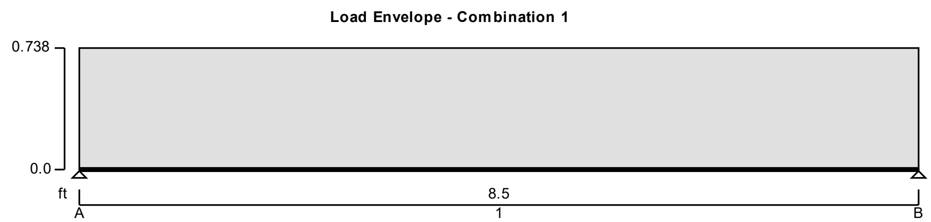
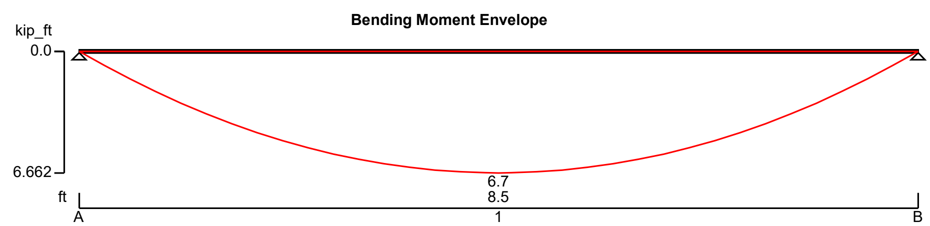
B6

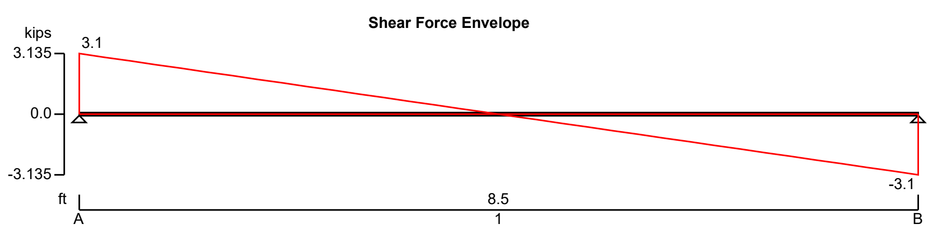
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS)

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

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 272 lb/ft

Snow full UDL 453 lb/ft

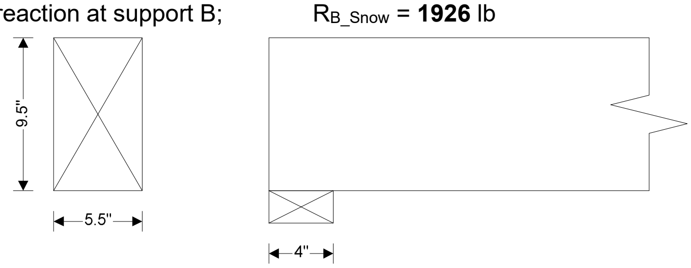
Load combinations

Load combination 1 | Support A | Dead × 1.00

		Snow × 1.00
	Span 1	Dead × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Snow × 1.00

Analysis results

Maximum moment;	M_{max} = **6662** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **6662** lb_ft	
Maximum shear;	F_{max} = **3135** lb;	F_{min} = **-3135** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **3135** lb	
Total load on member;	W_{tot} = **6270** lb	
Reaction at support A;	R_{A_max} = **3135** lb;	R_{A_min} = **3135** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **1209** lb	
Unfactored snow load reaction at support A;	R_{A_Snow} = **1926** lb	
Reaction at support B;	R_{B_max} = **3135** lb;	R_{B_min} = **3135** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **1209** lb	
Unfactored snow load reaction at support B;	R_{B_Snow} = **1926** lb	



Sawn lumber section details

Nominal breadth of sections;	b_{nom} = **6** in
Dressed breadth of sections;	b = **5.5** in
Nominal depth of sections;	d_{nom} = **10** in
Dressed depth of sections;	d = **9.5** in
Number of sections in member;	N = **1**
Overall breadth of member;	b_b = N × b = **5.5** in
Species, grade and size classification;	Douglas Fir-Larch, No.2 grade, Beams and stringers
Bending parallel to grain;	F_b = **875** lb/in^2
Tension parallel to grain;	F_t = **425** lb/in^2
Compression parallel to grain;	F_c = **600** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **625** lb/in^2
Shear parallel to grain;	F_v = **170** lb/in^2
Modulus of elasticity;	E = **1300000** lb/in^2
Modulus of elasticity, stability calculations;	E_{min} = **470000** lb/in^2
Mean shear modulus;	G_{def} = E / 16 = **81250** lb/in^2

Member details

Service condition;	**Dry**
Length of span;	L_{s1} = **8.5** ft

Length of bearing; L_b = **4** in

Load duration; **Two months**

Section properties

Cross sectional area of member; $A = N \times b \times d$ = **52.25** in^2

Section modulus; $S_x = N \times b \times d^2 / 6$ = **82.73** in^3

$S_y = d \times (N \times b)^2 / 6$ = **47.90** in^3

Second moment of area; $I_x = N \times b \times d^3 / 12$ = **392.96** in^4

$I_y = d \times (N \times b)^3 / 12$ = **131.71** in^4

Adjustment factors

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

Temperature factor - Table 2.3.3; C_t = **1.00**

Size factor for bending - Table 4D; C_{Fb} = **1.00**

Size factor for tension - Table 4D; C_{Ft} = **1.00**

Size factor for compression - Table 4D; C_{Fc} = **1.00**

Flat use factor - Table 4D; C_{fu} = **1.00**

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; $d_{nom} / (N \times b_{nom})$ = **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; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b$ = **625** lb/in^2

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b)$ = **143** lb/in^2

f_{c_perp} / F_{c_perp}' = **0.228**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1006** lb/in^2

Actual bending stress; $f_b = M / S_x$ = **966** lb/in^2

f_b / F_b' = **0.960**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_i$ = **196** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A)$ = **90** lb/in^2

f_v / F_v' = **0.460**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection; $E' = E \times C_{ME} \times C_t \times C_{iE} =$ **1300000** lb/in^2

Design deflection; $\delta_{adm} = 0.0042 \times L_{s1} =$ **0.428** in

Total deflection; $\delta_{b\_s1} =$ **0.170** in

$\delta_{b\_s1} / \delta_{adm} =$ **0.396**

PASS - Total deflection is less than design deflection

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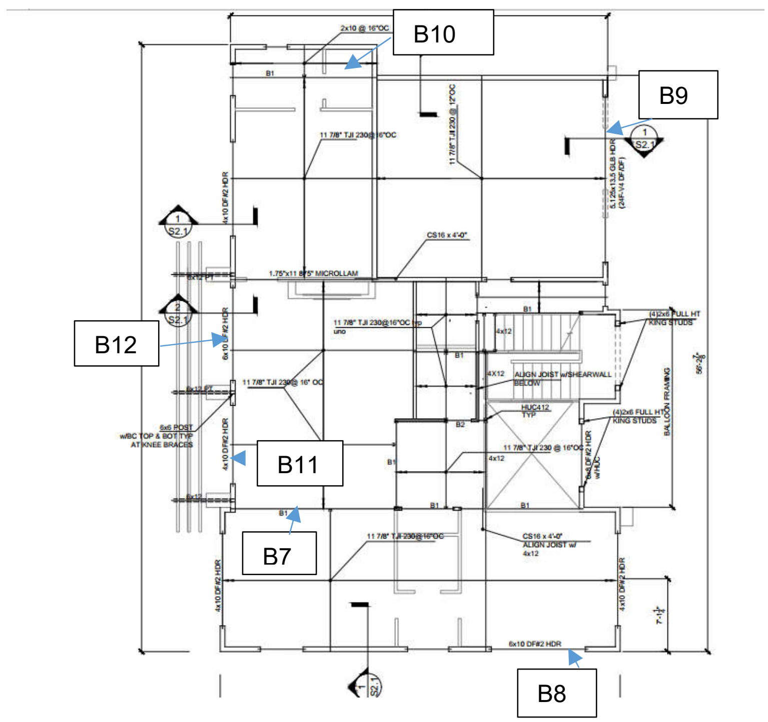
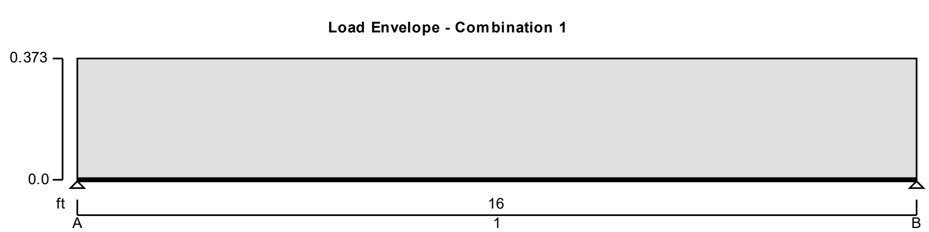


B7

STRUCTURAL COMPOSITE LUMBER BEAM ANALYSIS & DESIGN (NDS)

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

TEDDS calculation version 1.7.03



GUIBIN LU, PE
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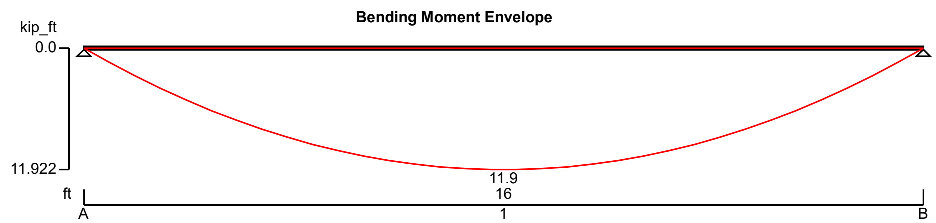
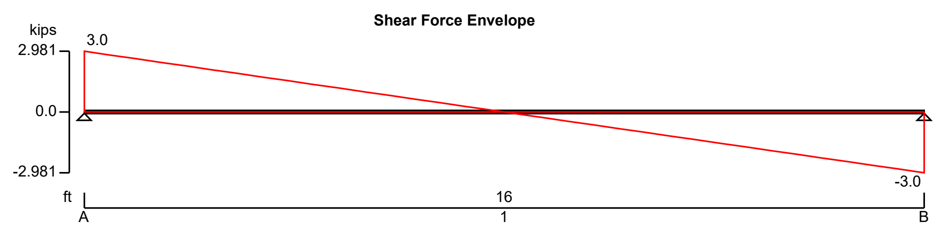
Ph: (360)747-7509

Project				Job Ref.	
Sun Residence Mercer Island				1956	
Section				Sheet no./rev.	
				25	
Calc. by	Date	Chk'd by	Date	App'd by	Date
GL	01-05-2020				





Applied loading

Beam loads

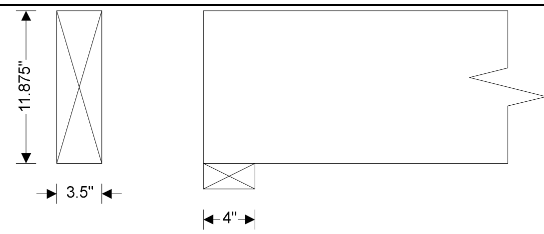
Dead self weight of beam × 1
Dead full UDL 83 lb/ft
Live full UDL 277 lb/ft

Load combinations

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

Analysis results

Maximum moment;	M_{max} = **11922** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **11922** lb_ft	
Maximum shear;	F_{max} = **2981** lb;	F_{min} = **-2981** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **2981** lb	
Total load on member;	W_{tot} = **5961** lb	
Reaction at support A;	R_{A_max} = **2981** lb;	R_{A_min} = **2981** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **768** lb	
Unfactored live load reaction at support A;	R_{A_Live} = **2213** lb	
Reaction at support B;	R_{B_max} = **2981** lb;	R_{B_min} = **2981** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **768** lb	
Unfactored live load reaction at support B;	R_{B_Live} = **2213** lb	



Composite section details

Breadth of composite section;	b = **3.5** in
Depth of composite section;	d = **11.875** in
Number of composite sections in member;	N = **1**
Overall breadth of composite member;	b_b = N × b = **3.5** in
Composite type and grade;	Parallam PSL, 2.0E-2900Fb grade
Bending parallel to grain;	F_b = **2900** lb/in^2
Tension parallel to grain;	F_t = **2025** lb/in^2
Compression parallel to grain;	F_c = **2900** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **625** lb/in^2
Shear parallel to grain;	F_v = **290** lb/in^2
Modulus of elasticity;	E = **2000000** lb/in^2
Modulus of elasticity, stability calculations;	E_{min} = **1017000** lb/in^2
Mean shear modulus;	G_{def} = E / 16 = **125000** lb/in^2
Average density;	ρ = **45** lb/ft^3

Member details

Service condition;	**Dry**
Length of span;	L_{s1} = **16** ft
Length of bearing;	L_b = **4** in
Load duration;	**Two months**

Section properties

Cross sectional area of member;	A = N × b × d = **41.56** in^2
Section modulus;	S_x = N × b × d^2 / 6 = **82.26** in^3
	S_y = d × (N × b)^2 / 6 = **24.24** in^3
Second moment of area;	I_x = N × b × d^3 / 12 = **488.41** in^4
	I_y = d × (N × b)^3 / 12 = **42.43** in^4

Adjustment factors

Load duration factor - Table 2.3.2;	C_D = **1.15**
Temperature factor - Table 2.3.3;	C_t = **1.00**
Size factor for bending;	C_{Fb} = (12 in / max(d, 3.5 in))$^{0.111}$ = **1.00**
Repetitive member factor - cl.8.3.7;	C_r = **1.00**
Length factor;	C_{Len} = **1.00**
Bearing area factor - cl.3.10.4;	C_b = **1.00**
Depth-to-breadth ratio;	d / (N × b) = **3.39**

- 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; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_b$ = **625** lb/in^2

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b)$ = **213** lb/in^2

f_{c_perp} / F_{c_perp}' = **0.341**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_r$ = **3339** lb/in^2

Actual bending stress; $f_b = M / S_x$ = **1739** lb/in^2

f_b / F_b' = **0.521**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t$ = **334** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A)$ = **108** lb/in^2

f_v / F_v' = **0.323**

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** lb/in^2

Design deflection; $\delta_{adm} = 0.0042 \times L_{s1}$ = **0.806** in

Total deflection; δ_{b_s1} = **0.562** in

$\delta_{b\_s1} / \delta_{adm}$ = **0.697**

PASS - Total deflection is less than design deflection

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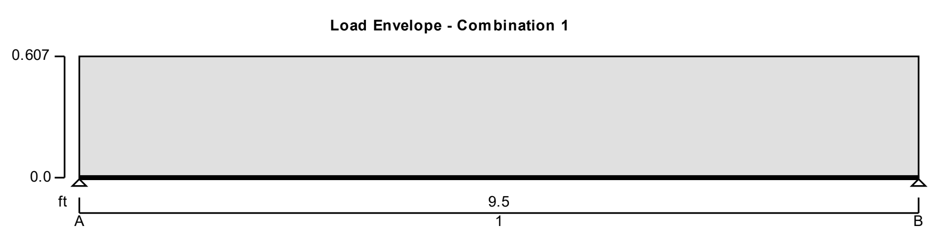
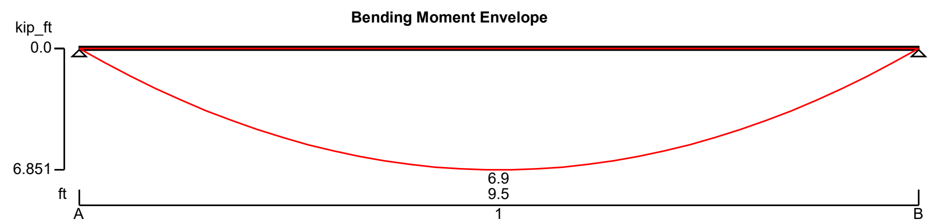
B8

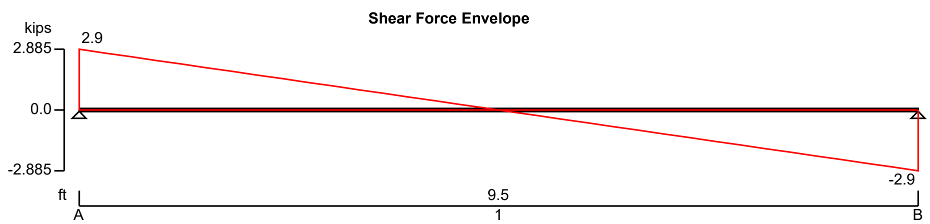
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS)

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

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 81 lb/ft
Live full UDL 270 lb/ft
Dead full UDL 140 lb/ft
Snow full UDL 100 lb/ft

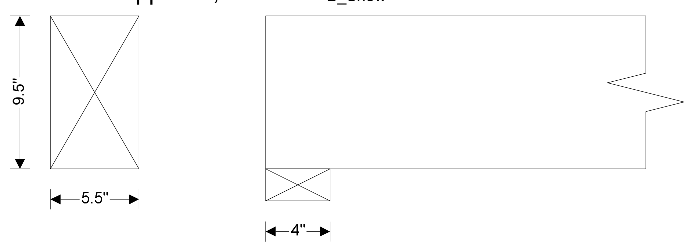
Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00
		Snow × 1.00
	Span 1	Dead × 1.00
		Live × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Live × 1.00
		Snow × 1.00

Analysis results

Maximum moment;	M_{max} = **6851** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **6851** lb_ft	
Maximum shear;	F_{max} = **2885** lb;	F_{min} = **-2885** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **2885** lb	
Total load on member;	W_{tot} = **5770** lb	
Reaction at support A;	R_{A_max} = **2885** lb;	R_{A_min} = **2885** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **1127** lb	
Unfactored live load reaction at support A;	R_{A_Live} = **1283** lb	

Unfactored snow load reaction at support A; R_{A_Snow} = **475** lb
Reaction at support B; R_{B_max} = **2885** lb; R_{B_min} = **2885** lb
Unfactored dead load reaction at support B; R_{B_Dead} = **1127** lb
Unfactored live load reaction at support B; R_{B_Live} = **1283** lb
Unfactored snow load reaction at support B; R_{B_Snow} = **475** lb



Sawn lumber section details
Nominal breadth of sections; b_{nom} = **6** in
Dressed breadth of sections; b = **5.5** in
Nominal depth of sections; d_{nom} = **10** in
Dressed depth of sections; d = **9.5** in
Number of sections in member; N = **1**
Overall breadth of member; $b_b = N \times b$ = **5.5** in
Species, grade and size classification; Douglas Fir-Larch, No.2 grade, Beams and stringers
Bending parallel to grain; F_b = **875** lb/in^2
Tension parallel to grain; F_t = **425** lb/in^2
Compression parallel to grain; F_c = **600** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Shear parallel to grain; F_v = **170** lb/in^2
Modulus of elasticity; E = **1300000** lb/in^2
Modulus of elasticity, stability calculations; E_{min} = **470000** lb/in^2
Mean shear modulus; $G_{def} = E / 16$ = **81250** lb/in^2

Member details
Service condition; **Dry**
Length of span; L_{s1} = **9.5** ft
Length of bearing; L_b = **4** in
Load duration; **Two months**

Section properties
Cross sectional area of member; $A = N \times b \times d$ = **52.25** in^2
Section modulus; $S_x = N \times b \times d^2 / 6$ = **82.73** in^3
$S_y = d \times (N \times b)^2 / 6$ = **47.90** in^3
Second moment of area; $I_x = N \times b \times d^3 / 12$ = **392.96** in^4
$I_y = d \times (N \times b)^3 / 12$ = **131.71** in^4

Adjustment factors
Load duration factor - Table 2.3.2; C_D = **1.15**
Temperature factor - Table 2.3.3; C_t = **1.00**
Size factor for bending - Table 4D; C_{Fb} = **1.00**

Size factor for tension - Table 4D; C_{Ft} = **1.00**

Size factor for compression - Table 4D; C_{Fc} = **1.00**

Flat use factor - Table 4D; C_{fu} = **1.00**

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; $d_{nom} / (N \times b_{nom})$ = **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; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b$ = **625** lb/in^2

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b)$ = **131** lb/in^2

f_{c_perp} / F_{c_perp}' = **0.210**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1006** lb/in^2

Actual bending stress; $f_b = M / S_x$ = **994** lb/in^2

f_b / F_b' = **0.988**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_i$ = **196** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A)$ = **83** lb/in^2

f_v / F_v' = **0.424**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection; $E' = E \times C_{ME} \times C_t \times C_{iE}$ = **1300000** lb/in^2

Design deflection; $\delta_{adm} = 0.0042 \times L_{s1}$ = **0.479** in

Total deflection; δ_{b_s1} = **0.218** in

$\delta_{b\_s1} / \delta_{adm}$ = **0.455**

PASS - Total deflection is less than design deflection

;

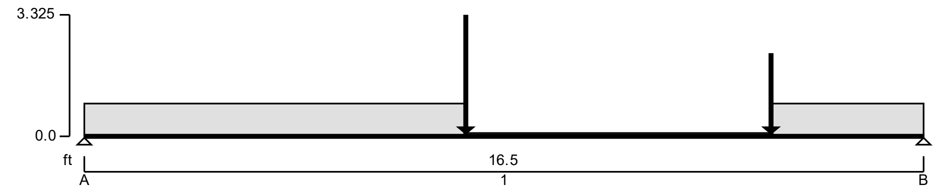
B9

STRUCTURAL GLUED LAMINATED TIMBER (GLULAM) BEAM ANALYSIS & DESIGN (NDS)

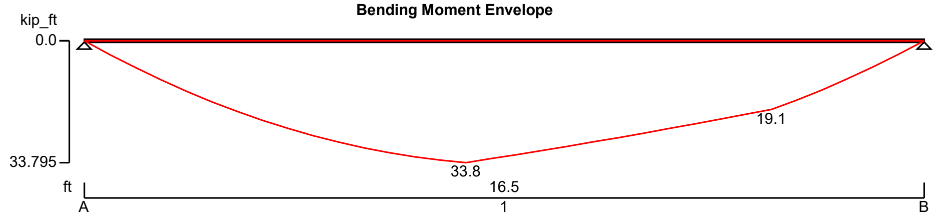
In accordance with the ANSI/AF&PA NDS-2015 using the ASD method

TEDDS calculation version 1.7.03

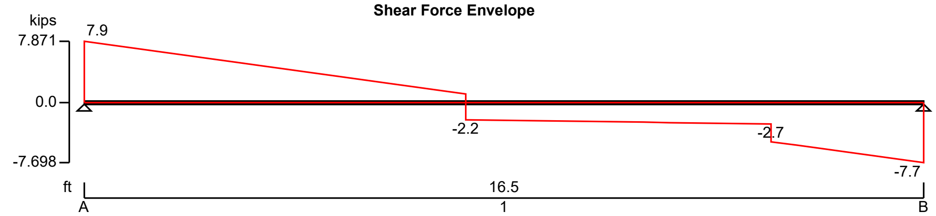
Load Envelope - Combination 1



Bending Moment Envelope



Shear Force Envelope



Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 16 lb/ft
Live full UDL 53 lb/ft
Dead point load 868 lb at 162.00 in
Snow point load 1409 lb at 162.00 in
Dead point load 1261 lb at 90.00 in
Snow point load 2064 lb at 90.00 in
Dead partial UDL 352 lb/ft from 0.00 in to 90.00 in
Snow partial UDL 454 lb/ft from 0.00 in to 90.00 in
Dead partial UDL 352 lb/ft from 162.00 in to 198.00 in
Snow partial UDL 454 lb/ft from 162.00 in to 198.00 in

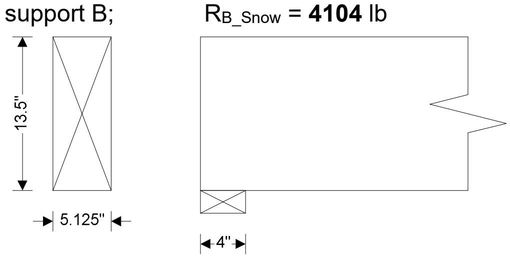
Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00
		Snow × 1.00
	Span 1	Dead × 1.00

		Live × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Live × 1.00
		Snow × 1.00

Analysis results

Maximum moment;	M_{max} = **33795** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **33795** lb_ft	
Maximum shear;	F_{max} = **7871** lb;	F_{min} = **-7698** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **7871** lb	
Total load on member;	W_{tot} = **15569** lb	
Reaction at support A;	R_{A_max} = **7871** lb;	R_{A_min} = **7871** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **3295** lb	
Unfactored live load reaction at support A;	R_{A_Live} = **439** lb	
Unfactored snow load reaction at support A;	R_{A_Snow} = **4138** lb	
Reaction at support B;	R_{B_max} = **7698** lb;	R_{B_min} = **7698** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **3156** lb	
Unfactored live load reaction at support B;	R_{B_Live} = **439** lb	
Unfactored snow load reaction at support B;	R_{B_Snow} = **4104** lb	



Glulam section details

Net finished breadth of sections;	b = **5.125** in
Net finished depth of sections;	d = **13.5** in
Number of sections in member;	N = **1**
Overall breadth of member;	b_b = N × b = **5.125** in
Alignment of laminations;	**Horizontal**
Stress class;	24F-V4 DF/DF
Tension parallel to grain;	F_t = **1100** lb/in^2
Compression parallel to grain;	F_c = **1650** lb/in^2

Bending about X-X axis properties (loaded perpendicular to wide faces of laminations):

Positive bending;	F_{bx_pos} = **2400** lb/in^2
Negative bending;	F_{bx_neg} = **1850** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **650** lb/in^2
Shear parallel to grain;	F_v = **265** lb/in^2
Modulus of elasticity;	E = **1800000** lb/in^2
Modulus of elasticity, stability calculations;	E_{min} = **950000** lb/in^2
Mean shear modulus;	G_{def} = E / 16 = **112500** lb/in^2

Bending about Y-Y axis properties (loaded parallel to wide faces of laminations):

Bending; F_{by} = **1450** lb/in^2

Modulus of elasticity; stability calculations; E_{ymin} = **850000** lb/in^2

Member details

Service condition; **Dry**

Length of span; L_{s1} = **16.5** ft

Length of bearing; L_b = **4** in

Load duration; **Two months**

Section properties

Cross sectional area of member; $A = N \times b \times d$ = **69.19** in^2

Section modulus; $S_x = N \times b \times d^2 / 6$ = **155.67** in^3

$S_y = d \times (N \times b)^2 / 6$ = **59.10** in^3

Second moment of area; $I_x = N \times b \times d^3 / 12$ = **1050.79** in^4

$I_y = d \times (N \times b)^3 / 12$ = **151.44** in^4

Adjustment factors

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

Temperature factor - Table 2.3.3; C_t = **1.00**

Flat use factor - Table 5A; C_{fu} = **1.10**

Bearing area factor - cl.3.10.4; C_b = **1.00**

Length of beam between points of zero moment; L_0 = **16.5** ft

For species other than Southern Pine; x = **10**

Volume factor - eq.5.3-1; $C_V = \min((21\text{ ft} / L_0)^{1/x} \times (12\text{ in} / d)^{1/x} \times (5.125\text{ in} / b)^{1/x}, 1)$ = **1.00**

Depth-to-breadth ratio; $d / (N \times b)$ = **2.63**

- 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; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_b$ = **650** lb/in^2

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b)$ = **384** lb/in^2

f_{c_perp} / F_{c_perp}' = **0.591**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_{bx\_pos} \times C_D \times C_t \times \min(C_L, C_V) \times C_c$ = **2760** lb/in^2

Actual bending stress; $f_b = M_{max} / S_x$ = **2605** lb/in^2

f_b / F_b' = **0.944**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t$ = **305** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A)$ = **171** lb/in^2

f_v / F_v' = **0.560**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection; $E' = E_x \times C_{ME} \times C_t$ = **1800000** lb/in^2

Design deflection; $\delta_{adm} = 0.0042 \times L_{s1} = \mathbf{0.832}$ in

Total deflection; $\delta_{b\_s1} = \mathbf{0.822}$ in

$\delta_{b\_s1} / \delta_{adm} = \mathbf{0.989}$

PASS - Total deflection is less than design deflection

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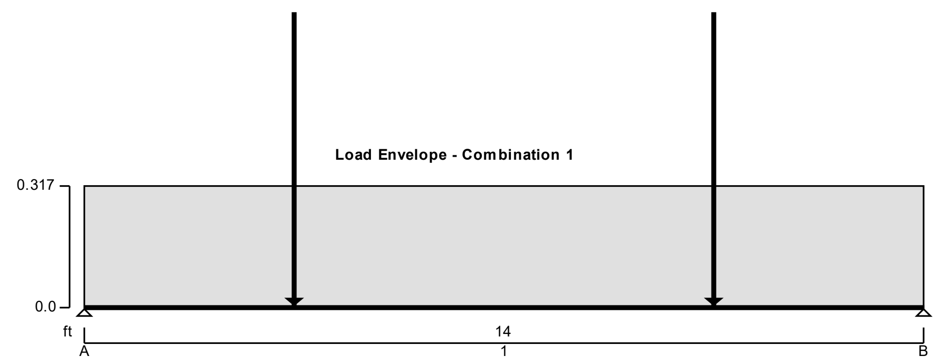
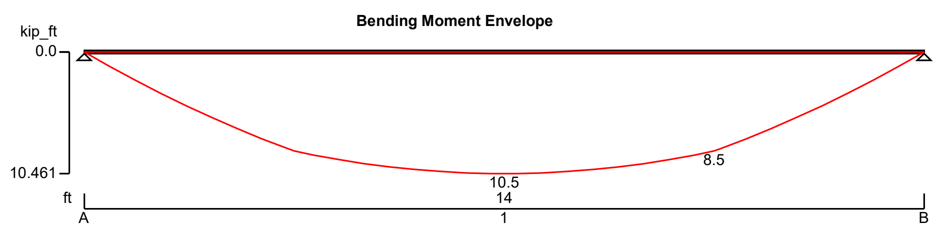
B10

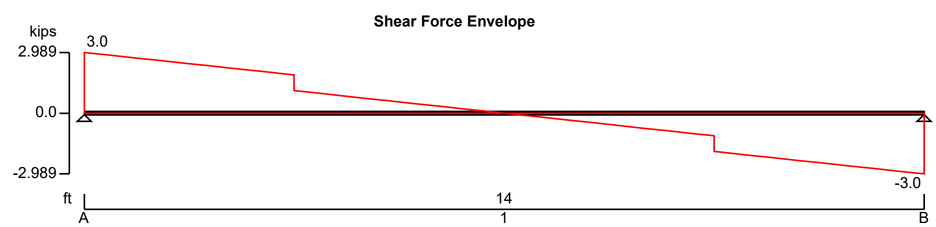
STRUCTURAL COMPOSITE LUMBER BEAM ANALYSIS & DESIGN (NDS)

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

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1

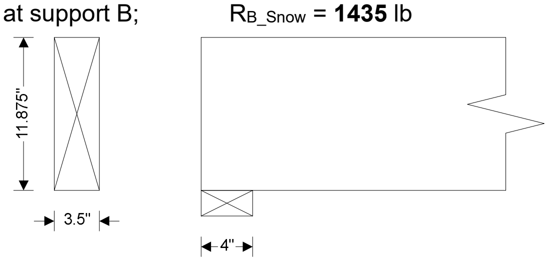
	Dead full UDL 24 lb/ft
	Live full UDL 80 lb/ft
	Dead full UDL 75 lb/ft
	Snow full UDL 125 lb/ft
	Dead point load 210 lb at 42.00 in
	Snow point load 560 lb at 42.00 in
	Dead point load 210 lb at 126.00 in
	Snow point load 560 lb at 126.00 in

Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00
		Snow × 1.00
	Span 1	Dead × 1.00
		Live × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Live × 1.00
		Snow × 1.00

Analysis results

Maximum moment;	M_{max} = **10461** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **10461** lb_ft	
Maximum shear;	F_{max} = **2989** lb;	F_{min} = **-2989** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **2989** lb	
Total load on member;	W_{tot} = **5978** lb	
Reaction at support A;	R_{A_max} = **2989** lb;	R_{A_min} = **2989** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **994** lb	
Unfactored live load reaction at support A;	R_{A_Live} = **560** lb	
Unfactored snow load reaction at support A;	R_{A_Snow} = **1435** lb	
Reaction at support B;	R_{B_max} = **2989** lb;	R_{B_min} = **2989** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **994** lb	
Unfactored live load reaction at support B;	R_{B_Live} = **560** lb	
Unfactored snow load reaction at support B;	R_{B_Snow} = **1435** lb	



Composite section details

Breadth of composite section;	b = **3.5** in
Depth of composite section;	d = **11.875** in
Number of composite sections in member;	N = **1**

Overall breadth of composite member;	$b_b = N \times b =$ **3.5** in
Composite type and grade;	Parallam PSL, 2.0E-2900Fb grade
Bending parallel to grain;	$F_b =$ **2900** lb/in^2
Tension parallel to grain;	$F_t =$ **2025** lb/in^2
Compression parallel to grain;	$F_c =$ **2900** lb/in^2
Compression perpendicular to grain;	$F_{c\_perp} =$ **625** lb/in^2
Shear parallel to grain;	$F_v =$ **290** lb/in^2
Modulus of elasticity;	$E =$ **2000000** lb/in^2
Modulus of elasticity, stability calculations;	$E_{min} =$ **1017000** lb/in^2
Mean shear modulus;	$G_{def} = E / 16 =$ **125000** lb/in^2
Average density;	$\rho =$ **45** lb/ft^3

Member details

Service condition;	**Dry**
Length of span;	$L_{s1} =$ **14** ft
Length of bearing;	$L_b =$ **4** in
Load duration;	**Two months**

Section properties

Cross sectional area of member;	$A = N \times b \times d =$ **41.56** in^2
Section modulus;	$S_x = N \times b \times d^2 / 6 =$ **82.26** in^3
	$S_y = d \times (N \times b)^2 / 6 =$ **24.24** in^3
Second moment of area;	$I_x = N \times b \times d^3 / 12 =$ **488.41** in^4
	$I_y = d \times (N \times b)^3 / 12 =$ **42.43** in^4

Adjustment factors

Load duration factor - Table 2.3.2;	$C_D =$ **1.15**
Temperature factor - Table 2.3.3;	$C_t =$ **1.00**
Size factor for bending;	$C_{Fb} = (12 \text{ in} / \max(d, 3.5 \text{ in}))^{0.111} =$ **1.00**
Repetitive member factor - cl.8.3.7;	$C_r =$ **1.00**
Length factor;	$C_{Len} =$ **1.00**
Bearing area factor - cl.3.10.4;	$C_b =$ **1.00**
Depth-to-breadth ratio;	$d / (N \times b) =$ **3.39**
- 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;	$F_{c\_perp}' = F_{c\_perp} \times C_t \times C_b =$ **625** lb/in^2
Applied compression stress perpendicular to grain;	$f_{c\_perp} = R_{A\_max} / (N \times b \times L_b) =$ **213** lb/in^2
	$f_{c\_perp} / F_{c\_perp}' =$ **0.342**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress;	$F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_r =$ **3339** lb/in^2
Actual bending stress;	$f_b = M / S_x =$ **1526** lb/in^2
	$f_b / F_b' =$ **0.457**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t =$ **334** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A) =$ **108** lb/in^2

$f_v / F_v' =$ **0.323**

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** lb/in^2

Design deflection; $\delta_{adm} = 0.0042 \times L_{s1} =$ **0.706** in

Total deflection; $\delta_{b\_s1} =$ **0.388** in

$\delta_{b\_s1} / \delta_{adm} =$ **0.549**

PASS - Total deflection is less than design deflection

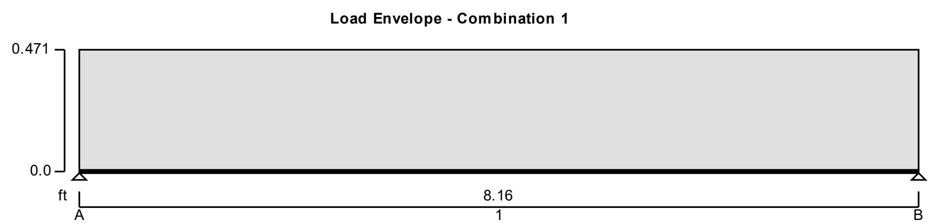
;

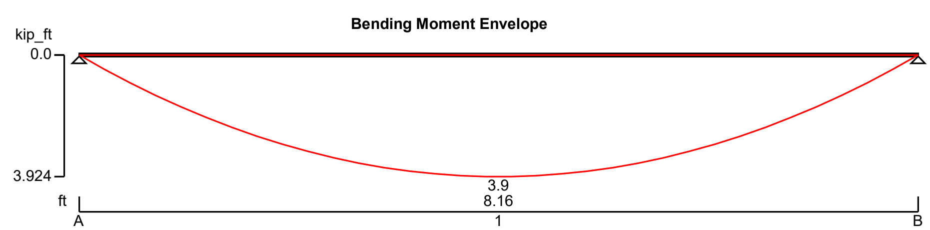
B11

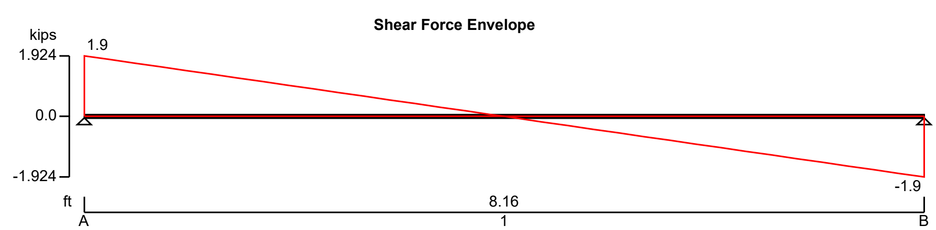
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS)

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

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 107 lb/ft

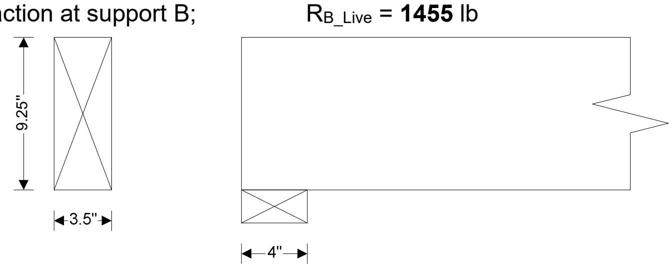
Live full UDL 357 lb/ft

Load combinations

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

Analysis results

Maximum moment;	M_{max} = **3924** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **3924** lb_ft	
Maximum shear;	F_{max} = **1924** lb;	F_{min} = **-1924** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **1924** lb	
Total load on member;	W_{tot} = **3847** lb	
Reaction at support A;	R_{A_max} = **1924** lb;	R_{A_min} = **1924** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **469** lb	
Unfactored live load reaction at support A;	R_{A_Live} = **1455** lb	
Reaction at support B;	R_{B_max} = **1924** lb;	R_{B_min} = **1924** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **469** lb	
Unfactored live load reaction at support B;	R_{B_Live} = **1455** lb	



Sawn lumber section details

Nominal breadth of sections;	b_{nom} = **4** in
Dressed breadth of sections;	b = **3.5** in
Nominal depth of sections;	d_{nom} = **10** in
Dressed depth of sections;	d = **9.25** in
Number of sections in member;	N = **1**
Overall breadth of member;	b_b = N × b = **3.5** in
Species, grade and size classification;	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain;	F_b = **900** lb/in^2
Tension parallel to grain;	F_t = **575** lb/in^2
Compression parallel to grain;	F_c = **1350** lb/in^2

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Compression perpendicular to grain;	F_{c_perp} = **625** lb/in^2
Shear parallel to grain;	F_v = **180** lb/in^2
Modulus of elasticity;	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations;	E_{min} = **580000** lb/in^2
Mean shear modulus;	$G_{def} = E / 16$ = **100000** lb/in^2

Member details

Service condition;	**Dry**
Length of span;	L_{s1} = **8.16** ft
Length of bearing;	L_b = **4** in
Load duration;	**Two months**

Section properties

Cross sectional area of member;	$A = N \times b \times d$ = **32.38** in^2
Section modulus;	$S_x = N \times b \times d^2 / 6$ = **49.91** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **18.89** in^3
Second moment of area;	$I_x = N \times b \times d^3 / 12$ = **230.84** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **33.05** in^4

Adjustment factors

Load duration factor - Table 2.3.2;	C_D = **1.15**
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} = **1.10**
Size factor for compression - Table 4A;	C_{Fc} = **1.00**
Flat use factor - Table 4A;	C_{fu} = **1.10**
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;	$d_{nom} / (N \times b_{nom})$ = **2.50**
- 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;	$F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b$ = **625** lb/in^2
Applied compression stress perpendicular to grain;	$f_{c\_perp} = R_{A\_max} / (N \times b \times L_b)$ = **137** lb/in^2
	f_{c_perp} / F_{c_perp}' = **0.220**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress;	$F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1242** lb/in^2
Actual bending stress;	$f_b = M / S_x$ = **943** lb/in^2
	f_b / F_b' = **0.760**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_i =$ **207** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A) =$ **89** lb/in^2

$f_v / F_v' =$ **0.431**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection; $E' = E \times C_{ME} \times C_t \times C_{iE} =$ **1600000** lb/in^2

Design deflection; $\delta_{adm} = 0.0042 \times L_{s1} =$ **0.411** in

Total deflection; $\delta_{b\_s1} =$ **0.127** in

$\delta_{b\_s1} / \delta_{adm} =$ **0.310**

PASS - Total deflection is less than design deflection

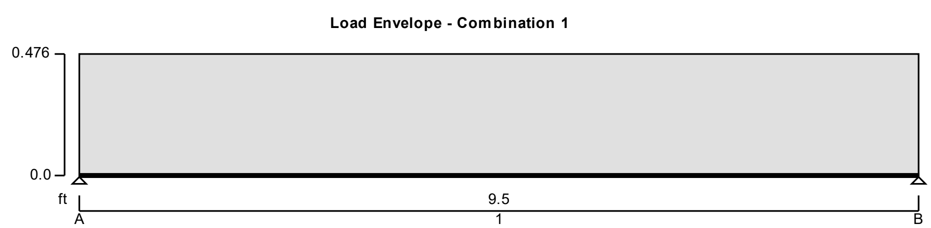
;

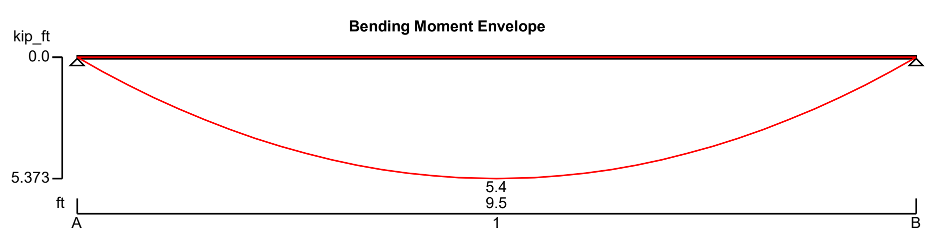
B12

STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS)

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

TEDDS calculation version 1.7.03





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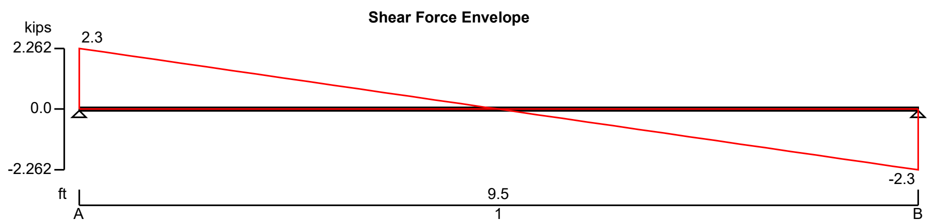
	Section				Sheet no./rev. 41	
	Calc. by GL	Date 01-05-2020	Chk'd by	Date	App'd by	Date



Applied loading

Beam loads

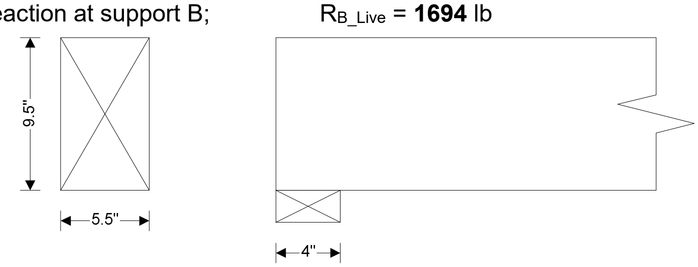
Dead self weight of beam × 1
Dead full UDL 107 lb/ft
Live full UDL 357 lb/ft

Load combinations

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

Analysis results

Maximum moment;	M_{max} = **5373** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **5373** lb_ft	
Maximum shear;	F_{max} = **2262** lb;	F_{min} = **-2262** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **2262** lb	
Total load on member;	W_{tot} = **4525** lb	
Reaction at support A;	R_{A_max} = **2262** lb;	R_{A_min} = **2262** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **568** lb	
Unfactored live load reaction at support A;	R_{A_Live} = **1694** lb	
Reaction at support B;	R_{B_max} = **2262** lb;	R_{B_min} = **2262** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **568** lb	
Unfactored live load reaction at support B;	R_{B_Live} = **1694** lb	



Sawn lumber section details

Nominal breadth of sections;	b_{nom} = **6** in
Dressed breadth of sections;	b = **5.5** in

Nominal depth of sections;	d_{nom} = **10** in
Dressed depth of sections;	d = **9.5** in
Number of sections in member;	N = **1**
Overall breadth of member;	b_b = N × b = **5.5** in
Species, grade and size classification;	Douglas Fir-Larch, No.2 grade, Beams and stringers
Bending parallel to grain;	F_b = **875** lb/in^2
Tension parallel to grain;	F_t = **425** lb/in^2
Compression parallel to grain;	F_c = **600** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **625** lb/in^2
Shear parallel to grain;	F_v = **170** lb/in^2
Modulus of elasticity;	E = **1300000** lb/in^2
Modulus of elasticity, stability calculations;	E_{min} = **470000** lb/in^2
Mean shear modulus;	G_{def} = E / 16 = **81250** lb/in^2

Member details

Service condition;	**Dry**
Length of span;	L_{s1} = **9.5** ft
Length of bearing;	L_b = **4** in
Load duration;	**Two months**

Section properties

Cross sectional area of member;	A = N × b × d = **52.25** in^2
Section modulus;	S_x = N × b × d^2 / 6 = **82.73** in^3
	S_y = d × (N × b)^2 / 6 = **47.90** in^3
Second moment of area;	I_x = N × b × d^3 / 12 = **392.96** in^4
	I_y = d × (N × b)^3 / 12 = **131.71** in^4

Adjustment factors

Load duration factor - Table 2.3.2;	C_D = **1.15**
Temperature factor - Table 2.3.3;	C_t = **1.00**
Size factor for bending - Table 4D;	C_{Fb} = **1.00**
Size factor for tension - Table 4D;	C_{Ft} = **1.00**
Size factor for compression - Table 4D;	C_{Fc} = **1.00**
Flat use factor - Table 4D;	C_{fu} = **1.00**
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 × 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; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b =$ **625** lb/in^2

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{B\_max} / (N \times b \times L_b) =$ **103** lb/in^2

$f_{c\_perp} / F_{c\_perp}' =$ **0.165**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r =$ **1006** lb/in^2

Actual bending stress; $f_b = M / S_x =$ **779** lb/in^2

$f_b / F_b' =$ **0.775**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_i =$ **196** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A) =$ **65** lb/in^2

$f_v / F_v' =$ **0.332**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection; $E' = E \times C_{ME} \times C_t \times C_{iE} =$ **1300000** lb/in^2

Design deflection; $\delta_{adm} = 0.0042 \times L_{s1} =$ **0.479** in

Total deflection; $\delta_{b\_s1} =$ **0.171** in

$\delta_{b\_s1} / \delta_{adm} =$ **0.357**

PASS - Total deflection is less than design deflection

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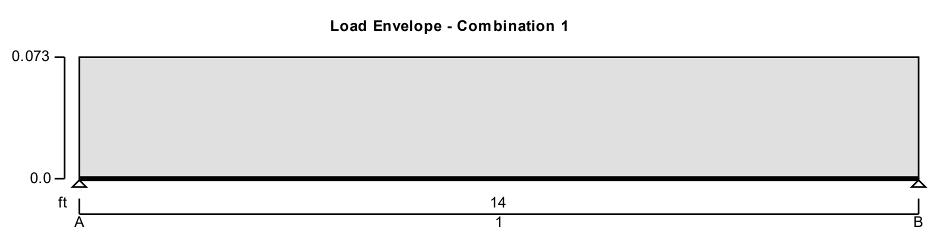
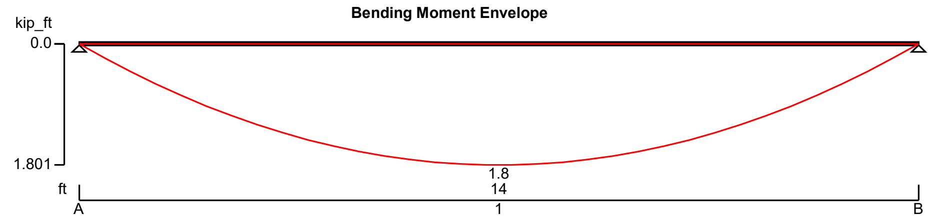
B13- TYPICAL 2X FLOOR JOIST

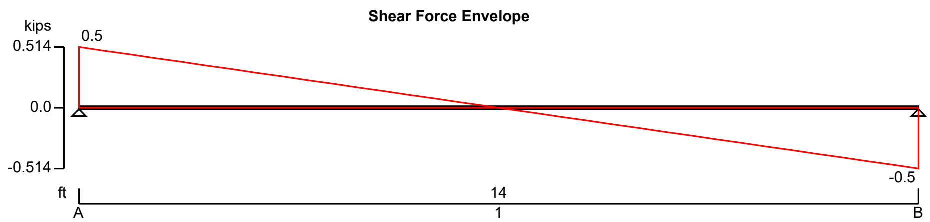
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS)

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

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 16 lb/ft

Live full UDL 53 lb/ft

Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00
		Snow × 1.00
	Span 1	Dead × 1.00
		Live × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Live × 1.00
		Snow × 1.00

Analysis results

Maximum moment;	M_{max} = **1801** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **1801** lb_ft	
Maximum shear;	F_{max} = **514** lb;	F_{min} = **-514** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **514** lb	
Total load on member;	W_{tot} = **1029** lb	
Reaction at support A;	R_{A_max} = **514** lb;	R_{A_min} = **514** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **142** lb	
Unfactored live load reaction at support A;	R_{A_Live} = **372** lb	
Reaction at support B;	R_{B_max} = **514** lb;	R_{B_min} = **514** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **142** lb	

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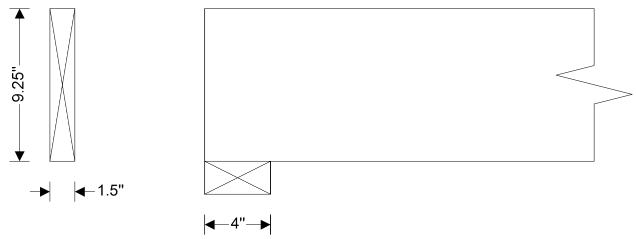
	Section			Sheet no./rev. 45	
	Calc. by GL	Date 01-05-2020	Chk'd by	Date	App'd by	Date

Unfactored live load reaction at support B; R_{B_Live} = **372** 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 = **1**

Overall breadth of member; $b_b = N \times b$ = **1.5** in

Species, grade and size classification; Hem-Fir, No.2 grade, 2" & wider

Bending parallel to grain; F_b = **850** lb/in^2

Tension parallel to grain; F_t = **525** lb/in^2

Compression parallel to grain; F_c = **1300** lb/in^2

Compression perpendicular to grain; F_{c_perp} = **405** lb/in^2

Shear parallel to grain; F_v = **150** lb/in^2

Modulus of elasticity; E = **1300000** lb/in^2

Modulus of elasticity, stability calculations; E_{min} = **470000** lb/in^2

Mean shear modulus; $G_{def} = E / 16$ = **81250** lb/in^2

Member details

Service condition; **Dry**

Length of span; L_{s1} = **14** ft

Length of bearing; L_b = **4** in

Load duration; **Two months**

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member; $A = N \times b \times d$ = **13.87** in^2

Section modulus; $S_x = N \times b \times d^2 / 6$ = **21.39** in^3

$S_y = d \times (N \times b)^2 / 6$ = **3.47** in^3

Second moment of area; $I_x = N \times b \times d^3 / 12$ = **98.93** in^4

$I_y = d \times (N \times b)^3 / 12$ = **2.60** in^4

Adjustment factors

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

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; $d_{nom} / (N \times b_{nom})$ = **5.00**

- 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; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b$ = **405** lb/in^2

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b)$ = **86** lb/in^2

f_{c_perp} / F_{c_perp}' = **0.212**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1237** lb/in^2

Actual bending stress; $f_b = M / S_x$ = **1010** lb/in^2

f_b / F_b' = **0.817**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_i$ = **173** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A)$ = **56** lb/in^2

f_v / F_v' = **0.322**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection; $E' = E \times C_{ME} \times C_t \times C_{iE}$ = **1300000** lb/in^2

Design deflection; $\delta_{adm} = 0.0042 \times L_{s1}$ = **0.706** in

Total deflection; δ_{b_s1} = **0.494** in

$\delta_{b\_s1} / \delta_{adm}$ = **0.700**

PASS - Total deflection is less than design deflection

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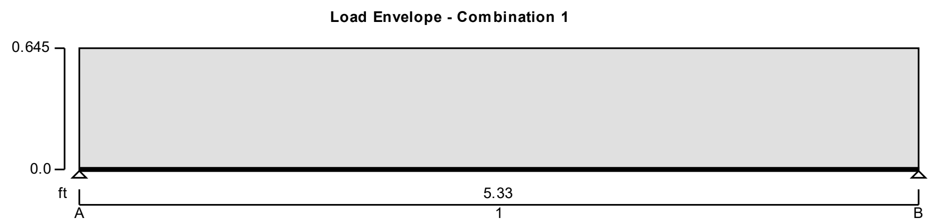
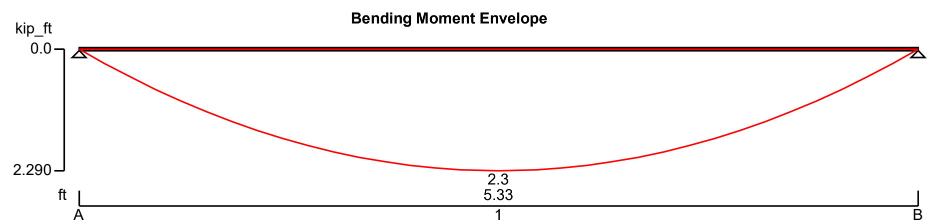
B14 – TYPICAL FIRST FLOOR WOOD BEAM

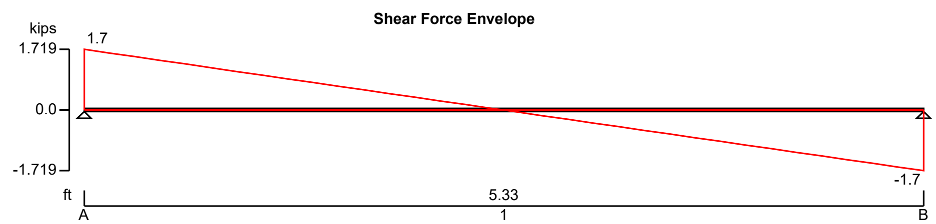
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS)

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

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 147 lb/ft

Live full UDL 490 lb/ft

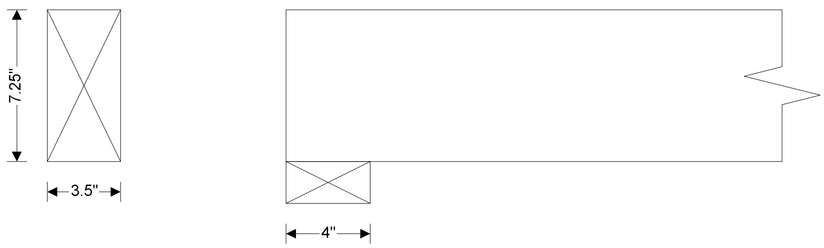
Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00
		Snow × 1.00
	Span 1	Dead × 1.00
		Live × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Live × 1.00
		Snow × 1.00

Analysis results

Maximum moment;	M_{max} = **2290** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **2290** lb_ft	

Maximum shear;	F_{max} = **1719** lb;	F_{min} = **-1719** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **1719** lb	
Total load on member;	W_{tot} = **3437** lb	
Reaction at support A;	R_{A_max} = **1719** lb;	R_{A_min} = **1719** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **413** lb	
Unfactored live load reaction at support A;	R_{A_Live} = **1306** lb	
Reaction at support B;	R_{B_max} = **1719** lb;	R_{B_min} = **1719** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **413** lb	
Unfactored live load reaction at support B;	R_{B_Live} = **1306** lb	



Sawn lumber section details

Nominal breadth of sections;	b_{nom} = **4** in
Dressed breadth of sections;	b = **3.5** in
Nominal depth of sections;	d_{nom} = **8** in
Dressed depth of sections;	d = **7.25** in
Number of sections in member;	N = **1**
Overall breadth of member;	$b_b = N \times b$ = **3.5** in
Species, grade and size classification;	Hem-Fir, No.2 grade, 2" & wider
Bending parallel to grain;	F_b = **850** lb/in^2
Tension parallel to grain;	F_t = **525** lb/in^2
Compression parallel to grain;	F_c = **1300** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **405** lb/in^2
Shear parallel to grain;	F_v = **150** lb/in^2
Modulus of elasticity;	E = **1300000** lb/in^2
Modulus of elasticity, stability calculations;	E_{min} = **470000** lb/in^2
Mean shear modulus;	G_{def} = E / 16 = **81250** lb/in^2

Member details

Service condition;	**Dry**
Length of span;	L_{s1} = **5.33** ft
Length of bearing;	L_b = **4** in
Load duration;	**Two months**

Section properties

Cross sectional area of member;	$A = N \times b \times d$ = **25.37** in^2
Section modulus;	$S_x = N \times b \times d^2 / 6$ = **30.66** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **14.80** in^3
Second moment of area;	$I_x = N \times b \times d^3 / 12$ = **111.15** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **25.90** in^4

Adjustment factors

Load duration factor - Table 2.3.2;	C_D = **1.15**
Temperature factor - Table 2.3.3;	C_t = **1.00**
Size factor for bending - Table 4A;	C_{Fb} = **1.30**
Size factor for tension - Table 4A;	C_{Ft} = **1.20**
Size factor for compression - Table 4A;	C_{Fc} = **1.05**
Flat use factor - Table 4A;	C_{fu} = **1.05**
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;	$F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b$ = **405** lb/in^2
Applied compression stress perpendicular to grain;	$f_{c\_perp} = R_{A\_max} / (N \times b \times L_b)$ = **123** lb/in^2
	f_{c_perp} / F_{c_perp}' = **0.303**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress;	$F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1271** lb/in^2
Actual bending stress;	$f_b = M / S_x$ = **896** lb/in^2
	f_b / F_b' = **0.705**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress;	$F_v' = F_v \times C_D \times C_t \times C_i$ = **173** lb/in^2
Actual shear stress - eq.3.4-2;	$f_v = 3 \times F / (2 \times A)$ = **102** lb/in^2
	f_v / F_v' = **0.589**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection;	$E' = E \times C_{ME} \times C_t \times C_{iE}$ = **1300000** lb/in^2
Design deflection;	$\delta_{adm} = 0.0042 \times L_{s1}$ = **0.269** in
Total deflection;	δ_{b_s1} = **0.081** in
	$\delta_{b\_s1} / \delta_{adm}$ = **0.302**

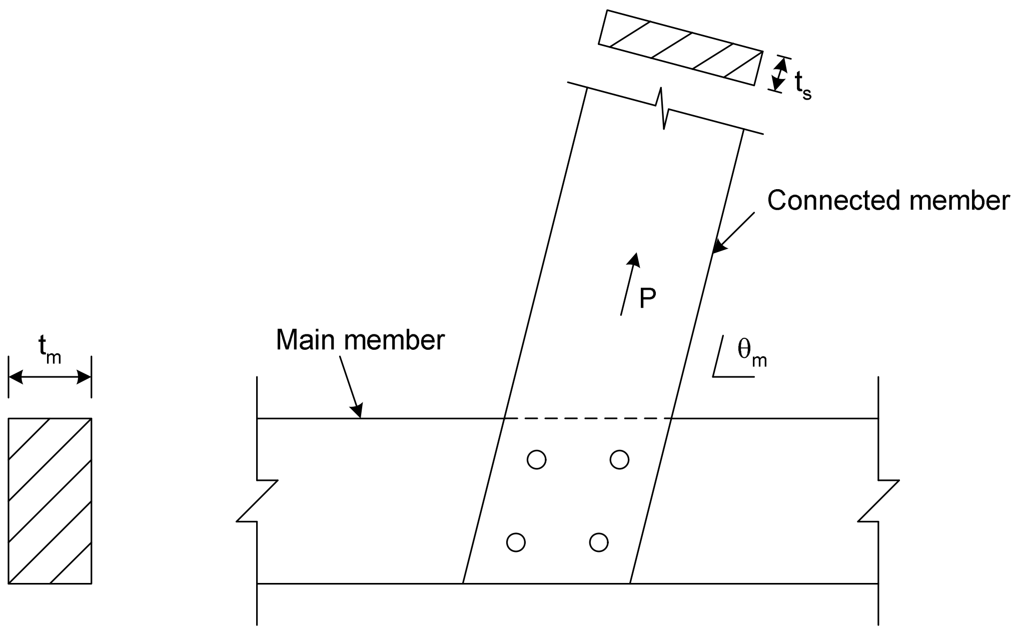
PASS - Total deflection is less than design deflection

;

WOOD CONNECTION (NDS)

BOLTED TIMBER TO TIMBER CONNECTION DESIGN (NDS 2015)

Tedds calculation version 1.2.00 -



Main timber member details

Species of main member;	**Douglas Fir-Larch**
Size of main member (Table 1B);	**6 x 10**; ;
Number of main member;	N_m = **1**
Thickness of main member;	t_m = **5.500** in
Angle of load to grain of main member;	θ_m = **55**°

Connected timber member details

Species of connected member;	**Hem-Fir**
Size of connected member (Table 1B);	**2 x 8**
Number of connected member;	N_s = **2**
Thickness of connected member;	t_s = **1.500** in
Number of interfaces;	$N_{int} = (N_m + N_s) - 1$ = **2**

Bolt details

Bolt diameter (Table L1);	**3/4"**
Number of rows of bolts;	R = **2**
Number of columns of bolts;	C = **1**
Total number of bolts;	$N_{total} = R \times C$ = **2**

Applied load

Applied load to the connection;	P = **1600** lb

Dowel bearing length (main) (12.3.5)

Dowel bearing length in main member;	$l_m = t_m$ = **5.500** in

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Dowel bearing length (connected) (12.3.5)

Dowel bearing length in connected member; $l_s = t_s =$ **1.500** in

Bending yield strength (bolt) (Table 12A to 12I footnote no. 2)

Bending yield strength of bolt; $F_{yb} =$ **45000** psi

Dowel bearing strength (main member) (Table 12.3.3 footnote no. 2)

Dowel bearing strength parallel to grain; $F_{e\_par} = 11200 \times G_m \times 1$ psi = **5600** psi

Dowel bearing strength perpendicular to grain; $F_{e\_perp} = 6100 \times G_m^{1.45} \times 1$ psi / $\sqrt{(D / 1\text{ in})}$ = **2578** psi

Dowel bearing strength for small dia. fasteners; $F_e = 16600 \times G_m^{1.84} \times 1$ psi = **4637** psi

Dowel bearing strength at an angle of load to grain; $F_{e\theta m} = (F_{e\_par} \times F_{e\_perp}) / ((F_{e\_par} \times (\sin(\theta_m))^2) + (F_{e\_perp} \times (\cos(\theta_m))^2))$

$F_{e\theta m} =$ **3135** psi

Dowel bearing strength of main member; $F_{em} =$ **3135** psi

Dowel bearing strength (connected timber member) (Table 12.3.3 footnote no. 2)

Dowel bearing strength parallel to grain; $F_{e\_par} = 11200 \times G_s \times 1$ psi= **4816** psi

Dowel bearing strength perpendicular to grain; $F_{e\_perp} = 6100 \times G_s^{1.45} \times 1$ psi / $\sqrt{(D / 1\text{ in})}$ = **2072** psi

Dowel bearing strength for small dia. fasteners; $F_e = 16600 \times G_s^{1.84} \times 1$ psi = **3513** psi

Dowel bearing strength at an angle of load to grain; $F_{e\theta s} = (F_{e\_par} \times F_{e\_perp}) / ((F_{e\_par} \times (\sin(\theta_s))^2) + (F_{e\_perp} \times (\cos(\theta_s))^2))$

$F_{e\theta s} =$ **4816** psi

Dowel bearing strength of connected member; $F_{es} =$ **4816** psi

Preliminary yield limit equation coefficients (Table 12.3.1A notes)

Dowel bearing strength ratio; $R_e = F_{em} / F_{es} =$ **0.651**

Dowel bearing length ratio; $R_t = l_m / l_s =$ **3.667**

Preliminary yield limit equation coefficient k_1; $k_1 = ((\sqrt{(R_e+(2\times R_e^2\times(1+R_t+R_t^2))+(R_t^2\times R_e^3))})-(R_e\times(1+R_t)))/(1+R_e)$

$k_1 =$ **0.849**

Preliminary yield limit equation coefficient k_2; $k_2 = -1+\sqrt{((2\times(1+R_e))+((2\times F_{yb}\times(1+(2\times R_e))\times D^2))/(3\times F_{em}\times l_m^2))}$

$k_2 =$ **0.926**

Preliminary yield limit equation coefficient k_3; $k_3 = -1+\sqrt{(((2\times(1+R_e))/R_e)+((2\times F_{yb}\times(2+R_e)\times D^2))/(3\times F_{em}\times l_s^2))}$

$k_3 =$ **2.379**

Angle of load to grain coefficient k_θ; $k_\theta = 1 + (0.25 \times \max(\theta_m, \theta_s) / 90) =$ **1.153**

Yield limit equations (double shear)

Mode I_m (eq. 12.3-7); $Z_{Im} = (D \times l_m \times F_{em}) / (4 \times k_\theta) =$ **2804** lb

Mode I_s (eq. 12.3-8); $Z_{Is} = (2 \times D \times l_s \times F_{es}) / (4 \times k_\theta) =$ **2350** lb

Mode III_s (eq. 12.3-9); $Z_{IIIs} = (2 \times k_3 \times D \times l_s \times F_{em}) / ((2 + R_e) \times 3.2 \times k_\theta) =$ **1716** lb

Mode IV (eq. 12.3-10); $Z_{IV} = (2 \times D^2) \times (\sqrt{((2 \times F_{em} \times F_{yb}) / (3 \times (1 + R_e)))}) / (3.2 \times k_\theta) =$ **2302** lb

$Z = \min(Z_{Im}, Z_{Is}, Z_{IIIs}, Z_{IV}) =$ **1716** lb

Nominal capacity of single fastener; Z = **1716** lb

Slenderness (Table 12.5.1C footnote no.1)

Slenderness; $l / D =$ **4.000**

Spacing requirements (perpendicular to grain loading)

Edge distance (Table 12.5.1C)

Loaded edge; $e_q = 4 \times D =$ **3.000** in

Unloaded edge; $e_p = 1.5 \times D =$ **1.125** in

End distance (Table 12.5.1A)

End distance (full stregnth); $a_{q\_full} = 4 \times D =$ **3.000** in

End distance (minimum); $a_{q\_min} = 2 \times D =$ **1.500** in

End distance (actual); $a_q =$ **1.500** in

Center to center spacing (Table 12.5.1B)

Center to center spacing (full strength); $s_{\_full} = 4 \times D =$ **3.000** in

Center to center spacing (minimum); $s_{\_min} = 3 \times D =$ **2.250** in

Center to center spacing (actual); $s =$ **3.000** in

Row spacing (Table 12.5.1D)

Row spacing; $s_{row} = ((5 \times l) + (10 \times D)) / 8 = 2.812$ in

Geometry factor C_Δ (12.5.1)

End distance (actual); $a_q =$ **1.500** in

End distance (full strength); $a_{q\_full} =$ **3.000** in

Geometry factor for end distance; $C_{\Delta 1} = a_q / a_{q\_full} =$ **0.50**

Center to center spacing (actual); $s =$ **3.000** in

Center to center spacing (full strength); $s_{\_full} =$ **3.000** in

Geometry factor for spacing; $C_{\Delta 2} = s / s_{\_full} =$ **1.00**

Geometry factor; $C_\Delta = \min(1, C_{\Delta 1}, C_{\Delta 2}) =$ **0.50**

Adjustment factor

Load duration factor (Table 2.3.2); $C_D =$ **1.15**

Wet service factor (Table 11.3.3); $C_M =$ **1.0**

Temperature factor (Table 11.3.4); $C_t =$ **1.0**

Group action factor (eq. 11.3-1); $C_g =$ **1.0**

Geometry factor (12.5.1); $C_\Delta =$ **0.50**

End grain factor (12.5.2); $C_{eg} =$ **1.0**

Diaphragm factor (12.5.3); $C_{di} =$ **1.0**

Toe nail factor (12.5.4); $C_{tn} =$ **1.0**

Total capacity of connection

Capacity of connection; $Z' = Z \times N_{total} \times C_D \times C_M \times C_\Delta =$ **1973** lb

Design result

PASS - Connection capacity exceeds applied load

;

SEISMIC FORCES (ASCE7)

SEISMIC FORCES (ASCE 7-10)

Tedds calculation version 3.0.10

Site parameters

Site class; D

Mapped acceleration parameters

at short periods; S_S = **1.376**; at 1 sec period; S_1 = **0.53**

Site coefficientat short periods; F_a = **1.0**; at 1 sec period; F_v = **1.5**

Spectral response acceleration parameters

at short period ; S_{MS} = **1.376**; at 1 sec period ; S_{M1} = **0.795**

Design spectral acceleration parameters

at short period; S_{DS} = **0.917**; at 1 sec period; S_{D1} = **0.530**

Seismic design category

Risk category; II; ;

Seismic design category; D

Approximate fundamental period

Height above base to highest level of building; h_n = **21.00** ft

Building period parameter C_t; C_t = **0.02**; Building period parameter x; x = **0.75**

Building fundamental period; T = T_a = **0.196** sec; Long-period transition period; T_L = **6** sec

Seismic response coefficient

Seismic force resisting system:A. Bearing_Wall_Systems

15. Light-frame (wood) walls sheathed with wood structural panels

Response modification factor; R = **6.5**;

Seismic importance factor; I_e = **1.000**; Seismic response coefficient; C_s = **0.141**

Seismic base shear

Effective seismic weight of the structure; W = **85.6** kips

Seismic response coefficient; C_s = **0.141**; Seismic base shear; V = **12.09** kips

Vertical force distribution table

Level	Height from base to Level i (ft), h_x	Portion of effective seismic weight assigned to Level i (kips), w_x	Distribution exponent related to building period, k	Vertical distribution factor, C_{vx}	Lateral force induced at Level i (kips), F_x;
1	12.0;	44.9;	1.00;	0.386;	4.7;
2	21.0;	40.8;	1.00;	0.614;	7.4;

;

;

WIND LOADING (ASCE7-10)

WIND LOADING (ASCE7-10)

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

Using the directional design method

Tedds calculation version 2.0.20

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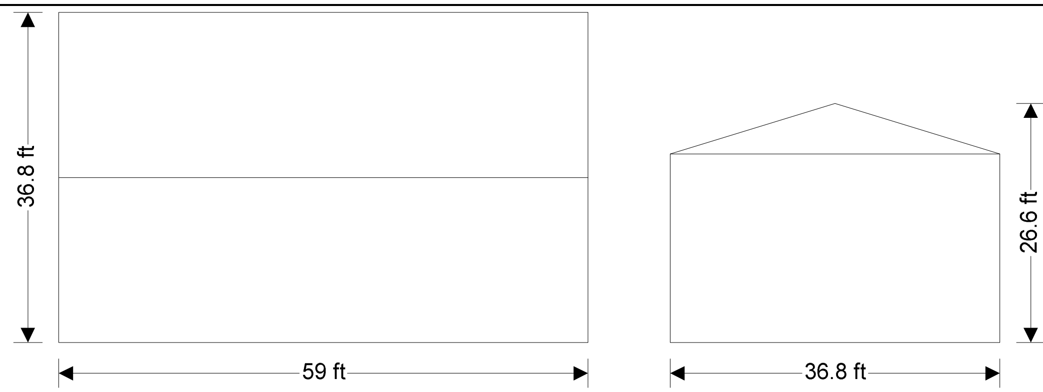
Project				Job Ref.	
Sun Residence Mercer Island				1956	
Section				Sheet no./rev.	
				54	
Calc. by	Date	Chk'd by	Date	App'd by	Date
GL	01-05-2020				



Plan   Elevation

Building data

Type of roof; Gable

Length of building; $b = \mathbf{59.00}$ ft

Width of building; $d = \mathbf{36.75}$ ft

Height to eaves; $H = \mathbf{21.00}$ ft

Pitch of roof; $\alpha_0 = \mathbf{17.0}$ deg

Mean height; $h = \mathbf{23.81}$ ft

General wind load requirements

Basic wind speed; $V = \mathbf{110.0}$ mph

Risk category; II

Velocity pressure exponent coeff (Table 26.6-1); $K_d = \mathbf{0.85}$

Exposure category (cl.26.7.3); B

Enclosure classification (cl.26.10); Enclosed buildings

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

Internal pressure coef –ve (Table 26.11-1); $GC_{pi\_n} = \mathbf{-0.18}$

Gust effect factor; $G_f = \mathbf{0.85}$

Topography

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

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

Velocity pressures table

z (ft)	K_z (Table 27.3-1)	q_z (psf)
15.00	0.57	15.01
15.00	0.57	15.01
20.00	0.62	16.32
21.00	0.63	16.53
23.81	0.65	17.13
26.62	0.67	17.72

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.); $q_i = \mathbf{17.13}$ psf

Pressures and forces

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

Net force; $F_w = p \times A_{ref}$;

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	23.81	-0.67	17.13	-12.82	1133.66	-14.53
B (-ve)	23.81	-0.56	17.13	-11.20	1133.66	-12.70

Total vertical net force; $F_{w,v}$ = **-26.04** kips

Total horizontal net force; $F_{w,h}$ = **-0.54** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	15.01	7.12	885.00	6.30
A_2	15.00	0.80	15.01	7.12	0.00	0.00
A_3	21.00	0.80	16.53	8.16	354.00	2.89
B	23.81	-0.50	17.13	-10.36	1239.00	-12.84
C	23.81	-0.70	17.13	-13.27	874.98	-11.61
D	23.81	-0.70	17.13	-13.27	874.98	-11.61

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_0} = b \times H$ = **1239.00** ft^2

Projected vertical area of roof; $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0)$ = **331.45** ft^2

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0}$ = **22.48** kips

Leeward net force; $F_l = F_{w,wB}$ = **-12.8** kips

Windward net force; $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3}$ = **9.2** kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **22.5** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	23.81	-0.13	17.13	1.20	1133.66	1.36
B (+ve)	23.81	-0.56	17.13	-5.04	1133.66	-5.71

Total vertical net force; $F_{w,v}$ = **-4.16** kips

Total horizontal net force; $F_{w,h}$ = **2.07** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	15.01	13.29	885.00	11.76
A_2	15.00	0.80	15.01	13.29	0.00	0.00

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_3	21.00	0.80	16.53	14.33	354.00	5.07
B	23.81	-0.50	17.13	-4.20	1239.00	-5.20
C	23.81	-0.70	17.13	-7.11	874.98	-6.22
D	23.81	-0.70	17.13	-7.11	874.98	-6.22

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_0} = b \times H = \mathbf{1239.00}$ ft^2

Projected vertical area of roof; $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) = \mathbf{331.45}$ ft^2

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = \mathbf{22.48}$ kips

Leeward net force; $F_l = F_{w,wB} = \mathbf{-5.2}$ kips

Windward net force; $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} = \mathbf{16.8}$ kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{24.1}$ kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	23.81	-0.90	17.13	-16.18	457.48	-7.40
B (-ve)	23.81	-0.90	17.13	-16.18	457.48	-7.40
C (-ve)	23.81	-0.50	17.13	-10.36	914.96	-9.48
D (-ve)	23.81	-0.30	17.13	-7.45	437.41	-3.26

Total vertical net force; $F_{w,v} = \mathbf{-26.34}$ kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	15.01	7.12	551.25	3.93
A_2	20.00	0.80	16.32	8.02	183.75	1.47
A_3	26.62	0.80	17.72	8.97	140.02	1.26
B	23.81	-0.38	17.13	-8.60	874.98	-7.52
C	23.81	-0.70	17.13	-13.27	1239.00	-16.45
D	23.81	-0.70	17.13	-13.27	1239.00	-16.45

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 = \mathbf{874.98}$ ft^2

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

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{14.00}$ kips

Leeward net force; $F_l = F_{w,wB} = \mathbf{-7.5}$ kips

Windward net force; $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3}$ = **6.7** kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **14.2** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	23.81	-0.18	17.13	0.46	457.48	0.21
B (+ve)	23.81	-0.18	17.13	0.46	457.48	0.21
C (+ve)	23.81	-0.18	17.13	0.46	914.96	0.42
D (+ve)	23.81	-0.18	17.13	0.46	437.41	0.20

Total vertical net force; $F_{w,v}$ = **1.00** kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	15.01	13.29	551.25	7.33
A_2	20.00	0.80	16.32	14.18	183.75	2.61
A_3	26.62	0.80	17.72	15.13	140.02	2.12
B	23.81	-0.38	17.13	-2.43	874.98	-2.13
C	23.81	-0.70	17.13	-7.11	1239.00	-8.81
D	23.81	-0.70	17.13	-7.11	1239.00	-8.81

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4$ = **874.98** ft^2

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

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90}$ = **14.00** kips

Leeward net force; $F_l = F_{w,wB}$ = **-2.1** kips

Windward net force; $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3}$ = **12.0** kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **14.2** kips

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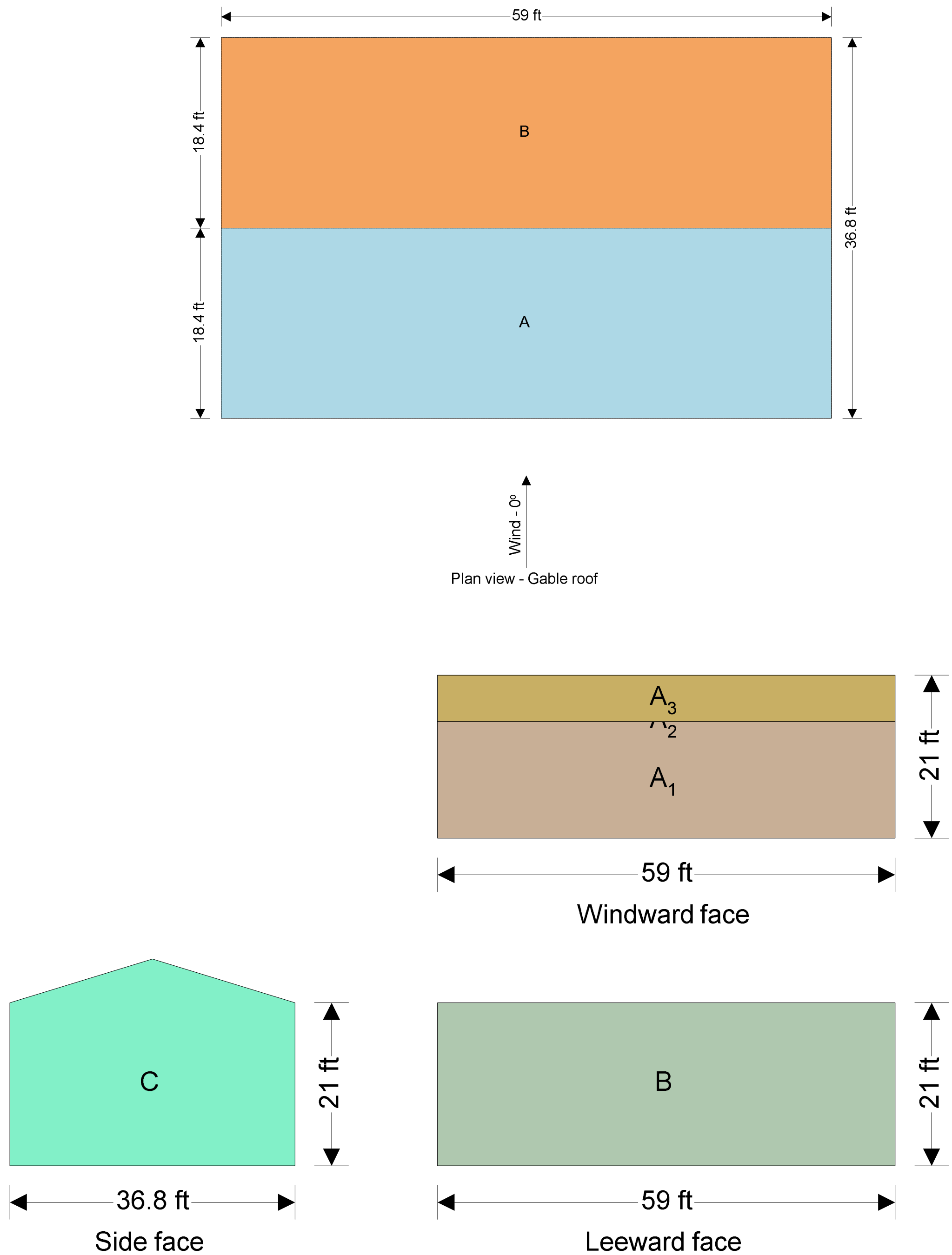
Project				Job Ref.	
Sun Residence Mercer Island				1956	
Section				Sheet no./rev.	
				58	
Calc. by	Date	Chk'd by	Date	App'd by	Date
GL	01-05-2020				



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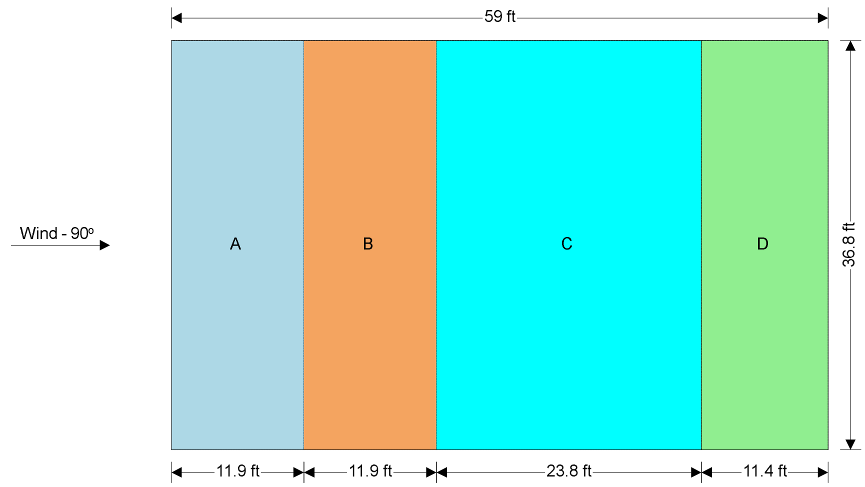
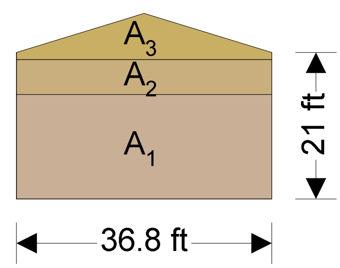
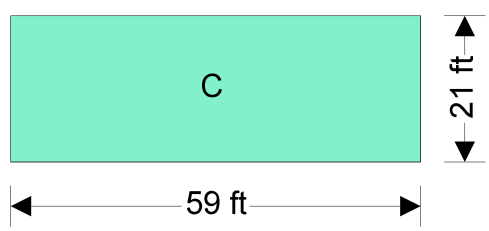
Project				Job Ref.	
Sun Residence Mercer Island				1956	
Section				Sheet no./rev.	
				59	
Calc. by	Date	Chk'd by	Date	App'd by	Date
GL	01-05-2020				



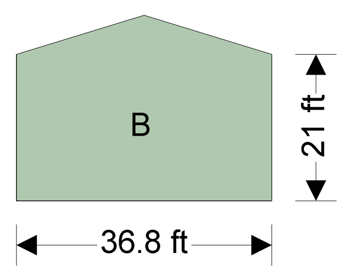
Plan view - Gable roof



Windward face



Side face

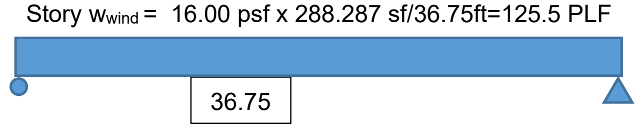


Leeward face

;_AT Roof Top Plates Level

Story w_{wind} = 16.00 psf x 288.287 sf/36.75ft=125.5 PLF

In the N-S direction



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Project				Job Ref.	
Sun Residence Mercer Island				1956	
Section				Sheet no./rev.	
				60	
Calc. by	Date	Chk'd by	Date	App'd by	Date
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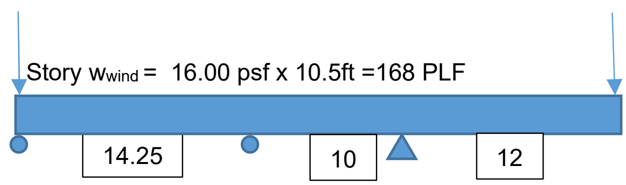
Wall Line	a				b	
Shear Force (LBF)	2306.0625				2306.063	
Wall Length (FT)	20.01				13.83	
Wall Unit Shear (LB/FT)	230.49				333.49	
Uplift (LBF)	832.14				1326.52	
Holdown	CS16				CS16	
SW Type	SW1				SW1	

At Upper Floor Top Plates Level

In the N-S direction

Story w_{wind} = 16.00 psf x 10.5ft =168 PLF

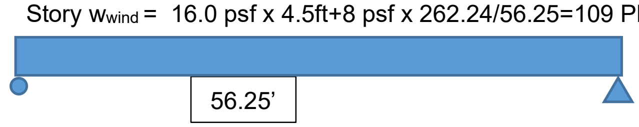


Wall Line	a		a.4		a.6	
Shear Force (LBF)	3503		2037.00		5162	
Wall Length (FT)	20.01		19.33		10	
Wall Unit Shear (LB/FT)	385.14		105.38		516.20	
Uplift (LBF)	2357.29		-464.29		2941.92	
Holdown	HTT4		NA		HTT4	
SW Type	SW4		SW1		SW2	

Roof Top Plates Level

In the E-W direction

Story w_{wind} = 16.0 psf x 4.5ft+8 psf x 262.24/56.25=109 PLF



Wall Line	1				2	
Shear Force (LBF)	3065.6				3065.6	
Wall Length (FT)	25.9				24.2	
Wall Unit Shear (LB/FT)	118.30				126.45	
Uplift (LBF)	458.84				502.84	
Holdown	CS16				CS16	
SW Type	SW1				SW1	

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Project	Job Ref.
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Section	Sheet no./rev. 61

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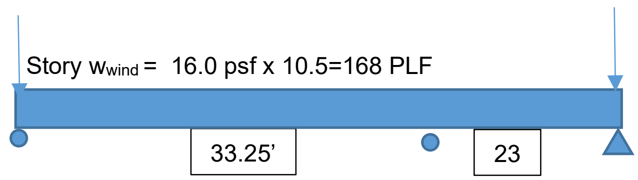
GL	01-05-2020				

Upper Top Plates Level

In the E-W direction



Wall Line	1		1.6		2	
Shear Force (LBF)	4877.725		3065.625		4319.1	
Wall Length (FT)	34.75		20.16		21.65	
Wall Unit Shear (LB/FT)	140.37		152.06		199.50	
Uplift (LBF)	1241.72		350.23		1701.05	
Holdown	HTT4		LTT19		HTT4	
SW Type	SW1		SW1		SW1	

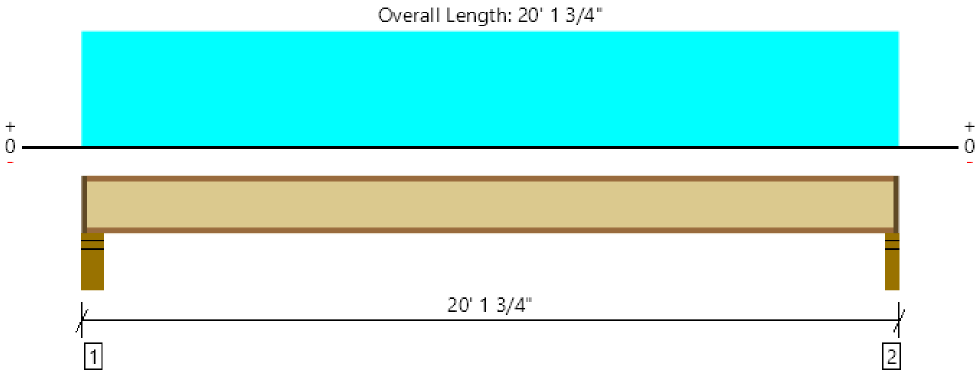
	SEISMIC lbf/ft (CAPACITY AT ULTIMATE LEVEL)	WIND lbf/ft (CAPACITY AT ULTIMATE LEVEL)
SW1	357	498
SW2	521	729
SW3	670	937
SW4	870	1220

MEMBER REPORT

PASSED

Floor, Floor: Joist at garage

1 piece(s) 11 7/8" TJI® 230 @ 12" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	514 @ 19' 11 1/4"	1183 (2.25")	Passed (43%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	504 @ 5 1/2"	1655	Passed (30%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2487 @ 10' 1 7/8"	4215	Passed (59%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.358 @ 10' 1 7/8"	0.489	Passed (L/656)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.465 @ 10' 1 7/8"	0.978	Passed (L/504)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	41	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Top Edge Bracing (Lu): Top compression edge must be braced at 5' 5" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 19' 11" o/c unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - HF	5.50"	4.25"	1.75"	122	406	528	1 1/4" Rim Board
2 - Stud wall - HF	3.50"	2.25"	1.75"	120	400	520	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 20' 1 3/4"	12"	12.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
GLu GL Architectural Engineering (360) 747-7509 akegl2002@gmail.com	

1/2/2020 9:58:17 PM UTC

ForteWEB v2.1, Engine: V7.3.2.309, Data: V7.2.0.2

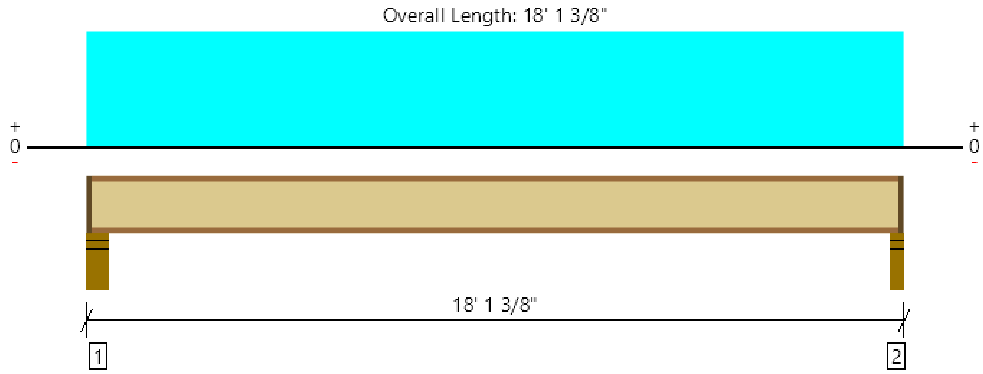
File Name: 1958_Sun Yong Mercer

SOLUTIONS REPORT

PASSED

Floor, Floor: Joist at living room

Current Solution: 1 piece(s) 11 7/8" TJI® 230 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	615 @ 17' 10 7/8"	1183 (2.25")	Passed (52%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	602 @ 5 1/2"	1655	Passed (36%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2664 @ 9' 1 11/16"	4215	Passed (63%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.306 @ 9' 1 11/16"	0.438	Passed (L/688)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.398 @ 9' 1 11/16"	0.877	Passed (L/529)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	42	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

All Product Solutions

Depth	Series	Plies	Spacing	**TJ-Pro™ Rating**	Wood Volume
11 7/8"	TJI® 210	1	12"	46	1.06
11 7/8"	TJI® 210	1	16"	41	0.79
11 7/8"	TJI® 210	2	12"	57	2.12
11 7/8"	TJI® 210	2	16"	53	1.59
11 7/8"	TJI® 230	1	12"	48	1.15
11 7/8"	TJI® 230	1	16"	42	0.86
11 7/8"	TJI® 230	2	12"	58	2.30
11 7/8"	TJI® 230	2	16"	54	1.72
11 7/8"	TJI® 360	1	12"	50	1.36
11 7/8"	TJI® 360	1	16"	45	1.02
11 7/8"	TJI® 360	2	12"	60	2.72
11 7/8"	TJI® 360	2	16"	56	2.04

The purpose of this report is for product comparison only. Load and support information necessary for professional design review is not displayed here. Please print an individual Member Report for submittal purposes.

ForteWEB Software Operator	Job Notes
GLu GL Architectural Engineering (360) 747-7509 akegl2002@gmail.com	

1/2/2020 9:58:03 PM UTC

ForteWEB v2.1, Engine: V7.3.2.309, Data: V7.2.0.2

File Name: 1958_Sun Yong Mercer