ENGINEERING CALCULATIONS

Location:

Prepared for:

Prepared by:

A.S. Engineering Services, P.C.

112 Wilson Drive

Port Jefferson, New York, 11777

Engineer of Record:

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112 Wilson Drive, Port Jefferson, NY 11777
(C) 631-560-0259

LOADING PARAMETERS

PER 2009 IRC

I =1.0
WIND SPEED = 90 MPH
EXP C
SEISMIC - SEE BELOW

FROM CALCS, MAXIMUM WIND PRESSURE ARE AS FOLLOWS

ROOM INPUT

PROJECTION	50.00	FT
EAVE HEIGTH	8.00	FT
RIDGE HEIGHT	16.00	FT
FRONT WALL LENGTH	95.00	FT

WIND PRESSURES	12.88	psf for	Windward Wall
	9.06	psf for	Leeward Wall
	11.61	psf for	Side Wall
	14.16	psf for	Roof

WIND BASE SHEAR CALCULATIONS

	X-Direction Surface Area =	600	sq ft for	peaked wall
	Y-Direction Survace Area =	950	sq ft for	side wall
Therefore,	X-Direction Wind Shear, V_{wx} =	13169	lbs	
	Y-Direction Wind Shear, V_{wY} =	12240	lbs	

NOTE THAT CORNER ZONE PRESSURES WERE UTILIZED FOR ENTIRE STRUCTURE, THEREFORE DESIGN IS CONSERVATIVE

BASE SHEAR:
Vx = 13.1K
Vy = 12.3K

BASE SHEAR MAIN HOUSE (EXCLUDES GARAGE) :
Vx = 13.1K
Vy = 8.4K

SHEAR AT ROOF:
Vx = 6.55K
Vy = 4.2K

MAIN ROOF DIAPHRAGM APPROXIMATELY 30'x65'
1/2 TO EACH SIDE
THEREFORE LOADS AS FOLLOWS:
Vx = 3.3K
Vy= 2.1K

SHEAR PER FOOT OF WALL:
vx = 3.3/65 = 51 PLF
vy = 70 PLF

THEREFORE DESIGN HORIZONTAL DIAPHRAGM TO RESIST THESE LOADS

LOADS ARE TRANSFERRED TO PERIMETER WALLS AND INTERNAL STEEL BEAM

APPLY FULL LOAD TO BEAM 3.3K SINCE SPLIT IN HALFT BETWEEN BEAM AND REAR WALL

IF APPLY LOAD AT TOP OF COLUMN, H = 11'

MOMENT TO BE RESISTED = 36.3K-FT = 435.6 K-IN

FOR STEEL COLUMN

Sreq = 15.7

USE HSS7x5x1/2 S=17.3 AND I = 50.7

CHECK DELTA = (P*L^3)/48EI = .11 OKAY LESS THAN .01 X H = 0.11

SEE RISA CALCS

NOW CHECK SHEAR WALLS

CHECK WORST CASE X FIRST V = 70 PLF

INSTEAD DUMPED FULL 3.3 KIPS INTO WALL ALONG REAR MASTER BEDROOM - OK

AGAIN DESIGN IS CONSERVATIVE SINCE SINGLE WALL CAN RESIST FULL SHEAR LOAD AND ALL WALLS ARE CONSTRUCTED IN SAME MANNER

SEE TEDDS OUTPUT

NOW CHECK Y-DIRECTION WALLS

2.1K EACH WALL

SEE TEDDS OUTPUT

CALC SEISMIC BASE SHEAR

TOTAL WEIGHT STRUCTURE

LENGHT EXTERIOR WALLS = 272'x12x10 AVE HEIGHT = 32640 LBS

LENGTH INTERIOR WALL BETWEEN KITCHEN AND GARAGE = 20x12x10=2400 LBS

ROOF = 3000 SF x 10 PSF = 30000 LBS

TOTAL WEIGHT STRUCTURE = 65040 LBS

SEISMIC BASE SHEAR = 7.5 KIPS

NOW TAKE 1/2 WALL WEIGHT + FULL ROOF FOR LOAD TO WALL = 47.5K

FORCE TO WALLS = 5.5 KIPS

0.5 TO EACH WALL = 2.75 KIPS

RECHECK TEDDS SHEAR WALL CALCS

WIND PRESSURE CALCS

WIND PERPENDICULAR TO RIDGE

WIND LOAD (ASCE 7 – 05)

TEDDS calculation version 1.2.03

Classification summary

Structure is a building

Structure is Rigid

Mean roof height; $h = \mathbf{15.0}$ ft

Horizontal dimension parallel to wind; $L = \mathbf{42.0}$ ft

Horizontal dimension normal to wind; $B = \mathbf{92.0}$ ft

Roof angle; $\theta = \mathbf{18.0}$ deg

Wind force resisting element is part of main wind force resisting system

Structure is enclosed

Structure is low rise

Procedure

Occupancy category (table 1-1); Category = **2**

Basic wind speed (sect. 6.5.4); $V = \mathbf{90.0}$ mph

Region; **Non-Hurricane Prone**

Importance factor (table 6-1); $I = \mathbf{1.00}$

Exposure category (sect. 6.5.6); **C**

Wind directionality factor; $K_d = \mathbf{0.85}$

Topographic factor; $K_{zt} = \mathbf{1.00}$

Design procedure - analytical procedure (Method2)

Velocity pressure at mean roof height ‘h’**(ASCE 7-05, cl. 6.5.10)**

Case of loading system (table 6-3); Case = **1**

Velocity pressure exposure coefficient; $K_h = \mathbf{0.85}$

Velocity pressure at mean roof height ‘h’; $q_h = 0.00256 \times K_h \times K_{zt} \times K_d \times V^2 \times I \times 1$ psf / mph^2 = **14.98** psf

Design wind pressure for MWFRS of low-rise enclosed and partially enclosed buildings (alternative procedure)

Velocity pressure at mean roof height ‘h’; $q_h = \mathbf{14.98}$ psf

External and internal pressure coefficients **(fig. 6-5)**

Positive internal pressure coefficient; $GC_{pi\_pos} = \mathbf{0.18}$

Negative internal pressure coefficient; $GC_{pi\_neg} = \mathbf{-0.18}$

Building surface 1

External pressure coeff. for surface 1 (fig. 6-10); $GC_{pf\_1} = \mathbf{0.51}$

With positive GC_{pi}; $p_{1\_S1} = q_h \times ((GC_{pf\_1}) - (GC_{pi\_pos})) = \mathbf{4.98}$ psf

With negative GC_{pi}; $p_{2\_S1} = q_h \times ((GC_{pf\_1}) - (GC_{pi\_neg})) = \mathbf{10.38}$ psf

Building surface 2

External pressure coeff. for surface 2 (fig. 6-10); $GC_{pf\_2} = \mathbf{-0.69}$

With positive GC_{pi}; $p_{1\_S2} = q_h \times ((GC_{pf\_2}) - (GC_{pi\_pos})) = \mathbf{-13.03}$ psf

With negative GC_{pi}; $p_{2\_S2} = q_h \times ((GC_{pf\_2}) - (GC_{pi\_neg})) = \mathbf{-7.64}$ psf

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Building surface 3

External pressure coeff. for surface 3 (fig. 6-10); GC_{pf_3} = **-0.47**

With positive GC_{pi}; $p_{1\_S3} = q_h \times ((GC_{pf\_3}) - (GC_{pi\_pos}))$ = **-9.67** psf

With negative GC_{pi}; $p_{2\_S3} = q_h \times ((GC_{pf\_3}) - (GC_{pi\_neg}))$ = **-4.27** psf

Building surface 4

External pressure coeff. for surface 4 (fig. 6-10); GC_{pf_4}= **-0.41**

With positive GC_{pi}; $p_{1\_S4} = q_h \times ((GC_{pf\_4}) - (GC_{pi\_pos}))$ = **-8.86** psf

With negative GC_{pi}; $p_{2\_S4} = q_h \times ((GC_{pf\_4}) - (GC_{pi\_neg}))$ = **-3.47** psf

Building surface 5

External pressure coeff. for surface 5 (fig. 6-10); GC_{pf_5} = **-0.45**

With positive GC_{pi}; $p_{1\_S5} = q_h \times ((GC_{pf\_5}) - (GC_{pi\_pos}))$ = **-9.44** psf

With negative GC_{pi}; $p_{2\_S5} = q_h \times ((GC_{pf\_5}) - (GC_{pi\_neg}))$ = **-4.05** psf

Building surface 6

External pressure coeff. for surface 6 (fig. 6-10); GC_{pf_6} = **-0.45**

With positive GC_{pi}; $p_{1\_S6} = q_h \times ((GC_{pf\_6}) - (GC_{pi\_pos}))$ = **-9.44** psf

With negative GC_{pi}; $p_{2\_S6} = q_h \times ((GC_{pf\_6}) - (GC_{pi\_neg}))$ = **-4.05** psf

Building surface 1E

External pressure coeff. for surface 1E (fig. 6-10); GC_{pf_1E} = **0.77**

With positive GC_{pi}; $p_{1\_S1E} = q_h \times ((GC_{pf\_1E}) - (GC_{pi\_pos}))$ = **8.91** psf

With negative GC_{pi}; $p_{2\_S1E} = q_h \times ((GC_{pf\_1E}) - (GC_{pi\_neg}))$ = **14.30** psf

Building surface 2E

External pressure coeff. for surface 2E (fig. 6-10); GC_{pf_2E} = **-1.07**

With positive GC_{pi}; $p_{1\_S2E} = q_h \times ((GC_{pf\_2E}) - (GC_{pi\_pos}))$ = **-18.73** psf

With negative GC_{pi}; $p_{2\_S2E} = q_h \times ((GC_{pf\_2E}) - (GC_{pi\_neg}))$ = **-13.33** psf

Building surface 3E

External pressure coeff. for surface 3E (fig. 6-10); GC_{pf_3E} = **-0.67**

With positive GC_{pi}; $p_{1\_S3E} = q_h \times ((GC_{pf\_3E}) - (GC_{pi\_pos}))$ = **-12.71** psf

With negative GC_{pi}; $p_{2\_S3E} = q_h \times ((GC_{pf\_3E}) - (GC_{pi\_neg}))$ = **-7.32** psf

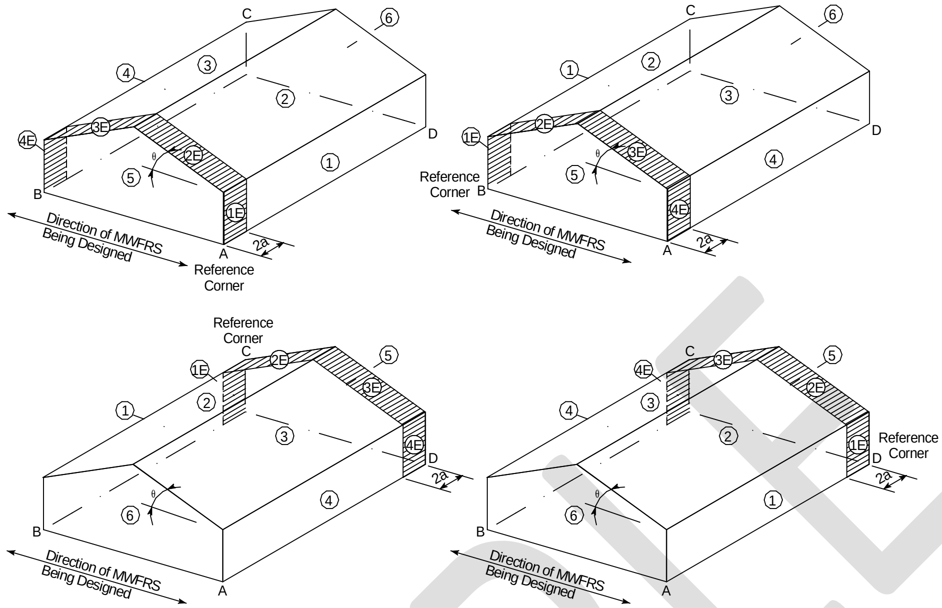
Building surface 4E

External pressure coeff. for surface 4E (fig. 6-10); GC_{pf_4E} = **-0.61**

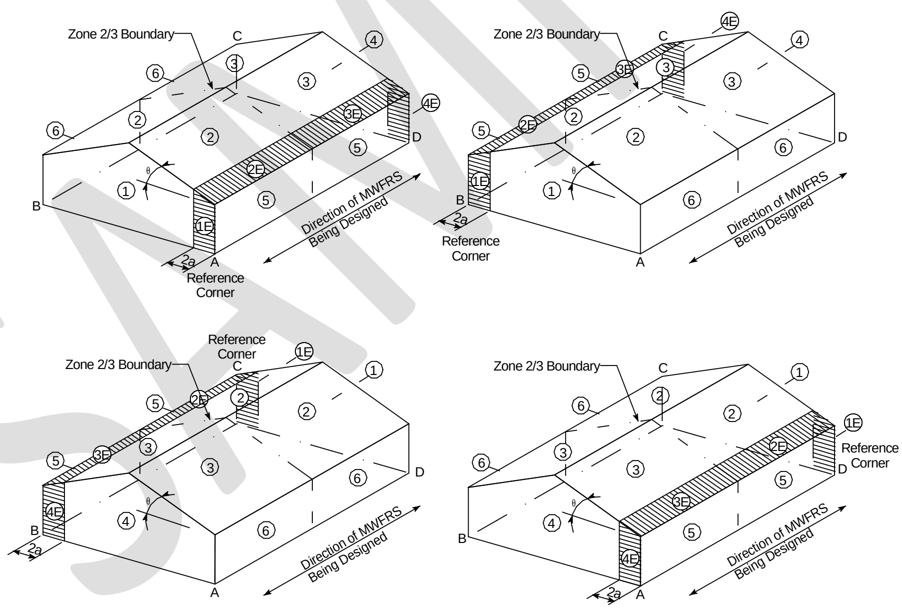
With positive GC_{pi}; $p_{1\_S4E} = q_h \times ((GC_{pf\_4E}) - (GC_{pi\_pos}))$ = **-11.87** psf

With negative GC_{pi}; $p_{2\_S4E} = q_h \times ((GC_{pf\_4E}) - (GC_{pi\_neg}))$ = **-6.47** psf

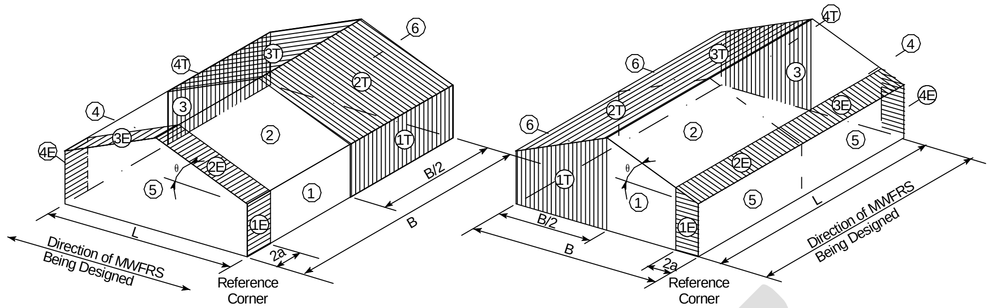
Note: - As per Section 6.1.4.1, the wind load to be used in the design of the MWFRS shall be not less than 10 psf multiplied by the area of the building or structure projected onto a vertical plane normal to the wind direction.



Basic Load Cases - Transverse Direction



Basic Load Cases - Longitudinal Direction



Torsional Load Case

Transverse Direction **Longitudinal Direction**

WIND PARALLEL TO RIDGE

WIND LOAD (ASCE 7 – 05)

TEDDS calculation version 1.2.03

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Mean roof height;	h = **15.0** ft
Horizontal dimension parallel to wind;	L = **92.0** ft
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Wind force resisting element is part of main wind force resisting system

Structure is enclosed

Structure is low rise

Procedure

Occupancy category (table 1-1);	Category = **2**
Basic wind speed (sect. 6.5.4);	V = **90.0** mph
Region;	**Non-Hurricane Prone**
Importance factor (table 6-1);	I = **1.00**
Exposure category (sect. 6.5.6);	**C**
Wind directionality factor;	K_d = **0.85**
Topographic factor;	K_{zt} = **1.00**

Design procedure - analytical procedure (Method2)

Velocity pressure at mean roof height ‘h’**(ASCE 7-05, cl. 6.5.10)**

Case of loading system (table 6-3);	Case = **1**
Velocity pressure exposure coefficient;	K_h = **0.85**
Velocity pressure at mean roof height ‘h’;	$q_h = 0.00256 \times K_h \times K_{zt} \times K_d \times V^2 \times I \times 1$ psf / mph^2 = **14.98** psf

Design wind pressure for MWFRS of low-rise enclosed and partially enclosed buildings (alternative procedure)

Velocity pressure at mean roof height ‘h’; q_h = **14.98** psf

External and internal pressure coefficients **(fig. 6-5)**

Positive internal pressure coefficient; GC_{pi_pos} = **0.18**

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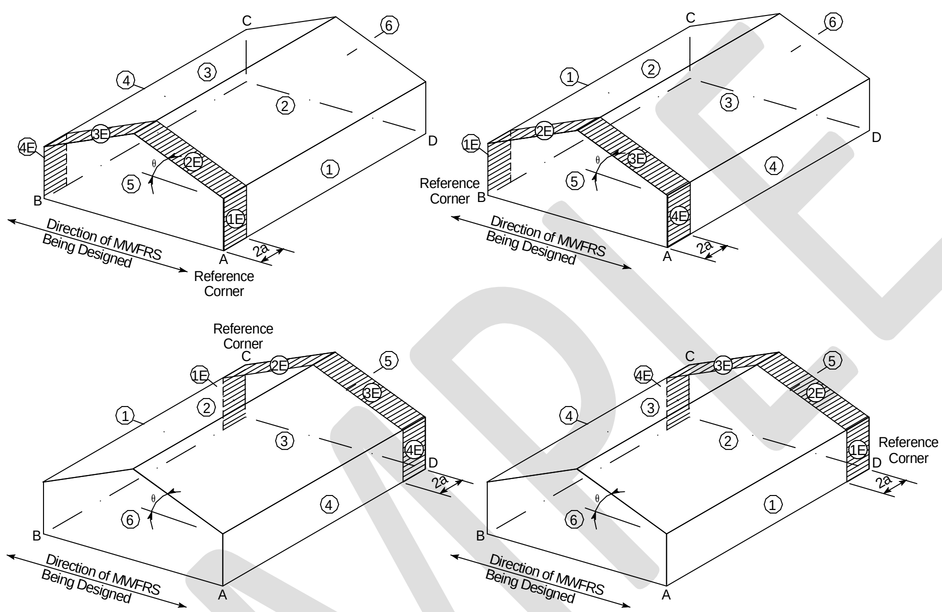

Basic Load Cases - Transverse Direction

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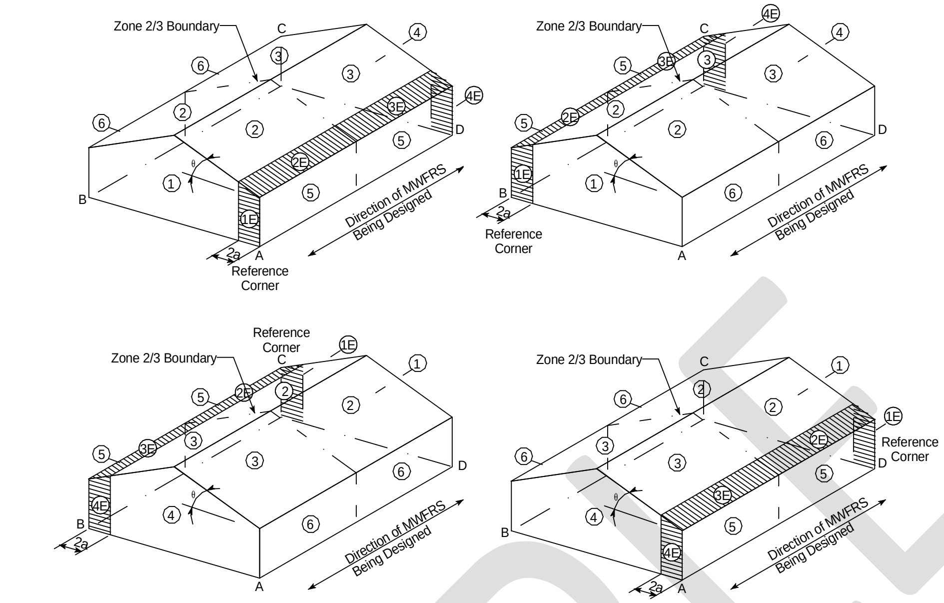
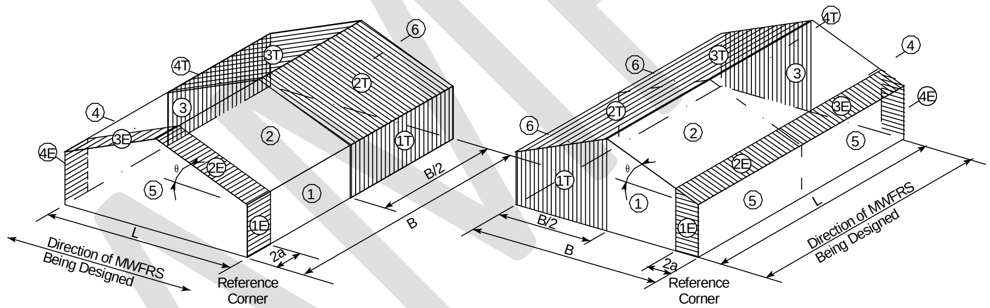
	Section				Sheet no./rev. 12 of 81	
	Calc. by AS	Date 5/15/2012	Chk'd by	Date	App'd by	Date



Basic Load Cases - Longitudinal Direction



Torsional Load Case

Transverse Direction **Longitudinal Direction**

SEISMIC CALCS

SEISMIC FORCES (ASCE 7-05)

Tedds calculation version 3.0.01

Site parameters

Site class;	D
Mapped acceleration parameters (Section 11.4.1)	
at short period;	S_S = **1.027**
at 1 sec period;	S_1 = **0.354**
Site coefficientat short period (Table 11.4-1);	F_a = **1.1**
at 1 sec period (Table 11.4-2);	F_v = **1.7**
Spectral response acceleration parameters	
at short period (Eq. 11.4-1);	$S_{MS} = F_a \times S_S$ = **1.119**
at 1 sec period (Eq. 11.4-2);	$S_{M1} = F_v \times S_1$ = **0.599**
Design spectral acceleration parameters (Sect 11.4.4)	
at short period (Eq. 11.4-3);	$S_{DS} = 2 / 3 \times S_{MS}$ = **0.746**
at 1 sec period (Eq. 11.4-4);	$S_{D1} = 2 / 3 \times S_{M1}$ = **0.399**
Seismic design category	
Occupancy category (Table 1-1);	II

Seismic design category based on short period response acceleration (Table 11.6-1)
D

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)
D

Seismic design category;	D
Approximate fundamental period	
Height above base to highest level of building;	h_n = **14** ft

From Table 12.8-2:

Structure type;	All other systems
Building period parameter C_t;	C_t = **0.02**
Building period parameter x;	x = **0.75**
Approximate fundamental period (Eq 12.8-7);	$T_a = C_t \times (h_n)^x \times 1sec / (1ft)^x$= **0.145** sec
Building fundamental period (Sect 12.8.2);	$T = T_a$ = **0.145** sec
Long-period transition period;	T_L = **16** sec
Seismic response coefficient	
Seismic force-resisting system (Table 12.14-1);	A. Bearing_Wall_Systems 13.Light-framed walls sheathed with wood panels ratedfor shr/stl
Response modification factor (Table 12.14-1);	R = **6.5**
Seismic importance factor (Table 11.5-2);	I_e = **1.000**
Seismic response coefficient (Sect 12.8.1.1)	
Calculated (Eq 12.8-2);	$C_{s\_calc} = S_{DS} / (R / I_e)$= **0.115**

Maximum (Eq 12.8-3); $C_{s\_max} = S_{D1} / (T \times (R / I_e)) = \mathbf{0.424}$

Minimum (Eq 12.8-5,Supp. No. 2); $C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01) = \mathbf{0.033}$

Seismic response coefficient; $C_s = \mathbf{0.115}$

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure; W = **65.0** kips

Seismic response coefficient; $C_s = \mathbf{0.115}$

Seismic base shear (Eq 12.8-1); $V = C_s \times W = \mathbf{7.5}$ kips

;

SEISMIC ACCELERATION PARAMETERS FROM USGS

Conterminous 48 States
2009 International Building Code
Zip Code = 98245
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Data are based on a 0.05 deg grid spacing

Period (sec)	Centroid Sa (g)	
0.2	1.027	(Ss)
1.0	0.354	(S1)

Period (sec)	Maximum Sa (g)	
0.2	1.057	(Ss)
1.0	0.371	(S1)

Period (sec)	Minimum Sa (g)	
0.2	1.024	(Ss)
1.0	0.352	(S1)

SNOW LOADS

SNOW LOADING (ASCE7-10)

TEDDS calculation version 1.0.01

Building details

Roof type;	**Monopitch**
Width of roof;	**b =** 42.00 **ft**
Slope of roof 1;	α **=** 18.00 **deg**

Ground snow load

Ground snow load;	p_g = 25.00 lb/ft^2
Density of snow;	$\gamma = \min(0.13 \times p_g / 1ft + 14lb/ft^3, 30lb/ft^3)$ = 17.25 lb/ft^3
Terrain type;	**B**
Exposure condition (Table 7-2);	**Fully exposed**
Exposure factor (Table 7-2);	C_e = 0.90
Thermal condition (Table 7-3);	**Structures kept just above freezing**
Thermal factor (Table 7-3);	C_t = 1.10
Importance category (Table 1-1);	**II**
Importance factor (Table 7-4);	I_s = 1.00
Flat roof snow load (Sect 7.3);	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g$ = 17.33 lb/ft^2

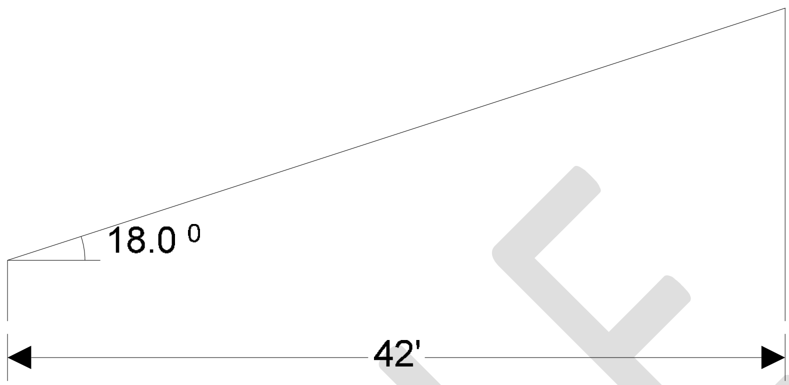
Cold roof slope factor (C_t > 1.0)

Roof surface type;	**Non slippery**
Ventilation;	**Ventilated**
Thermal resistance (R-value);	**R =** 38.00;^{o}F h ft^2 / Btu
Roof slope factor Fig 7-2b (solid line);	C_s = 1.00

Monoslope

Sloped roof snow load (Cl.7.4);	$p_s = C_s \times p_f$ = 17.33 lb/ft^2

Balanced load 17.3 psf



Roof elevation

Drift calculations

Balanced snow load height;	$h_b = p_f / \gamma = 1.00$ **ft**
Length of upper roof;	$l_u = 42.00$ **ft**
Length of lower roof;	$l_l = 15.00$ **ft**
Height diff between uppper and lower roofs;	$h_{diff} = 6.00$ **ft**
Height from balance load to top of upper roof;	$h_c = h_{diff} - h_b = 5.00$ **ft**
Drift height leeward drift;	$h_{d\_l} = 0.43 \times (\max(20\ ft, l_u) \times 1ft^2)^{1/3} \times (p_g / 1lb/ft^2 + 10)^{1/4} - 1.5ft = 2.14$ **ft**
Drift height windward drift;	$h_{d\_w} = 0.75 \times (0.43 \times (\max(25\ ft, l_l) \times 1ft^2)^{1/3} \times (p_g / 1lb/ft^2 + 10)^{1/4} - 1.5ft) = 1.17$ **ft**
Maximum lw/ww drift height;	$h_{d\_max} = \max(h_{d\_w}, h_{d\_l}) = 2.14$ **ft**
Drift height;	$h_d = \min(h_{d\_max}, h_c) = 2.14$ **ft**
Drift width;	$W_d = \min(4 \times h_{d\_max}, 8 \times h_c) = 8.54$ **ft**
Drift surcharge load;	$p_d = h_d \times \gamma = 36.84$ **lb/ft^2**

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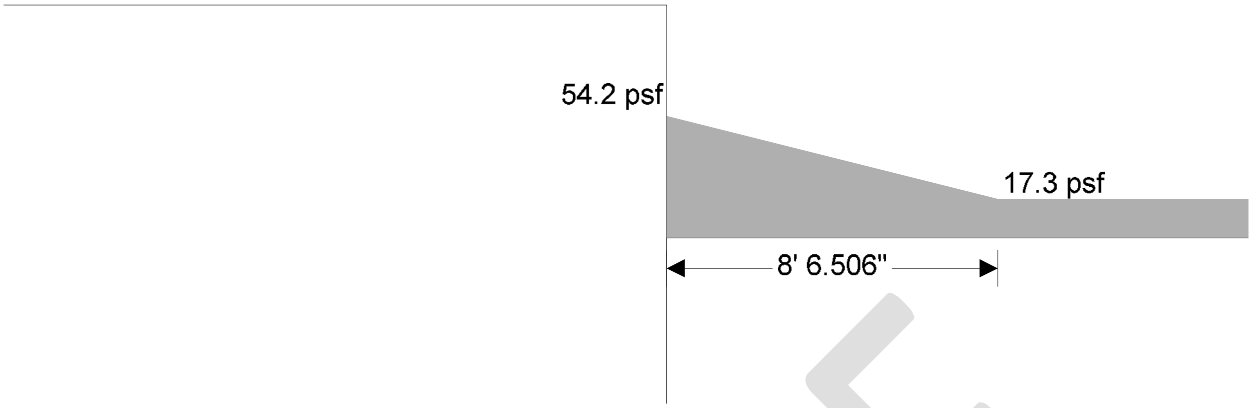


Elevation on snow drift

UTILIZED 20 PSF FOR UNFORM LOAD IN COMPONENT ANALYSIS

DRIFT LOAD ONLYL APPLIES TO RAFTERS OF LOWER ROOF THAT FRAME INTO BEAM - SINCE STILL UTILIZING 2x12 FOR THESE RAFTERS - BY OBSERVATION DESIGN OK

EQUIVALENT UNIFORM DRIFT LOAD = 21 PSF

MOMENT CONNECTION CALCS

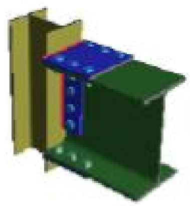
ALTHOUGH CALCS DO NOT EXACTLY MATCH CONDITION, THEY ARE VALID - ONLY DIFFERENCE IS COLUMN SIZE/TYPE. WELD OF PLATES TO COLUMNS IS MORE THAN ADEQUATE

SEE FOLLOWING SHEETS FOR RISA OUTPUT

Connection 1

ASD

Column/Beam Flange Plate Moment Connection



Material Properties:				
Column	W8X35	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Beam	W16X67	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Plate	P0.38X4.00X12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Moment Plate	P0.50X10.25X9.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:		
Load	15.00 kips	*User Input Load*
Moment	436.00 kips-in	*User Input Moment*
Top Column Dist	20.00 in	*User Input Top Column Dist*
Column Force	30.00 kips	*User Input Column Force*
Story Shear	0.00 kips	*User Input Story Shear*

Note: Unless specified, all code references are from AISC 360-05 — **Collapse All** — **Expand All**

Limit State	Required	Available	Unity Check	Result
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	Pass	*Condition: S_{min} >= (2+2/3) * d_{bolt}*		*(J3.3)*
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	Pass	*Condition: S_{max}<= min(12.00 in, 24*t)*		*(J3.5a)*
S_{max}	3.00 in	*Max bolt spacing*		
t	0.38 in	*Thickness of governing element (Plate)*		
Check Min Edge Distance	Pass	*Condition: ED_{min} >= ED_{allow}*		*(J3.4)*
Check Max Edge Distance	Pass	*Condition: ED_{max} <= min (6.00 in, 12*t)*		*(J3.5)*
Geometry Restrictions at Flange Beam				**PASS**
Check Min Bolt Spacing	Pass	*Condition: S_{min} >= (2+2/3) * d_{bolt}*		*(J3.3)*
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	Pass	*Condition: S_{max}<= min(12.00 in, 24*t)*		*(J3.5a)*
S_{max}	7.25 in	*Max bolt spacing*		
t	0.50 in	*Thickness of governing element (Moment Plate)*		
Check Min Edge Distance	Pass	*Condition: ED_{min} >= ED_{allow}*		*(J3.4)*
Check Max Edge Distance	Pass	*Condition: ED_{max} <= min (6.00 in, 12*t)*		*(J3.5)*
Flange Plate Weld Limitations				**PASS**
Check Weld Min Size	Pass			

D	0.25 in	*Weld size*
D_{min}	0.19 in	*Min size allowed*
t_{min}	0.50 in	*Controlling member thickness*
Check Weld Min Length	Pass	*Condition: $w_p >= 4 * D$*
D	0.25 in	*Weld size*
w_p	10.25 in	*Plate width*
Check Weld Max Length	Pass	*Condition: $w_p <= 100 * D$*
D	0.25 in	*Weld size*
w_p	10.25 in	*Plate width*

Column Weld Limitations — **PASS**

Weld Max/Min Size, Length		*(J2.2b)*
Check Weld Min Size	Pass	
D	0.25 in	*Weld size*
D_{min}	0.19 in	*Min size allowed*
t_{min}	0.38 in	*Controlling member thickness*
Check Weld Min Length	Pass	*Condition: $L_{min} >= 4*D$*
D	0.25 in	*Weld size*
L_{min}	12.00 in	*Min weld segment length*
Check Weld Max Length	Pass	*Condition: $L_{max} <= 100*D$*
D	0.25 in	*Weld size*
L_{max}	12.00 in	*Max weld segment length*

Shear Plate Weld Strength at Column — 15.00 kips — 78.38 kips — **0.19** — **PASS**

$Rn/\Omega = 2 * C_1 * \alpha * 0.928 * D_{16} * L$

Double Fillet

C_1	1.00	*Electrode strength coefficient*
α	0.88	*Base material proration factor*
D_{16}	4.00	*Weld fillet size in sixteenths of an inch*
L	12.00 in	*Weld length per side*
Rn/Ω	78.38 kips	*Weld strength*

Beam Web Shear Yield — 15.00 kips — 129.01 kips — **0.12** — **PASS**

$R_n = 0.6 * F_y * A_g * C_v$ — $\Omega = 1.50$ *(G2-1)*

F_y	50.00 ksi	*Minimum yield stress of material*
A_g	6.45 in^2	*Gross area subject to shear*
C_v	1.00	*Web shear coefficient (G2-2)*
R_n/Ω	129.01 kips	*Shear yield strength*

Vert. Plate Shear Yield — 15.00 kips — 64.80 kips — **0.23** — **PASS**

$R_n = 0.6 * F_y * A_g$ — $\Omega = 1.50$ *(J4-3)*

F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.50 in^2		*Gross area subject to shear*		
R_n/Ω	64.80 kips		*Shear yield strength*		
Beam Web Shear Rupture		15.00 kips	98.82 kips	**0.15**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$			$\Omega = 2.0$ *(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	5.07 in^2		*Net area subject to shear*		
R_n/Ω	98.82 kips		*Shear rupture strength*		
Vert. Plate Shear Rupture		15.00 kips	55.46 kips	**0.27**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$			$\Omega = 2.0$ *(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	3.19 in^2		*Net area subject to shear*		
R_n/Ω	55.46 kips		*Shear rupture strength*		
Beam Web Block Shear		15.00 kips	81.48 kips	**0.18**	**PASS**
$R_n = [\min(0.6*F_u*A_{nv}, 0.6*F_y*A_g) + U_{bs}*F_u*A_{nt}]$			$\Omega = 2.00$ *(J4-5)*		
A_g	4.69 in^2		*Gross area subject to shear*		
A_{nv}	3.48 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.42 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
R_n/Ω	81.48 kips		*Block shear strength*		
Vert. Plate Block Shear		15.00 kips	59.52 kips	**0.25**	**PASS**
$R_n = [\min(0.6*F_u*A_{nv}, 0.6*F_y*A_g) + U_{bs}*F_u*A_{nt}]$			$\Omega = 2.00$ *(J4-5)*		
A_g	3.94 in^2		*Gross area subject to shear*		
A_{nv}	2.79 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.59 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
R_n/Ω	59.52 kips		*Block shear strength*		
Bolt Shear at Beam Web		15.00 kips	42.41 kips	**0.35**	**PASS**
$R_n = F_{nv}*A_b*N_{bolt}*C$			$\Omega = 2.00$ *(J3-1)*		
F_{nv}	48.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		

N_{bolt}	4	*Number of bolts*
C	1.00	*Eccentricity coefficient*
R_n/Ω	42.41 kips	*Bolt shear rupture strength*

Bolt Bearing at Beam Web — 15.00 kips — 92.43 kips — **0.16** — **PASS**

$R_n = N_{rows} * N_{cols} * R_{n\text{-}spacing}$ — $\Omega = 2.00$ *(J3-6a)*

N_{rows}	1	*Number of rows of bolts*
N_{cols}	4	*Number of bolts per row*
d	0.75 in	*Bolt diameter*
t	0.40 in	*Thickness of material*
F_u	65.00 ksi	*Minimum tensile stress of material*
$L_{c\text{-}spacing}$	2.19 in	*Vertical distance from edges of adjacent holes*
$R_{n\text{-}spacing}$	46.22 kips	*Strength at spaces* = $min(R_{n\text{-}spacing\text{-}tearout}, R_{n\text{-}bearing})$
$R_{n\text{-}bearing}$	46.22 kips	*Bearing* = $2.4*d*t*F_u$
$R_{n\text{-}spacing\text{-}tearout}$	67.40 kips	*Tear out at spaces* = $1.2*L_{c\text{-}spacing}*t*F_u$
R_n/Ω	92.43 kips	*Bolt bearing strength*

Bolt Bearing at Vert. Plate — 15.00 kips — 73.00 kips — **0.21** — **PASS**

$R_n = N_{rows} * [R_{n\text{-}edge} + (N_{cols} - 1) * R_{n\text{-}spacing}]$ — $\Omega = 2.00$ *(J3-6a)*

N_{rows}	1	*Number of rows of bolts*
N_{cols}	4	*Number of bolts per row*
d	0.75 in	*Bolt diameter*
t	0.38 in	*Thickness of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
$L_{c\text{-}edge}$	1.09 in	*Vertical distance from edge of hole to edge of material*
$L_{c\text{-}spacing}$	2.19 in	*Vertical distance from edges of adjacent holes*
$R_{n\text{-}edge}$	28.55 kips	*Strength at edge* = $min(R_{n\text{-}edge\text{-}tearout}, R_{n\text{-}bearing})$
$R_{n\text{-}spacing}$	39.15 kips	*Strength at spaces* = $min(R_{n\text{-}spacing\text{-}tearout}, R_{n\text{-}bearing})$
$R_{n\text{-}bearing}$	39.15 kips	*Bearing* = $2.4*d*t*F_u$
$R_{n\text{-}edge\text{-}tearout}$	28.55 kips	*Tear out at edge* = $1.2*L_{c\text{-}edge}*t*F_u$
$R_{n\text{-}spacing\text{-}tearout}$	57.09 kips	*Tear out at spaces* = $1.2*L_{c\text{-}spacing}*t*F_u$
R_n/Ω	73.00 kips	*Bolt bearing strength*

Bolt Shear at Flange Plate — 436.00 kips-in — 996.56 kips-in — **0.44** — **PASS**

$R_n = F_{nv} * A_b * N_{bolt} * C$ — $\Omega = 2.00$ *(J3-1)*

F_{nv}	48.00 ksi	*Shear stress N type*
A_b	0.44 in^2	*Area of bolt*

N_{bolt}	6	*Number of bolts*
C	1.00	*Eccentricity coefficient*
R_n/Ω	63.62 kips	*Bolt shear rupture strength*
d_m	15.66 in	*Moment arm between the flange forces*
M_n/Ω	996.56 kips-in	*Moment = $R_n * d_m$*

Bolt Bearing at Beam Flange	436.00 kips-in	3326.35 kips-in	**0.13**	**PASS**

$R_n = N_{rows}*[R_{n\text{-}edge}+(N_{cols}-1)*R_{n\text{-}spacing}]$ $\Omega = 2.00$ *(J3-6a)*

N_{rows}	2	*Number of rows of bolts*
N_{cols}	3	*Number of bolts per row*
d	0.75 in	*Bolt diameter*
t	0.66 in	*Thickness of material*
F_u	65.00 ksi	*Minimum tensile stress of material*
$L_{c\text{-}edge}$	1.09 in	*Distance from edge of hole to edge of material*
$L_{c\text{-}spacing}$	2.19 in	*Distance between adjacent hole edges*
$R_{n\text{-}edge}$	56.73 kips	*Strength at edge = min($R_{n\text{-}edge\text{-}tearout}$, $R_{n\text{-}bearing}$)*
$R_{n\text{-}spacing}$	77.80 kips	*Strength at spaces = min($R_{n\text{-}spacing\text{-}tearout}$, $R_{n\text{-}bearing}$)*
$R_{n\text{-}bearing}$	77.80 kips	*Bearing = 2.4*d*t*F_u*
$R_{n\text{-}edge\text{-}tearout}$	56.73 kips	*Tear out at edge = 1.2*$L_{c\text{-}edge}$*t*F_u*
$R_{n\text{-}spacing\text{-}tearout}$	113.47 kips	*Tear out at spaces = 1.2*$L_{c\text{-}spacing}$*t*F_u*
R_n/Ω	212.34 kips	*Bolt bearing strength*
d_m	15.66 in	*Moment arm between the flange forces*
M_n/Ω	3326.35 kips-in	*Moment = $R_n * d_m$*

Bolt Bearing at Flange Plate	436.00 kips-in	2397.64 kips-in	**0.18**	**PASS**

$R_n = N_{rows}*[R_{n\text{-}edge}+(N_{cols}-1)*R_{n\text{-}spacing}]$ $\Omega = 2.00$ *(J3-6a)*

N_{rows}	2	*Number of rows of bolts*
N_{cols}	3	*Number of bolts per row*
d	0.75 in	*Bolt diameter*
t	0.50 in	*Thickness of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
$L_{c\text{-}edge}$	1.09 in	*Distance from edge of hole to edge of material*
$L_{c\text{-}spacing}$	2.19 in	*Distance between adjacent hole edges*
$R_{n\text{-}edge}$	38.06 kips	*Strength at edge = min($R_{n\text{-}edge\text{-}tearout}$, $R_{n\text{-}bearing}$)*
$R_{n\text{-}spacing}$	52.20 kips	*Strength at spaces = min($R_{n\text{-}spacing\text{-}tearout}$, $R_{n\text{-}bearing}$)*
$R_{n\text{-}bearing}$	52.20 kips	*Bearing = 2.4*d*t*F_u*
$R_{n\text{-}edge\text{-}tearout}$	38.06 kips	*Tear out at edge = 1.2*$L_{c\text{-}edge}$*t*F_u*

$R_{n\text{-spacing-tearout}}$	76.13 kips	*Tear out at spaces* $= 1.2*L_{c\text{-spacing}}*t*F_u$
R_n/Ω	142.46 kips	*Bolt bearing strength*
d_m	16.83 in	*Moment arm between the flange forces*
M_n/Ω	2397.64 kips-in	*Moment* $= R_n * d_m$

Beam Flange Block Shear 436.00 kips-in 2872.68 kips-in 0.15 **PASS**

$R_n = 2 * [\min(0.6*F_u*A_{nv}, 0.6*F_y*A_g) + U_{bs}*F_u*A_{nt}]$ $\Omega = 2.00$ *(J4-5)*

A_g	4.99 in^2	*Gross area subject to shear*
A_{nv}	3.53 in^2	*Net area subject to shear*
U_{bs}	1.00	*Uniform tension stress factor*
A_{nt}	0.70 in^2	*Net area subject to tension*
F_u	65.00 ksi	*Minimum tensile stress of material*
F_y	50.00 ksi	*Minimum yield stress of material*
R_n/Ω	183.38 kips	*Block shear strength*
d_m	15.66 in	*Moment arm between the flange forces*
M_n/Ω	2872.68 kips-in	*Moment* $= R_n * d_m$

Flange Plate Block Shear 436.00 kips-in 1881.80 kips-in 0.23 **PASS**

$R_n = 2 * [\min(0.6*F_u*A_{nv}, 0.6*F_y*A_g) + U_{bs}*F_u*A_{nt}]$ $\Omega = 2.00$ *(J4-5)*

A_g	3.75 in^2	*Gross area subject to shear*
A_{nv}	2.66 in^2	*Net area subject to shear*
U_{bs}	1.00	*Uniform tension stress factor*
A_{nt}	0.53 in^2	*Net area subject to tension*
F_u	58.00 ksi	*Minimum tensile stress of material*
F_y	36.00 ksi	*Minimum yield stress of material*
R_n/Ω	111.81 kips	*Block shear strength*
d_m	16.83 in	*Moment arm between the flange forces*
M_n/Ω	1881.80 kips-in	*Moment* $= R_n * d_m$

Flange Plate Tearout 436.00 kips-in 2918.95 kips-in 0.15 **PASS**

$R_n = [\min(0.6*F_u*A_{nv}, 0.6*F_y*A_g) + U_{bs}*F_u*A_{nt}]$ $\Omega = 2.00$ *(J4-5)*

A_g	7.50 in^2	*Gross area subject to shear*
A_{nv}	5.31 in^2	*Net area subject to shear*
U_{bs}	1.00	*Uniform tension stress factor*
A_{nt}	3.19 in^2	*Net area subject to tension*
F_u	58.00 ksi	*Minimum tensile stress of material*
F_y	36.00 ksi	*Minimum yield stress of material*
R_n/Ω	173.44 kips	*Plate tearout strength*

d_m	16.83 in	*Moment arm between the flange forces*
M_n/Ω	2918.95 kips-in	*Moment = R_n * d_m*

Flange Plate Weld Strength	436.00 kips-in	1502.43 kips-in	0.29	PASS

$Rn/\Omega = 1.5 * 2 * \alpha * 0.928 * C_1 * D_{16} * w_p$

Double Fillet

d_m	16.83 in	*Moment arm. $d_m = d_b + t_p$*
α	0.78	*Base material proration factor*
C_1	1.00	*Electrode strength coefficient*
D_{16}	4.00	*Weld fillet size in sixteenths of an inch*
w_p	10.25 in	*Plate width*
d_b	16.33 in	*Beam overall depth*
t_p	0.50 in	*Plate thickness*
Rn/Ω	89.27 kips	*Weld strength*
Mn/Ω	1502.43 kips-in	*Moment = R_n * d_m*

Flange Plate Tension Yield	436.00 kips-in	1859.36 kips-in	0.23	PASS

$R_n = F_y * A_g$ $\Omega = 1.67$ *(J4-1)*

F_y	36.00 ksi	*Minimum yield stress of material*
A_g	5.13 in^2	*Gross area subject to shear*
R_n/Ω	110.48 kips	*Tensile yield strength*
d_m	16.83 in	*Moment arm between the flange forces*
M_n/Ω	1859.36 kips-in	*Moment = R_n * d_m*

Beam Flange Tension Rupture	436.00 kips-in	2872.68 kips-in	0.15	PASS

$R_n = F_u * A_e$ $\Omega = 2.0$ *(J4-2)*

F_u	65.00 ksi	*Minimum tensile stress of material*
A_e	5.64 in^2	*Effective area subject to tension = $min(A_n, 0.85A_g)$*
A_g	6.81 in^2	*Gross area subject to tension*
A_n	5.64 in^2	*Net area subject to tension*
R_n/Ω	183.38 kips	*Tensile rupture strength*
d_m	15.66 in	*Moment arm between the flange forces*
M_n/Ω	2872.68 kips-in	*Moment = R_n * d_m*

Flange Plate Tension Rupture	436.00 kips-in	2074.30 kips-in	0.21	PASS

$R_n = F_u * A_e$ $\Omega = 2.0$ *(J4-2)*

F_u	58.00 ksi	*Minimum tensile stress of material*
A_e	4.25 in^2	*Effective area subject to tension = $min(A_n, 0.85A_g)$*
A_g	5.13 in^2	*Gross area subject to tension*
	4.25 in^2	*Net area subject to tension*

A_n		
R_n/Ω	123.25 kips	*Tensile rupture strength*
d_m	16.83 in	*Moment arm between the flange forces*
M_n/Ω	2074.30 kips-in	*Moment = $R_n * d_m$*

Flange Plate Compression **PASS**

Check condition : KL/r <= 25.0		*(J4-6)*
K	0.65	*Effective length factor*
L	2.00 in	*Unbraced length*
r	0.14 in	*Radius of gyration*
KL/r	9.01	*Plate slenderness*

Column Flange Bending 436.00 kips-in 771.66 kips-in **0.57** **PASS**

$R_n = 6.25 * t_f^2 * F_y$ $\Omega = 1.67$ *(J10-1)*

d_{end}	36.58 in	*Distance from concentrated force to top of column*
t_f	0.50 in	*Column flange thickness*
F_y	50.00 ksi	*Minimum yield stress of column*
R_n/Ω	45.85 kips	*Column flange local bending*
d_c	16.83 in	*Moment arm from centerlines forces*
M_n/Ω	771.66 kips-in	*Moment = $R_n * d_c$*

Column Web Yielding 436.00 kips-in 946.94 kips-in **0.46** **PASS**

$R_n = (5 * k + N) * F_y * t_w$ $\Omega = 1.50$ *(J10-2)*

d_{end}	36.58 in	*Distance from concentrated force to top of column*
d_{col}	8.12 in	*Column depth*
k	0.89 in	*Distance from outer face of the flange to the web toe of the fillet*
N	1.00 in	*Length of bearing*
F_y	50.00 ksi	*Minimum yield stress of column*
t_w	0.31 in	*Column web thickness*
R_n/Ω	56.26 kips	*Column web local yielding*
d_c	16.83 in	*Moment arm from centerlines forces*
M_n/Ω	946.94 kips-in	*Moment = $R_n * d_c$*

Column Web Buckling 436.00 kips-in 1368.11 kips-in **0.32** **PASS**

$R_n = 24 * t_w^3 * (E * F_y)^{0.5} / h$ $\Omega = 1.67$ *(J10-8)*

d_{end}	36.58 in	*Distance from concentrated force to top of column*
d_{col}	8.12 in	*Column depth*
t_w	0.31 in	*Column web thickness*
F_y	50.00 ksi	*Minimum yield stress of column*
E	29000.00 ksi	*Modulus of elasticity of column*

h	6.34 in	*Clear distance between flanges* $h=d-2*k_{des}$
R_n/Ω	81.29 kips	*Column web compression buckling*
d_c	16.83 in	*Moment arm from centerlines forces*
M_n/Ω	1368.11 kips-in	*Moment* $= R_n * d_c$

Column Web Crippling	436.00 kips-in	1164.65 kips-in	**0.37**	**PASS**

$R_n=0.8*t_w^2*(1+3*(N/d_{col})*(t_w/t_f)^{1.5})*(E*F_y*t_f/t_w)^{0.5}$ $\quad \Omega = 2.00 \quad$ *(J10-4)*

d_{end}	36.58 in	*Distance from concentrated force to top of column*
N/d_{col}	0.12	*Bearing length to column depth ratio*
d_{col}	8.12 in	*Column depth*
t_w	0.31 in	*Column web thickness*
t_f	0.50 in	*Column flange thickness*
N	1.00 in	*Length of bearing*
F_y	50.00 ksi	*Minimum yield stress of column*
E	29000.00 ksi	*Modulus of elasticity of column*
R_n/Ω	69.20 kips	*Column web crippling capacity*
d_c	16.83 in	*Moment arm from centerlines forces*
M_n/Ω	1164.65 kips-in	*Moment* $= R_n * d_c$

Column Panel Zone Shear	436.00 kips-in	761.04 kips-in	**0.57**	**PASS**

$R_n = 0.60 * F_y * d * t_w \quad (P_r<=0.4*P_c)$ $\quad \Omega = 1.67 \quad$ *(J10-9)*

P_r	30.00 kips	*Axial force in the column at the connection*
P_c	309.00 kips	$P_c=0.6*P_y=0.6*F_y*A$
F_y	50.00 ksi	*Minimum yield stress of column*
A	10.30 in^2	*Column cross-sectional area*
d	8.12 in	*Column depth*
t_w	0.31 in	*Column web thickness*
R_n/Ω	45.22 kips	*Web panel zone capacity*
d_c	16.83 in	*Moment arm from centerlines forces*
M_n/Ω	761.04 kips-in	*Moment* $= R_n*d_c$

SHEARWALL CALCS

INCLUDES BOTH WIND AND SEISMIC LOAD AND IN X-DIRECTION

WOOD SHEAR WALL DESIGN (NDS2005)

Using allowable stress design and the segmented shear wall method

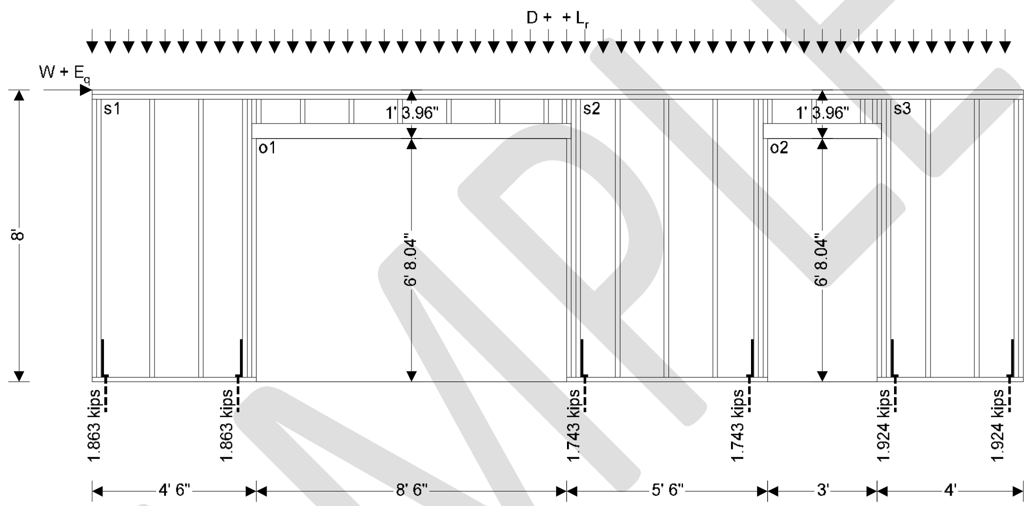
Tedds calculation version 1.0.02

Panel details

Structural wood panel sheathing on one side

Panel height; $h = \mathbf{8}$ ft

Panel length; $b = \mathbf{25.5}$ ft



Panel opening details

Width of opening; $w_{o1} = \mathbf{8.5}$ ft

Height of opening; $h_{o1} = \mathbf{6.67}$ ft

Height to underside of lintel over opening; $l_{o1} = \mathbf{6.67}$ ft

Position of opening; $P_{o1} = \mathbf{4.5}$ ft

Width of opening; $w_{o2} = \mathbf{3}$ ft

Height of opening; $h_{o2} = \mathbf{6.67}$ ft

Height to underside of lintel over opening; $l_{o2} = \mathbf{6.67}$ ft

Position of opening; $P_{o2} = \mathbf{18.5}$ ft

Total area of wall; $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2} = \mathbf{127.295}$ ft^2

Panel construction

Nominal stud size; 2" x 6"

Dressed stud size; 1.5" x 5.5"

Cross-sectional area of studs; $A_s = \mathbf{8.25}$ in^2

Stud spacing; $s = \mathbf{16}$ in

Nominal end post size; 2 x 2" x 6"

Dressed end post size; 2 x 1.5" x 5.5"

Cross-sectional area of end posts;	A_e = **16.5** in^2
Hole diameter;	Dia = **1** in
Net cross-sectional area of end posts;	A_{en} = **13.5** in^2
Nominal collector size;	2 x 2" x 6"
Dressed collector size;	2 x 1.5" x 5.5"
Service condition;	Dry
Temperature;	100 degF or less

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification;	Douglas Fir-Larch, stud grade, 2" & wider
Specific gravity;	G = **0.50**
Tension parallel to grain;	F_t = **450** lb/in^2
Compression parallel to grain;	F_c = **850** lb/in^2
Modulus of elasticity;	E = **1400000** lb/in^2
Minimum modulus of elasticity;	E_{min} = **510000** lb/in^2

Sheathing details

Sheathing material;	**7/16" wood panel sheathing**
Fastener type;	**8d common nails at 4" centers**

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design;	$\mathbf{v_s}$ **=** 760 **lb/ft**
Nominal unit shear capacity for wind design;	$\mathbf{v_w}$ **=** 1065 **lb/ft**
Apparent shear wall shear stiffness;	$\mathbf{G_a}$ **=** 22 **kips/in**

Loading details

Dead load acting on top of panel;	D = **307** lb/ft
Roof live load acting on top of panel;	L_r = **225** lb/ft
Self weight of panel;	S_{wt} = **12** lb/ft^2
In plane wind load acting at head of panel;	W = **3.3** kips
In plane seismic load acting at head of panel;	E_q = **3** kips
Seismic response coefficient;	C_s = **0.2**

From IBC 2009 cl.1605.3.1 Basic load combinations

Load combination no.1;	D + W
Load combination no.2;	D + 0.7E
Load combination no.3;	D + 0.75W + 0.75L_f + 0.75L_r
Load combination no.4;	D + 0.525E + 0.75L_f + 0.75L_r
Load combination no.5;	0.6D + W
Load combination no.6;	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2;	C_D = **1.60**
Size factor for tension – Table 4A;	C_{Ft} = **1.00**
Size factor for compression – Table 4A;	C_{Fc} = **1.00**
Wet service factor for tension – Table 4A;	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A;	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3; C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3; C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8; C_i = **1.00**

Buckling stiffness factor – cl.4.4.2; C_T = **1.00**

Adjusted modulus of elasticity; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **510000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1376** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **1360** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; **$C_P = (1 + (F_{cE} / F_c^*)) / (2$ ´ $c)$ – Ö$([(1 + (F_{cE} / F_c^*)) / (2$ ´ $c)]^2 - (F_{cE} / F_c^*) / c)$**

= 0.70

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Segment 1 shear wall length; b_1 = **4.5** ft

Shear wall aspect ratio; h / b_1 = **1.778**

Segment 2 shear wall length; b_2 = **5.5** ft

Shear wall aspect ratio; h / b_2 = **1.455**

Segment 3 shear wall length; b_3 = **4** ft

Shear wall aspect ratio; h / b_3 = **2**

Segmented shear wall capacity

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + \ + L_r) \times b))$ = **4.213** kips

Shear capacity for seismic loading; $V_s = v_s \times (b_1 + b_2 + b_3) / 2$ = **5.32** kips

V_{s_max} / V_s = **0.792**

PASS - Shear capacity for seismic load exceeds maximum shear force

Maximum shear force under wind loading; $V_{w\_max} = W$ = **3.3** kips

Shear capacity for wind loading; $V_w = v_w \times (b_1 + b_2 + b_3) / 2$ = **7.455** kips

V_{w_max} / V_w = **0.443**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for segment 1

Shear wall aspect ratio; h / b_1 = **1.778**

Shear force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + \ + L_r) \times b))$ = **4.213** kips

Axial force for maximum tension; $P = 0.6 \times (D + S_{wt} \times h)$ = **241.8** lb/ft

Maximum tensile force in chord; $T = V \times h / (b_1 + b_2 + b_3) - P \times b_1 / 2$ = **1.863** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **138** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **720** lb/in^2

f_t / F_t' = **0.192**

PASS - Design tensile stress exceeds maximum applied tensile stress

Shear force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + \ + L_r) \times b))$ = **4.213** kips

Axial force for maximum compression; $P = (D + S_{wt} \times h)$ = **403** lb/ft

Maximum compressive force in chord; $C = V \times h / (b_1 + b_2 + b_3) + P \times b_1 / 2$ = **3.314** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **201** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **945** lb/in^2

f_c / F_c' = **0.213**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for segment 2

Shear wall aspect ratio; h / b_2 = **1.455**

Shear force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + \ + L_r) \times b))$ = **4.213** kips

Axial force for maximum tension; $P = 0.6 \times (D + S_{wt} \times h)$ = **241.8** lb/ft

Maximum tensile force in chord; $T = V \times h / (b_1 + b_2 + b_3) - P \times b_2 / 2$ = **1.743** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **129** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **720** lb/in^2

f_t / F_t' = **0.179**

PASS - Design tensile stress exceeds maximum applied tensile stress

Shear force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + \ + L_r) \times b))$ = **4.213** kips

Axial force for maximum compression; $P = (D + S_{wt} \times h)$ = **403** lb/ft

Maximum compressive force in chord; $C = V \times h / (b_1 + b_2 + b_3) + P \times b_2 / 2$ = **3.516** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **213** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **945** lb/in^2

f_c / F_c' = **0.225**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for segment 3

Shear wall aspect ratio; h / b_3 = **2**

Shear force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + \ + L_r) \times b))$ = **4.213** kips

Axial force for maximum tension; $P = 0.6 \times (D + S_{wt} \times h)$ = **241.8** lb/ft

Maximum tensile force in chord; $T = V \times h / (b_1 + b_2 + b_3) - P \times b_3 / 2$ = **1.924** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **143** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **720** lb/in^2

f_t / F_t' = **0.198**

PASS - Design tensile stress exceeds maximum applied tensile stress

Shear force for maximum compression; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + \ + L_r) \times b))$ = **4.213** kips

Axial force for maximum compression; $P = (D + S_{wt} \times h)$ = **403** lb/ft

Maximum compressive force in chord; $C = V \times h / (b_1 + b_2 + b_3) + P \times b_3 / 2$ = **3.213** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **195** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **945** lb/in^2

f_c / F_c' = **0.206**

PASS - Design compressive stress exceeds maximum applied compressive stress

Collector capacity

Maximum shear force in collector; $V_{max} = \max(V_{s\_max}, V_{w\_max})$ = **4.213** kips

Unit shear above opening; $v_a = V_{max} / b$ = **165.219** lb/ft

Unit shear below opening; $v_b = V_{max} / (b_1 + b_2 + b_3)$ = **300.935** lb/ft

Maximum tensile force in collector; $T = b_1 \times v_b - P_{o1} \times v_a$ = **0.611** kips

Maximum applied tensile stress; $f_t = T / (2 \times A_s)$ = **37** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **720** lb/in^2

f_t / F_t' = **0.051**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum compressive force in collector; $C = (P_{o1} + w_{o1}) \times v_a - b_1 \times v_b =$ **0.794** kips

Maximum applied compressive stress; $f_c = C / (2 \times A_s) =$ **48** lb/in^2

Column stability factor; $C_P =$ **1.00**

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **1360** lb/in^2

$f_c / F_c' =$ **0.035**

PASS - Design compressive stress exceeds maximum applied compressive stress

Deflection

Design shear force; $V = E_q + C_s \times (A \times S_{wt} + (D + \ + L_r) \times b) =$ **6.019** kips

Induced unit shear; $v = V / (b_1 + b_2 + b_3) =$ **429.908** lb/ft

Vertical elongation of wall anchorage; D_a = 0.25 **in**

Shear wall deflection – Eqn. 4.3-1; d_{sw} **= 8 ´ v ´ h^3 / (E ´ A_e ´ ($b_1 + b_2 + b_3$)) + v ´ h / (1000 ´ G_a) + h ´ D_a / ($b_1 + b_2 + b_3$) =** 0.208 **in**

Deflection limit; $\delta_{limit} = h / 240 =$ **0.4** in

$\delta_{sw} / \delta_{limit} =$ **0.521**

PASS - Shear wall deflection is less than deflection limit

INCLUDES BOTH WIND AND SEISMIC LOAD AND IN Y-DIRECTION

WOOD SHEAR WALL DESIGN (NDS2005)

Using allowable stress design and the segmented shear wall method

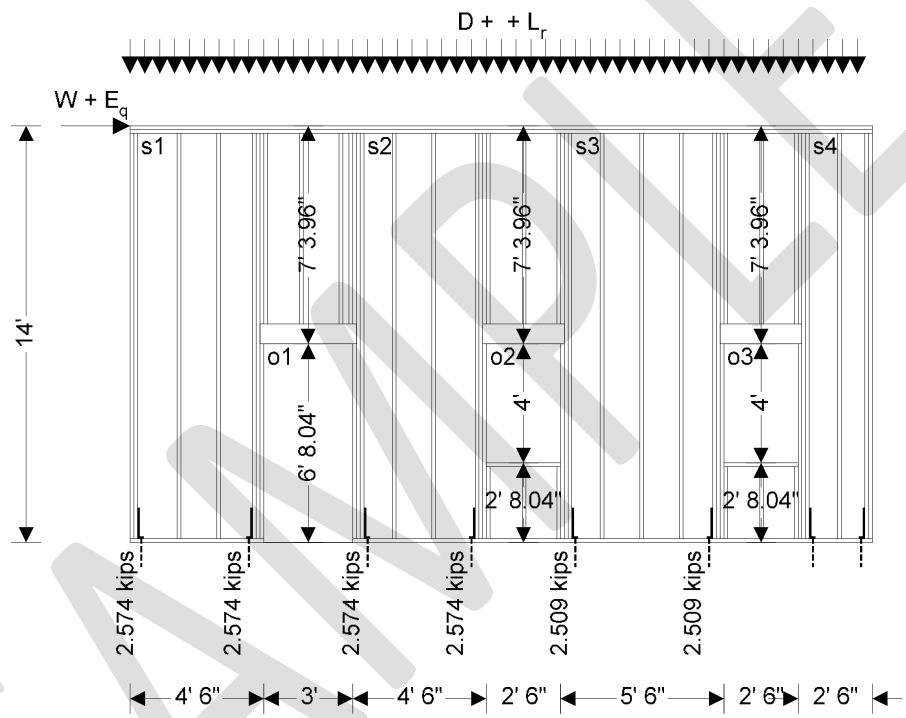
Tedds calculation version 1.0.02

Panel details

Structural wood panel sheathing on one side

Panel height; h = **14** ft

Panel length; b = **25** ft



Panel opening details

Width of opening; w_{o1} = **3** ft

Height of opening; h_{o1} = **6.67** ft

Height to underside of lintel over opening; l_{o1} = **6.67** ft

Position of opening; P_{o1} = **4.5** ft

Width of opening; w_{o2} = **2.5** ft

Height of opening; h_{o2} = **4** ft

Height to underside of lintel over opening; l_{o2} = **6.67** ft

Position of opening; P_{o2} = **12** ft

Width of opening; w_{o3} = **2.5** ft

Height of opening; h_{o3} = **4** ft

Height to underside of lintel over opening; l_{o3} = **6.67** ft

Position of opening; P_{o3} = **20** ft

Total area of wall; $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2} - w_{o3} \times h_{o3} = \mathbf{309.99}\ ft^2$

Panel construction

Nominal stud size; 2" x 6"

Dressed stud size; 1.5" x 5.5"

Cross-sectional area of studs; $A_s = \mathbf{8.25}\ in^2$

Stud spacing; s = **16** in

Nominal end post size; 2 x 2" x 6"

Dressed end post size; 2 x 1.5" x 5.5"

Cross-sectional area of end posts; $A_e = \mathbf{16.5}\ in^2$

Hole diameter; Dia = **1** in

Net cross-sectional area of end posts; $A_{en} = \mathbf{13.5}\ in^2$

Nominal collector size; 2 x 2" x 6"

Dressed collector size; 2 x 1.5" x 5.5"

Service condition; Dry

Temperature; 100 degF or less

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification; Douglas Fir-Larch, stud grade, 2" & wider

Specific gravity; G = **0.50**

Tension parallel to grain; $F_t = \mathbf{450}\ lb/in^2$

Compression parallel to grain; $F_c = \mathbf{850}\ lb/in^2$

Modulus of elasticity; $E = \mathbf{1400000}\ lb/in^2$

Minimum modulus of elasticity; $E_{min} = \mathbf{510000}\ lb/in^2$

Sheathing details

Sheathing material; **7/16" wood panel sheathing**

Fastener type; **8d common nails at 4" centers**

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design; $\mathbf{v_s =}$ 760 **lb/ft**

Nominal unit shear capacity for wind design; $\mathbf{v_w =}$ 1065 **lb/ft**

Apparent shear wall shear stiffness; $\mathbf{G_a =}$ 22 **kips/in**

Loading details

Dead load acting on top of panel; D = **50** lb/ft

Roof live load acting on top of panel; $L_r = \mathbf{50}$ lb/ft

Self weight of panel; $S_{wt} = \mathbf{12}\ lb/ft^2$

In plane wind load acting at head of panel; W = **2.2** kips

In plane seismic load acting at head of panel; $E_q = \mathbf{3}$ kips

Seismic response coefficient; $C_s = \mathbf{0.2}$

From IBC 2009 cl.1605.3.1 Basic load combinations

Load combination no.1; D + W

Load combination no.2; D + 0.7E

Load combination no.3; $D + 0.75W + 0.75L_f + 0.75L_r$

Load combination no.4; $D + 0.525E + 0.75L_f + 0.75L_r$

Load combination no.5; 0.6D + W

Load combination no.6; 0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2; C_D = **1.60**

Size factor for tension – Table 4A; C_{Ft} = **1.00**

Size factor for compression – Table 4A; C_{Fc} = **1.00**

Wet service factor for tension – Table 4A; C_{Mt} = **1.00**

Wet service factor for compression – Table 4A; C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3; C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3; C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8; C_i = **1.00**

Buckling stiffness factor – cl.4.4.2; C_T = **1.00**

Adjusted modulus of elasticity; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **510000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **449** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **1360** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; **$C_P = (1 + (F_{cE} / F_c^*)) / (2 ´ c) – Ö([(1 + (F_{cE} / F_c^*)) / (2 ´ c)]^2 - (F_{cE} / F_c^*) / c)$**

= 0.30

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Segment 1 shear wall length; b_1 = **4.5** ft

Shear wall aspect ratio; h / b_1 = **3.111**

Segment 2 shear wall length; b_2 = **4.5** ft

Shear wall aspect ratio; h / b_2 = **3.111**

Segment 3 shear wall length; b_3 = **5.5** ft

Shear wall aspect ratio; h / b_3 = **2.545**

Segment 4 shear wall length; b_4 = **2.5** ft

Shear wall aspect ratio; h / b_4 = **5.6**

Segmented shear wall capacity

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + \; + L_r) \times b))$ = **2.971** kips

Shear capacity for seismic loading; $V_s = v_s \times (2 \times b_1^2 / h + 2 \times b_2^2 / h + 2 \times b_3^2 / h) / 2$ = **3.841** kips

V_{s_max} / V_s = **0.773**

PASS - Shear capacity for seismic load exceeds maximum shear force

Maximum shear force under wind loading; V_{w_max} = W = **2.2** kips

Shear capacity for wind loading; $V_w = v_w \times (2 \times b_1^2 / h + 2 \times b_2^2 / h + 2 \times b_3^2 / h) / 2$ = **5.382** kips

V_{w_max} / V_w = **0.409**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for segment 1

Shear wall aspect ratio; h / b_1 = **3.111**

Shear force for maximum tension; $V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + \; + L_r) \times b))$ = **2.971** kips

Axial force for maximum tension; $P = 0.6 \times (D + S_{wt} \times h)$ = **130.8** lb/ft

Maximum tensile force in chord;	$T = V \times h / (b_1 + b_2 + b_3) - P \times b_1 / 2 =$ **2.574** kips
Maximum applied tensile stress;	$f_t = T / A_{en} =$ **191** lb/in^2
Design tensile stress;	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **720** lb/in^2
	$f_t / F_t' =$ **0.265**
	PASS - Design tensile stress exceeds maximum applied tensile stress
Shear force for maximum compression;	$V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + \ + L_r) \times b)) =$ **2.971** kips
Axial force for maximum compression;	$P = (D + S_{wt} \times h) =$ **218** lb/ft
Maximum compressive force in chord;	$C = V \times h / (b_1 + b_2 + b_3) + P \times b_1 / 2 =$ **3.359** kips
Maximum applied compressive stress;	$f_c = C / A_e =$ **204** lb/in^2
Design compressive stress;	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **413** lb/in^2
	$f_c / F_c' =$ **0.493**
	PASS - Design compressive stress exceeds maximum applied compressive stress
Chord capacity for segment 2	
Shear wall aspect ratio;	$h / b_2 =$ **3.111**
Shear force for maximum tension;	$V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + \ + L_r) \times b)) =$ **2.971** kips
Axial force for maximum tension;	$P = 0.6 \times (D + S_{wt} \times h) =$ **130.8** lb/ft
Maximum tensile force in chord;	$T = V \times h / (b_1 + b_2 + b_3) - P \times b_2 / 2 =$ **2.574** kips
Maximum applied tensile stress;	$f_t = T / A_{en} =$ **191** lb/in^2
Design tensile stress;	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **720** lb/in^2
	$f_t / F_t' =$ **0.265**
	PASS - Design tensile stress exceeds maximum applied tensile stress
Shear force for maximum compression;	$V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + \ + L_r) \times b)) =$ **2.971** kips
Axial force for maximum compression;	$P = (D + S_{wt} \times h) =$ **218** lb/ft
Maximum compressive force in chord;	$C = V \times h / (b_1 + b_2 + b_3) + P \times b_2 / 2 =$ **3.359** kips
Maximum applied compressive stress;	$f_c = C / A_e =$ **204** lb/in^2
Design compressive stress;	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **413** lb/in^2
	$f_c / F_c' =$ **0.493**
	PASS - Design compressive stress exceeds maximum applied compressive stress
Chord capacity for segment 3	
Shear wall aspect ratio;	$h / b_3 =$ **2.545**
Shear force for maximum tension;	$V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + \ + L_r) \times b)) =$ **2.971** kips
Axial force for maximum tension;	$P = 0.6 \times (D + S_{wt} \times h) =$ **130.8** lb/ft
Maximum tensile force in chord;	$T = V \times h / (b_1 + b_2 + b_3) - P \times b_3 / 2 =$ **2.509** kips
Maximum applied tensile stress;	$f_t = T / A_{en} =$ **186** lb/in^2
Design tensile stress;	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **720** lb/in^2
	$f_t / F_t' =$ **0.258**
	PASS - Design tensile stress exceeds maximum applied tensile stress
Shear force for maximum compression;	$V = 0.7 \times (E_q + C_s \times (A \times S_{wt} + (D + \ + L_r) \times b)) =$ **2.971** kips
Axial force for maximum compression;	$P = (D + S_{wt} \times h) =$ **218** lb/ft
Maximum compressive force in chord;	$C = V \times h / (b_1 + b_2 + b_3) + P \times b_3 / 2 =$ **3.468** kips
Maximum applied compressive stress;	$f_c = C / A_e =$ **210** lb/in^2
Design compressive stress;	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **413** lb/in^2
	$f_c / F_c' =$ **0.509**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for segment 4

Shear wall aspect ratio; h / b_4 = **5.6**

Segment not considered, shear wall aspect ratio exceeds maximum allowable

Collector capacity

Maximum shear force in collector; $V_{max} = max(V_{s\_max}, V_{w\_max})$ = **2.971** kips

Unit shear above opening; $v_a = V_{max} / b$ = **118.831** lb/ft

Unit shear below opening; $v_b = V_{max} / (b_1 + b_2 + b_3)$ = **204.882** lb/ft

Maximum tensile force in collector; $T = (b_1 + b_2 + b_3) \times v_b - P_{o3} \times v_a$ = **0.594** kips

Maximum applied tensile stress; $f_t = T / (2 \times A_s)$ = **36** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **720** lb/in^2

f_t / F_t' = **0.050**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum compressive force in collector; $C = max((P_{o1} + w_{o1}) \times v_a - b_1 \times v_b, 0\ kips)$ = **0** kips

Maximum applied compressive stress; $f_c = C / (2 \times A_s)$ = **0** lb/in^2

Column stability factor; C_P = **1.00**

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **1360** lb/in^2

f_c / F_c' = **0.000**

PASS - Design compressive stress exceeds maximum applied compressive stress

Deflection

Design shear force; $V = E_q + C_s \times (A \times S_{wt} + (D + \ + L_r) \times b)$ = **4.244** kips

Induced unit shear; $v = V / (b_1 + b_2 + b_3)$ = **292.688** lb/ft

Vertical elongation of wall anchorage; D_a **=** 0.25 **in**

Shear wall deflection – Eqn. 4.3-1; **d_{sw} = 8 ´ v ´ h^3 / (E ´ A_e ´ ($b_1 + b_2 + b_3$)) + v ´ h / (1000 ´ G_a) + h ´ D_a / ($b_1 + b_2 + b_3$) =** 0.472 **in**

Deflection limit; $\delta_{limit} = h / 240$ = **0.7** in

$\delta_{sw} / \delta_{limit}$ = **0.674**

PASS - Shear wall deflection is less than deflection limit

ASES A.S. Engineering Services, PLLC 112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259	Project				Job Ref.	
	Section				Sheet no./rev. 38 of 81	
	Calc. by AS	Date 5/15/2012	Chk'd by	Date	App'd by	Date

INDIVIDUAL BEAM ANALYSIS

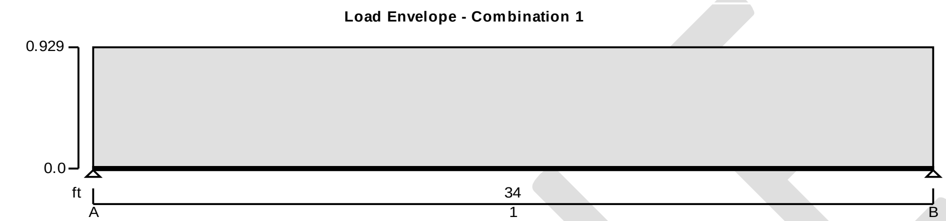
MAIN STEEL BEAM

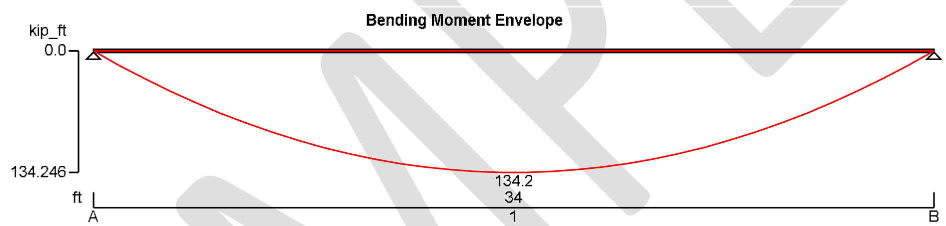
NOTE - DESIGNED WITH GREEN ROOF LOADS

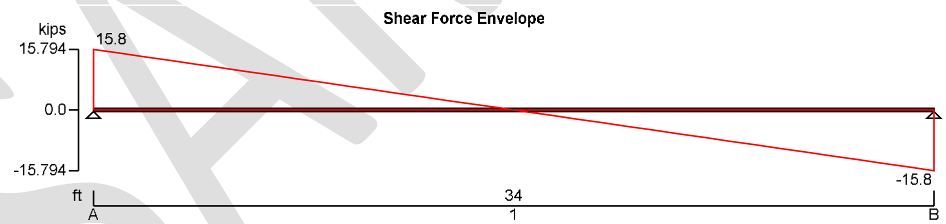
STEEL BEAM ANALYSIS & DESIGN (AISC360-05)

In accordance with AISC360 13th Edition published 2005 using the ASD method

Tedds calculation version 3.0.04







Support conditions

Support A	**Vertically restrained**
	Rotationally free
Support B	**Vertically restrained**
	Rotationally free

Applied loading

Beam loads	**Dead self weight of beam ´ 1**
Span 1 loads	**LOWER ROOF SNOW LOAD WITH DRIFTING - Dead UDL 0.165 kips/ft from 0.00 in to 408.00 in**

LOWER ROOF DEAD LOAD WITH GREEN ROOF - Dead UDL 0.165 kips/ft from 0.00 in to 408.00 in

SNOW LOAD MAIN ROOF RAFTERS - Snow UDL 0.225 kips/ft from 0.00 in to 408.00 in

DEAD LOAD MAIN ROOF RAFTERS w/ GREEN ROOF LOAD - Dead UDL 0.307 kips/ft from 0.00 in to 408.00 in

Load combinations

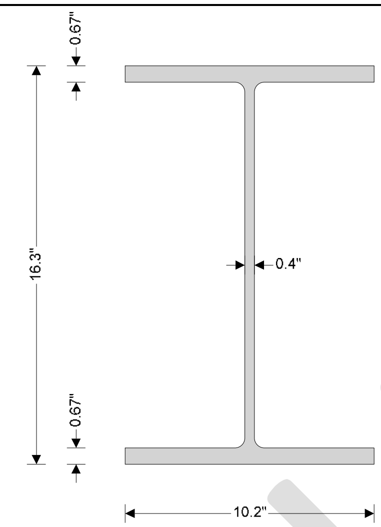
Load combination 1	Support A	Dead × 1.00
		Live × 1.00
		Roof live × 1.00
		Snow × 1.00
	Span 1	Dead × 1.00
		Live × 1.00
		Roof live × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Live × 1.00
		Roof live × 1.00
		Snow × 1.00

Analysis results

Maximum moment;	M_{max} = **134.2** kips_ft;	M_{min} = **0** kips_ft
Maximum shear;	V_{max} = **15.8** kips;	V_{min} = **-15.8** kips
Deflection;	δ_{max} = **1** in;	δ_{min} = **0** in
Maximum reaction at support A;	R_{A_max} = **15.8** kips;	R_{A_min} = **15.8** kips
Unfactored dead load reaction at support A;	$\mathbf{R_{A\_Dead}}$ **=** 12 **kips**	
Unfactored snow load reaction at support A;	$\mathbf{R_{A\_Snow}}$ **=** 3.8 **kips**	
Maximum reaction at support B;	R_{B_max} = **15.8** kips;	R_{B_min} = **15.8** kips
Unfactored dead load reaction at support B;	$\mathbf{R_{B\_Dead}}$ **=** 12 **kips**	
Unfactored snow load reaction at support B;	$\mathbf{R_{B\_Snow}}$ **=** 3.8 **kips**	

Section details

Section type;	**W 16x67 (AISC 13th Edn 2005)**
ASTM steel designation;	**A992**
Steel yield stress;	F_y = **50** ksi
Steel tensile stress;	F_u = **65** ksi
Modulus of elasticity;	E = **29000** ksi



Safety factors

Safety factor for tensile yielding; Ω_{ty} = **1.67**

Safety factor for tensile rupture; Ω_{tr} = **2.00**

Safety factor for compression; Ω_c = **1.67**

Safety factor for flexure; Ω_b = **1.67**

Safety factor for shear; Ω_v = **1.50**

Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local bending - Section B4

Classification of flanges in flexure - Table B4.1 (case 1)

Width to thickness ratio; $b_f / (2 \times t_f)$ = **7.67**

Limiting ratio for compact section; $\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]}$ = **9.15**

Limiting ratio for non-compact section; $\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]}$ = **24.08**; Compact

Classification of web in flexure - Table B4.1 (case 9)

Width to thickness ratio; $(d - 2 \times k) / t_w$ = **35.85**

Limiting ratio for compact section; $\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]}$ = **90.55**

Limiting ratio for non-compact section; $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]}$ = **137.27**; Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength; $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **15.794** kips

Web area; $A_w = d \times t_w$ = **6.439** in^2

Web plate buckling coefficient; k_v = **5**

Web shear coefficient - eq G2-2; C_v = **1.000**

Nominal shear strength - eq G2-1; $V_n = 0.6 \times F_y \times A_w \times C_v$ = **193.155** kips

Allowable shear strength; $V_c = V_n / \Omega_v$ = **128.770** kips

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure in the major axis - Chapter F

Required flexural strength; $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) =$ **134.246** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1; $\mathbf{M_{nyld} = M_p = F_y} \times \mathbf{Z_x =}$ 541.667 **kips_ft**

Nominal flexural strength; $M_n = M_{nyld} =$ **541.667** kips_ft

Allowable flexural strength; $M_c = M_n / \Omega_b =$ **324.351** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live, roof live and snow loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 =$ **1.133** in

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **1.01** in

PASS - Maximum deflection does not exceed deflection limit

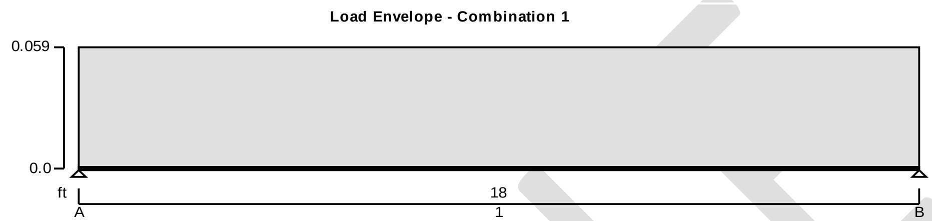
RAFTERS w/ GREEN ROOF

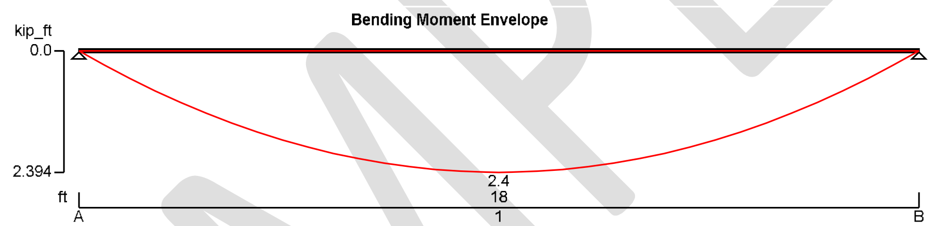
MAXIMUM 20 PSF ADDITIONAL DEAD LOAD FROM GREEN ROOF

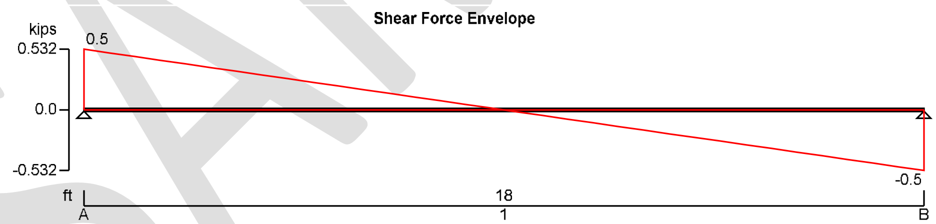
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.04







Applied loading

Beam loads

Dead self weight of beam × 1

Span 1 loads

green roof	Dead UDL 20 lb/ft from 0.00 in to 216.00 in
snow	Snow UDL 25 lb/ft from 0.00 in to 216.00 in
live	Roof Live UDL 20 lb/ft from 0.00 in to 216.00 in
dead	Dead UDL 10 lb/ft from 0.00 in to 216.00 in

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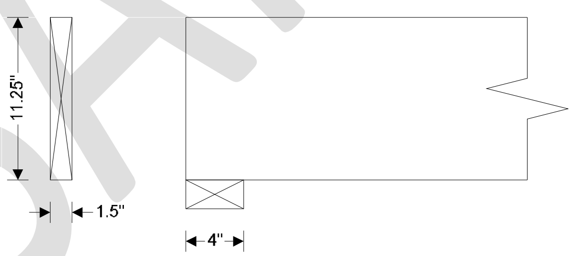
Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00
		Snow × 1.00
		Roof Live × 0.00
	Span 1	Dead × 1.00
		Live × 1.00
		Snow × 1.00
		Roof Live × 0.00
	Support B	Dead × 1.00
		Live × 1.00
		Snow × 1.00
		Roof Live × 0.00

Analysis results

Maximum moment;	M_{max} = **2394** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **2394** lb_ft	
Maximum shear;	F_{max} = **532** lb;	F_{min} = **-532** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **532** lb	
Total load on member;	W_{tot} = **1064** lb	
Reaction at support A;	R_{A_max} = **532** lb;	R_{A_min} = **532** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **307** lb	
Unfactored snow load reaction at support A;	R_{A_Snow} = **225** lb	
Unfactored roof live load reaction at support A;	$R_{A\_Roof\ Live}$ = **180** lb	
Reaction at support B;	R_{B_max} = **532** lb;	R_{B_min} = **532** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **307** lb	
Unfactored snow load reaction at support B;	R_{B_Snow} = **225** lb	
Unfactored roof live load reaction at support B;	$R_{B\_Roof\ Live}$ = **180** 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} = **12** in
Dressed depth of sections;	d = **11.25** in
Number of sections in member;	N = **1**
Overall breadth of member;	b_b = N × b = **1.5** in

Table 4A - Reference design values for visually graded dimension lumber (2"-4" thick)

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

Mean shear modulus; G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition; **Dry**

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

Load duration; **Ten years**

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member; $A = N \times b \times d$ = **16.87** in^2

Section modulus; $S_x = N \times b \times d^2 / 6$ = **31.64** in^3

$S_y = d \times (N \times b)^2 / 6$ = **4.22** in^3

Second moment of area; $I_x = N \times b \times d^3 / 12$ = **177.98** in^4

$I_y = d \times (N \times b)^3 / 12$ = **3.16** in^4

Adjustment factors

Load duration factor - Table 2.3.2; C_D = **1.00**

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

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

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

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 - eq.3.10-2; C_b = **1.00**

Depth-to-breadth ratio; $d_{nom} / (N \times b_{nom})$ = **6.00**

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

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{B\_max} / (N \times b \times L_b)$ = **89** lb/in^2

f_{c_perp} / F_{c_perp}' = **0.142**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r =$ **1035** lb/in^2

Actual bending stress; $f_b = M / S_x =$ **908** lb/in^2

$f_b / F_b' =$ **0.877**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_i =$ **180** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A) =$ **47** lb/in^2

$f_v / F_v' =$ **0.263**

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; $\delta_{adm} = 0.003 \times L_{s1} =$ **0.648** in

Bending deflection; $\delta_{b\_s1} =$ **0.656** in

Shear deflection; $\delta_{v\_s1} =$ **0.027** in

Total deflection; $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **0.683** in

$\delta_a / \delta_{adm} =$ **1.055**

FAIL - Design deflection exceeds total deflection

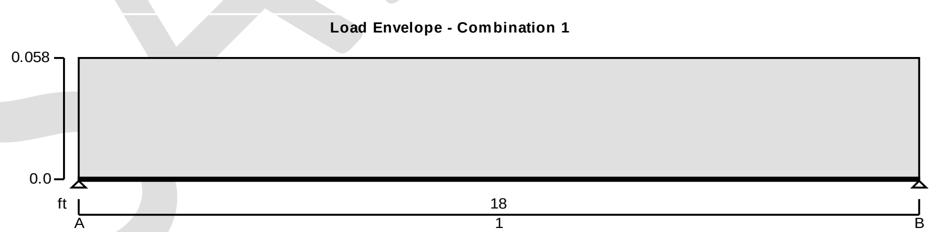
BUT DEFLECTION EXCEEDED BY ONLY 5% THEREFORE OK

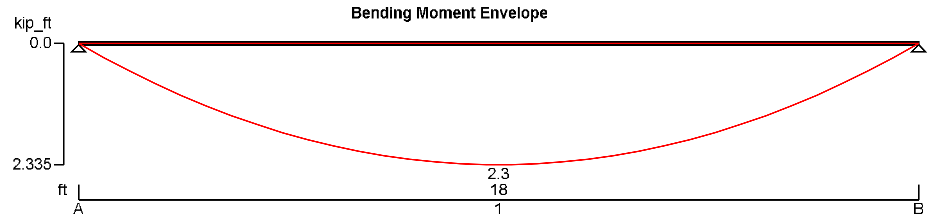
NEXT CHECKED SAME RAFTERS BUT WITH LARGE NOTCH - ONLY FOR SHEAR

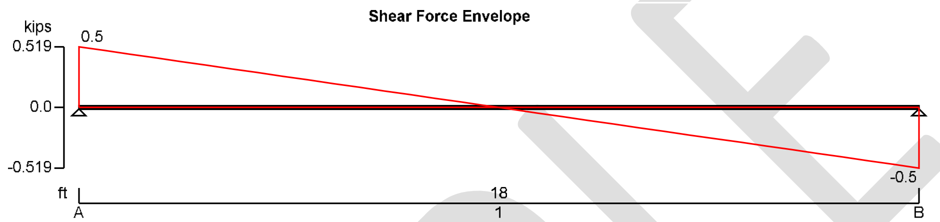
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.04







Applied loading

Beam loads

Dead self weight of beam × 1

Span 1 loads	
green roof	Dead UDL 20 lb/ft from 0.00 in to 216.00 in
snow	Snow UDL 25 lb/ft from 0.00 in to 216.00 in
live	Roof Live UDL 20 lb/ft from 0.00 in to 216.00 in
dead	Dead UDL 10 lb/ft from 0.00 in to 216.00 in

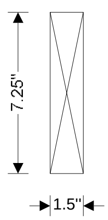
Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00
		Snow × 1.00
		Roof Live × 0.00
	Span 1	Dead × 1.00
		Live × 1.00
		Snow × 1.00
		Roof Live × 0.00
	Support B	Dead × 1.00
		Live × 1.00
		Snow × 1.00
		Roof Live × 0.00

Analysis results

Maximum moment;	M_{max} = **2335** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **2335** lb_ft	
Maximum shear;	F_{max} = **519** lb;	F_{min} = **-519** lb

Design shear;	$F = \max(\text{abs}(F_{max}),\text{abs}(F_{min}))$ = **519** lb	
Total load on member;	W_{tot} = **1038** lb	
Reaction at support A;	R_{A_max} = **519** lb;	R_{A_min} = **519** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **294** lb	
Unfactored snow load reaction at support A;	R_{A_Snow} = **225** lb	
Unfactored roof live load reaction at support A;	$R_{A\_Roof\ Live}$ = **180** lb	
Reaction at support B;	R_{B_max} = **519** lb;	R_{B_min} = **519** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **294** lb	
Unfactored snow load reaction at support B;	R_{B_Snow} = **225** lb	
Unfactored roof live load reaction at support B;	$R_{B\_Roof\ Live}$ = **180** 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} = **8** in
Dressed depth of sections;	d = **7.25** in
Number of sections in member;	N = **1**
Overall breadth of member;	$b_b = N \times b$ = **1.5** in

Table 4A - Reference design values for visually graded dimension lumber (2"-4" thick)

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
Mean shear modulus;	$G_{def} = E / 16$ = **100000** lb/in^2

Member details

Service condition;	**Dry**
Length of bearing;	L_b = **4** in
Load duration;	**Ten years**

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member;	$A = N \times b \times d$ = **10.87** in^2
Section modulus;	$S_x = N \times b \times d^2 / 6$ = **13.14** in^3

	$S_y = d \times (N \times b)^2 / 6 =$ **2.72** in^3
Second moment of area;	$I_x = N \times b \times d^3 / 12 =$ **47.63** in^4
	$I_y = d \times (N \times b)^3 / 12 =$ **2.04** in^4
Adjustment factors	
Load duration factor - Table 2.3.2;	$C_D =$ **1.00**
Temperature factor - Table 2.3.3;	$C_t =$ **1.00**
Size factor for bending - Table 4A;	$C_{Fb} =$ **1.20**
Size factor for tension - Table 4A;	$C_{Ft} =$ **1.20**
Size factor for compression - Table 4A;	$C_{Fc} =$ **1.05**
Flat use factor - Table 4A;	$C_{fu} =$ **1.15**
Incising factor for modulus of elasticity - Table 4.3.8;	$C_{iE} =$ **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8	
	$C_i =$ **1.00**
Incising factor for perpendicular compression - Table 4.3.8	
	$C_{ic\_perp} =$ **1.00**
Repetitive member factor - cl.4.3.9;	$C_r =$ **1.15**
Bearing area factor - eq.3.10-2;	$C_b = (L_b + 0.375 \text{ in}) / L_b =$ **1.09**
Depth-to-breadth ratio;	$d_{nom} / (N \times b_{nom}) =$ **4.00**
- 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;	$F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b =$ **684** 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
	$f_{c\_perp} / F_{c\_perp}' =$ **0.126**

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;	$f_b = M / S_x =$ **2132** lb/in^2
	$f_b / F_b' =$ **1.717**

FAIL - Design bending stress is less than actual bending stress

Strength in shear parallel to grain - cl.3.4.1	
Design shear stress;	$F_v' = F_v \times C_D \times C_t \times C_i =$ **180** lb/in^2
Actual shear stress - eq.3.4-2;	$f_v = 3 \times F / (2 \times A) =$ **72** lb/in^2
	$f_v / F_v' =$ **0.398**

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;	$\delta_{adm} = 0.003 \times L_{s1} =$ **0.648** in
Bending deflection;	$\delta_{b\_s1} =$ **2.406** in
Shear deflection;	$\delta_{v\_s1} =$ **0.042** in
Total deflection;	$\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **2.448** in
	$\delta_a / \delta_{adm} =$ **3.778**

FAIL - Design deflection exceeds total deflection

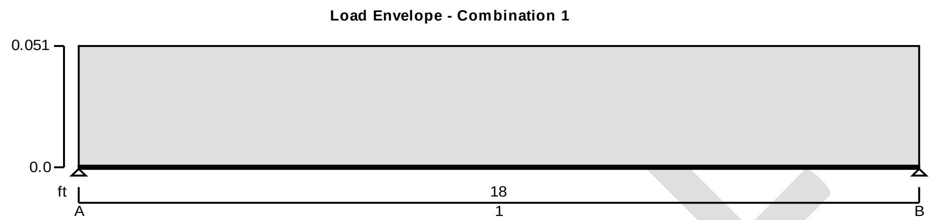
NOTE - PURPOSE OF THIS CALC WAS ONLY TO CHECK SHEAR AT NOTCH OF RAFTER

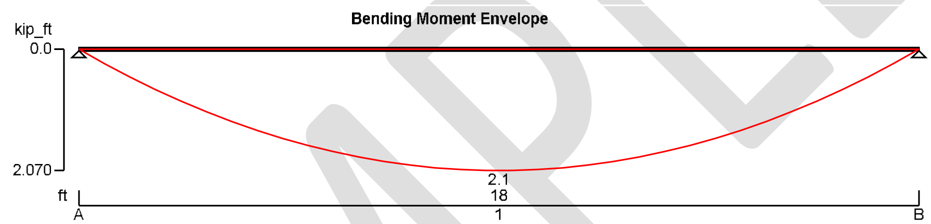
RAFTERS w/o GREEN ROOF

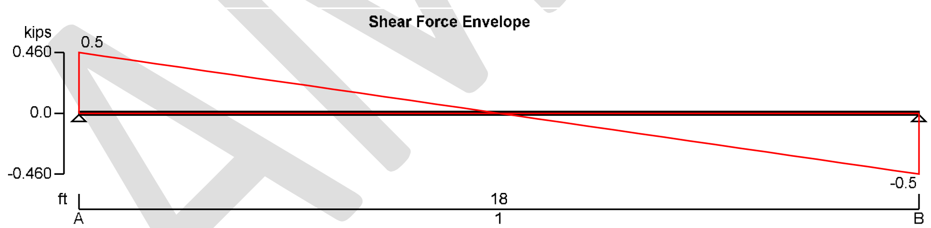
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.04







Applied loading

Beam loads

Dead self weight of beam × 1

Span 1 loads

snow	Snow UDL 34 lb/ft from 0.00 in to 216.00 in
live	Roof Live UDL 26 lb/ft from 0.00 in to 216.00 in
dead	Dead UDL 13 lb/ft from 0.00 in to 216.00 in

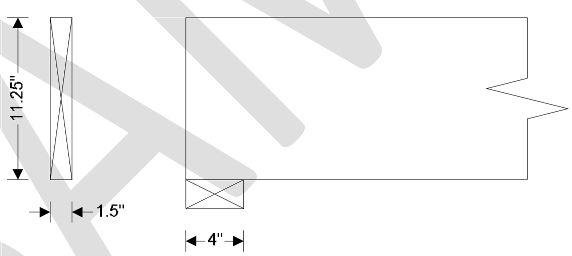
Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00

		Snow × 1.00
		Roof Live × 0.00
	Span 1	Dead × 1.00
		Live × 1.00
		Snow × 1.00
		Roof Live × 0.00
	Support B	Dead × 1.00
		Live × 1.00
		Snow × 1.00
		Roof Live × 0.00

Analysis results

Maximum moment;	M_{max} = **2070** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **2070** lb_ft	
Maximum shear;	F_{max} = **460** lb;	F_{min} = **-460** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **460** lb	
Total load on member;	W_{tot} = **920** lb	
Reaction at support A;	R_{A_max} = **460** lb;	R_{A_min} = **460** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **154** lb	
Unfactored snow load reaction at support A;	R_{A_Snow} = **306** lb	
Unfactored roof live load reaction at support A;	$R_{A\_Roof\ Live}$ = **234** lb	
Reaction at support B;	R_{B_max} = **460** lb;	R_{B_min} = **460** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **154** lb	
Unfactored snow load reaction at support B;	R_{B_Snow} = **306** lb	
Unfactored roof live load reaction at support B;	$R_{B\_Roof\ Live}$ = **234** 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} = **12** in
Dressed depth of sections;	d = **11.25** in
Number of sections in member;	N = **1**
Overall breadth of member;	$b_b = N \times b$ = **1.5** in

Table 4A - Reference design values for visually graded dimension lumber (2"-4" thick)

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
Mean shear modulus; $G_{def} = E / 16$ = **100000** lb/in^2

Member details
Service condition; **Dry**
Length of bearing; L_b = **4** in
Load duration; **Ten years**
The beam is one of three or more repetitive members

Section properties
Cross sectional area of member; $A = N \times b \times d$ = **16.87** in^2
Section modulus; $S_x = N \times b \times d^2 / 6$ = **31.64** in^3
$S_y = d \times (N \times b)^2 / 6$ = **4.22** in^3
Second moment of area; $I_x = N \times b \times d^3 / 12$ = **177.98** in^4
$I_y = d \times (N \times b)^3 / 12$ = **3.16** in^4

Adjustment factors
Load duration factor - Table 2.3.2; C_D = **1.00**
Temperature factor - Table 2.3.3; C_t = **1.00**
Size factor for bending - Table 4A; C_{Fb} = **1.00**
Size factor for tension - Table 4A; C_{Ft} = **1.00**
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 - eq.3.10-2; $C_b = (L_b + 0.375 \text{ in}) / L_b$ = **1.09**
Depth-to-breadth ratio; $d_{nom} / (N \times b_{nom})$ = **6.00**
 - 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; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b$ = **684** lb/in^2
Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b)$ = **77** lb/in^2
f_{c_perp} / F_{c_perp}' = **0.112**
PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1
Design bending stress; $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1035** lb/in^2
Actual bending stress; $f_b = M / S_x$ = **785** lb/in^2
f_b / F_b' = **0.758**

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PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_i =$ **180** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A) =$ **41** lb/in^2

$f_v / F_v' =$ **0.227**

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; $\delta_{adm} = 0.003 \times L_{s1} =$ **0.648** in

Bending deflection; $\delta_{b\_s1} =$ **0.640** in

Shear deflection; $\delta_{v\_s1} =$ **0.027** in

Total deflection; $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **0.666** in

$\delta_a / \delta_{adm} =$ **1.028**

FAIL - Design deflection exceeds total deflection

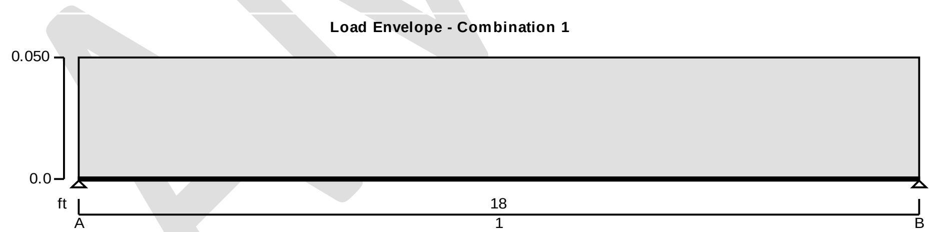
AGAIN - DEFLECTION FAILS BY LESS THAN 3% THEREFORE OK

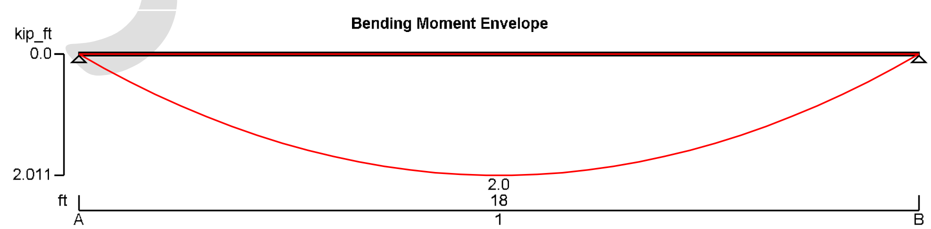
NOW CHECK AGAIN FOR SHEAR DUE TO NOTCH ONLY

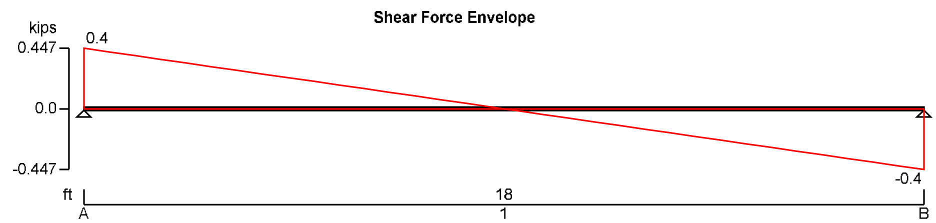
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.04







Applied loading

Beam loads

Dead self weight of beam × 1

Span 1 loads

snow	Snow UDL 34 lb/ft from 0.00 in to 216.00 in
live	Roof Live UDL 26 lb/ft from 0.00 in to 216.00 in
dead	Dead UDL 13 lb/ft from 0.00 in to 216.00 in

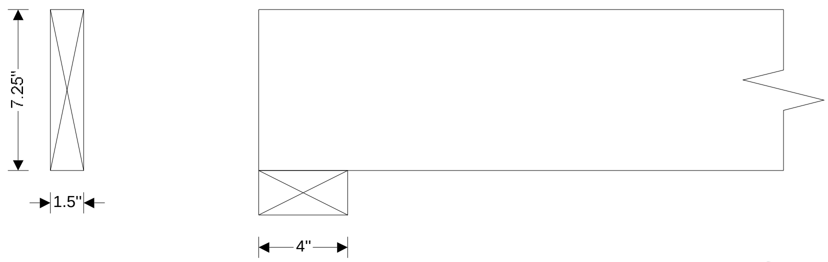
Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00
		Snow × 1.00
		Roof Live × 0.00
	Span 1	Dead × 1.00
		Live × 1.00
		Snow × 1.00
		Roof Live × 0.00
	Support B	Dead × 1.00
		Live × 1.00
		Snow × 1.00
		Roof Live × 0.00

Analysis results

Maximum moment;	M_{max} = **2011** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **2011** lb_ft	
Maximum shear;	F_{max} = **447** lb;	F_{min} = **-447** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **447** lb	
Total load on member;	W_{tot} = **894** lb	
Reaction at support A;	R_{A_max} = **447** lb;	R_{A_min} = **447** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **141** lb	
Unfactored snow load reaction at support A;	R_{A_Snow} = **306** lb	
Unfactored roof live load reaction at support A;	$R_{A\_Roof\ Live}$ = **234** lb	
Reaction at support B;	R_{B_max} = **447** lb;	R_{B_min} = **447** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **141** lb	
Unfactored snow load reaction at support B;	R_{B_Snow} = **306** lb	
Unfactored roof live load reaction at support B;	$R_{B\_Roof\ Live}$ = **234** lb	

ASES 112 Wilson Drive, Port Jefferson, NY 11777 (C) 631-560-0259	Project				Job Ref.	
	Section				Sheet no./rev. 55 of 81	
	Calc. by AS	Date 5/15/2012	Chk'd by	Date	App'd by	Date



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} = **8** in

Dressed depth of sections; d = **7.25** in

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

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

Table 4A - Reference design values for visually graded dimension lumber (2"-4" thick)

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

Mean shear modulus; G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition; **Dry**

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

Load duration; **Ten years**

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member; $A = N \times b \times d =$ **10.87** in^2

Section modulus; $S_x = N \times b \times d^2 / 6 =$ **13.14** in^3

$S_y = d \times (N \times b)^2 / 6 =$ **2.72** in^3

Second moment of area; $I_x = N \times b \times d^3 / 12 =$ **47.63** in^4

$I_y = d \times (N \times b)^3 / 12 =$ **2.04** in^4

Adjustment factors

Load duration factor - Table 2.3.2; C_D = **1.00**

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

Size factor for bending - Table 4A; C_{Fb} = **1.20**

Size factor for tension - Table 4A; C_{Ft} = **1.20**

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

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

Incising factor for modulus of elasticity - Table 4.3.8; C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9; C_r = **1.15**

Bearing area factor - eq.3.10-2; $C_b = (L_b + 0.375\text{ in}) / L_b$ = **1.09**

Depth-to-breadth ratio; $d_{nom} / (N \times b_{nom})$ = **4.00**

- 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; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b$ = **684** lb/in^2

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b)$ = **74** lb/in^2

f_{c_perp} / F_{c_perp}' = **0.109**

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; $f_b = M / S_x$ = **1836** lb/in^2

f_b / F_b' = **1.478**

FAIL - Design bending stress is less than actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_i$ = **180** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A)$ = **62** lb/in^2

f_v / F_v' = **0.342**

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; $\delta_{adm} = 0.003 \times L_{s1}$ = **0.648** in

Bending deflection; δ_{b_s1} = **2.344** in

Shear deflection; δ_{v_s1} = **0.041** in

Total deflection; $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **2.385** in

δ_a / δ_{adm} = **3.680**

FAIL - Design deflection exceeds total deflection

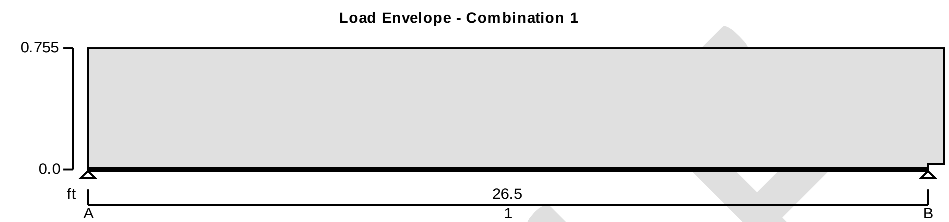
CHECK WAS ONLY FOR SHEAR

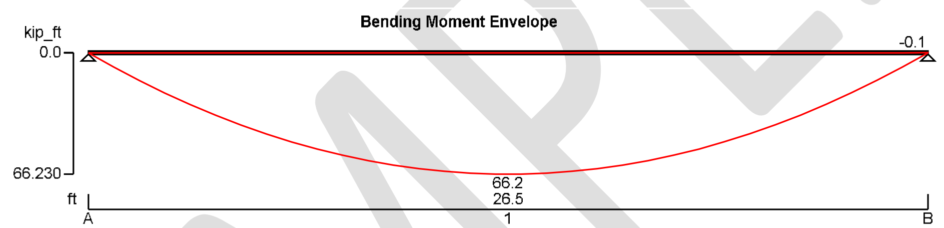
INTERIOR GARAGE BEAM

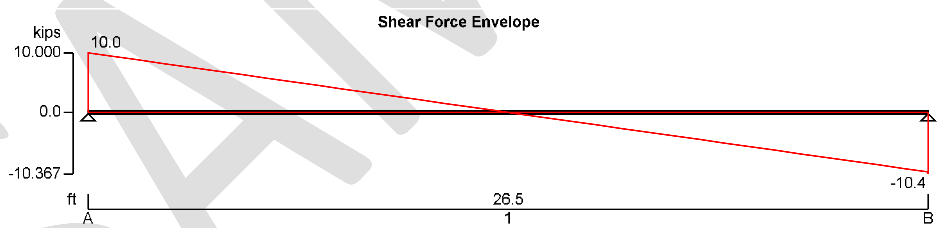
STRUCTURAL COMPOSITE LUMBER BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.04







Applied loading

Beam loads

Dead self weight of beam × 1

Span 1 loads

ROOF — Live UDL 330 lb/ft from 0.00 in to 324.00 in

ROOF — Dead UDL 390 lb/ft from 0.00 in to 324.00 in

Load combinations

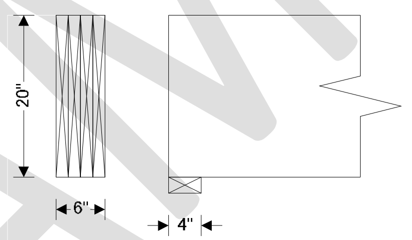
Load combination 1 — Support A — Dead × 1.00

Live × 1.00

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

Analysis results

Maximum moment;	M_{max} = **66230** lb_ft;	M_{min} = **-90** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **66230** lb_ft	
Maximum shear;	F_{max} = **10000** lb;	F_{min} = **-10367** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **10367** lb	
Total load on member;	W_{tot} = **20368** lb	
Reaction at support A;	R_{A_max} = **10000** lb;	R_{A_min} = **10000** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **5629** lb	
Unfactored live load reaction at support A;	R_{A_Live} = **4371** lb	
Reaction at support B;	R_{B_max} = **10367** lb;	R_{B_min} = **10367** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **5828** lb	
Unfactored live load reaction at support B;	R_{B_Live} = **4539** lb	



Composite section details

Breadth of composite section;	b = **1.5** in
Depth of composite section;	d = **20** in
Number of composite sections in member;	N = **4**
Overall breadth of composite member;	b_b = N $\times$ b = **6** in

Reference design values for structural composite lumber

Composite type and grade;	Microllam LVL, 1.9E-2600Fb grade
Bending parallel to grain;	F_b = **2600** lb/in^2
Tension parallel to grain;	F_t = **1555** lb/in^2
Compression parallel to grain;	F_c = **2510** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **750** lb/in^2
Shear parallel to grain;	F_v = **285** lb/in^2

Modulus of elasticity;	$E = \mathbf{1900000}$ lb/in^2
Mean shear modulus;	$G_{def} = E / 16 = \mathbf{118750}$ lb/in^2
Average density;	$\rho = \mathbf{42}$ lb/ft^3

Member details

Service condition;	**Dry**
Length of bearing;	$L_b = \mathbf{4}$ in
Load duration;	**Ten years**

Section properties

Cross sectional area of member;	$A = N \times b \times d = \mathbf{120.00}$ in^2
Section modulus;	$S_x = N \times b \times d^2 / 6 = \mathbf{400.00}$ in^3
	$S_y = d \times (N \times b)^2 / 6 = \mathbf{120.00}$ in^3
Second moment of area;	$I_x = N \times b \times d^3 / 12 = \mathbf{4000.00}$ in^4
	$I_y = d \times (N \times b)^3 / 12 = \mathbf{360.00}$ in^4

Adjustment factors

Load duration factor - Table 2.3.2;	$C_D = \mathbf{1.00}$
Temperature factor - Table 2.3.3;	$C_t = \mathbf{1.00}$
Size factor for bending;	$C_{Fb} = (12\text{ in} / \max(d, 3.5\text{ in}))^{0.136} = \mathbf{0.93}$
Size factor for shear;	$C_{Fv} = (12\text{ in} / d)^{0.136} = \mathbf{0.93}$
Repetitive member factor - cl.8.3.7;	$C_r = \mathbf{1.00}$
Length factor;	$C_{Len} = \mathbf{1.00}$
Bearing area factor - eq.3.10-2;	$C_b = \mathbf{1.00}$
Depth-to-breadth ratio;	$d / (N \times b) = \mathbf{3.33}$
- 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;	$F_{c\_perp}' = F_{c\_perp} \times C_t \times C_b = \mathbf{750}$ lb/in^2
Applied compression stress perpendicular to grain;	$f_{c\_perp} = R_{B\_max} / (N \times b \times L_b) = \mathbf{432}$ lb/in^2
	$f_{c\_perp} / F_{c\_perp}' = \mathbf{0.576}$

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress;	$F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_r = \mathbf{2426}$ lb/in^2
Actual bending stress;	$f_b = M / S_x = \mathbf{1987}$ lb/in^2
	$f_b / F_b' = \mathbf{0.819}$

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress;	$F_v' = F_v \times C_D \times C_t \times C_{Fv} = \mathbf{266}$ lb/in^2
Actual shear stress - eq.3.4-2;	$f_v = 3 \times F / (2 \times A) = \mathbf{130}$ lb/in^2
	$f_v / F_v' = \mathbf{0.487}$

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection;	$E' = E \times C_M \times C_t = \mathbf{1900000}$ lb/in^2
Design deflection;	$\delta_{adm} = 0.004 \times L_{s1} = \mathbf{1.272}$ in

Bending deflection;	δ_{b_s1} = **1.101** in
Shear deflection;	δ_{v_s1} = **0.067** in
Total deflection;	$\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **1.168** in
	δ_a / δ_{adm} = **0.919**

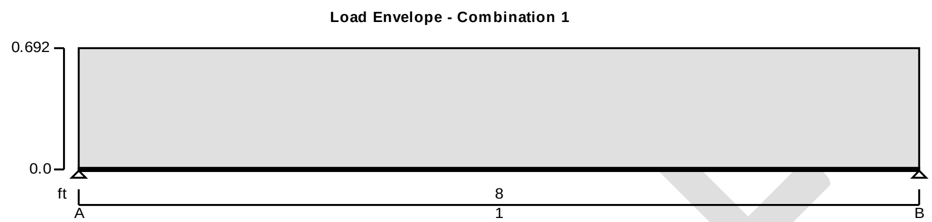
PASS - Design deflection is less than total deflection

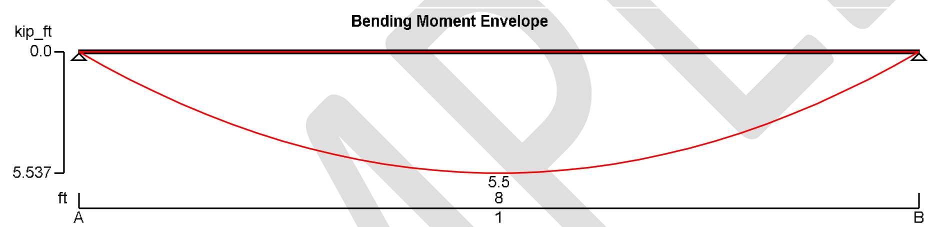
DINING ROOM HEADER

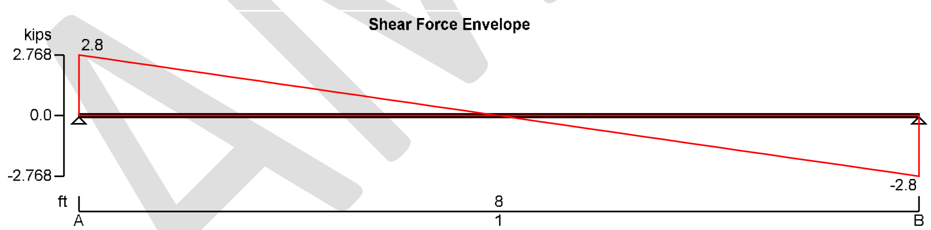
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.04







Applied loading

Beam loads

Dead self weight of beam × 1

Span 1 loads

CEILING LOAD	Dead UDL 150 lb/ft from 0.00 in to 96.00 in
ROOF SNOW LOAD	Live UDL 225 lb/ft from 0.00 in to 96.00 in
ROOF DEAD LOAD WITH GREEN ROOF	Dead UDL 307 lb/ft from 0.00 in to 96.00 in

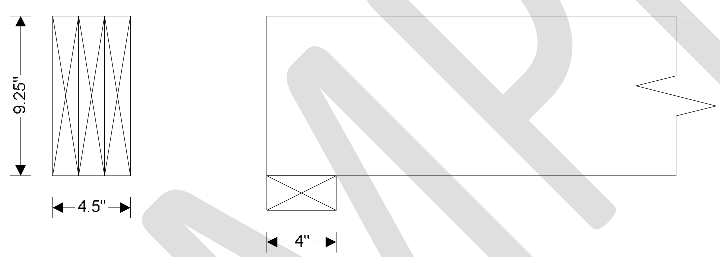
Load combinations

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} = **5537** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **5537** lb_ft	
Maximum shear;	F_{max} = **2768** lb;	F_{min} = **-2768** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **2768** lb	
Total load on member;	W_{tot} = **5537** lb	
Reaction at support A;	R_{A_max} = **2768** lb;	R_{A_min} = **2768** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **1868** lb	
Unfactored live load reaction at support A;	R_{A_Live} = **900** lb	
Reaction at support B;	R_{B_max} = **2768** lb;	R_{B_min} = **2768** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **1868** lb	
Unfactored live load reaction at support B;	R_{B_Live} = **900** lb	



Sawn lumber section details

Nominal breadth of sections;	b_{nom} = **2** in
Dressed breadth of sections;	b = **1.5** in
Nominal depth of sections;	d_{nom} = **10** in
Dressed depth of sections;	d = **9.25** in
Number of sections in member;	N = **3**
Overall breadth of member;	b_b = N × b = **4.5** in

Table 4A - Reference design values for visually graded dimension lumber (2"-4" thick)

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
Mean shear modulus;	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition;	**Dry**

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

Load duration; **Ten years**

Section properties

Cross sectional area of member; $A = N \times b \times d = \mathbf{41.62}$ in^2

Section modulus; $S_x = N \times b \times d^2 / 6 = \mathbf{64.17}$ in^3

$S_y = d \times (N \times b)^2 / 6 = \mathbf{31.22}$ in^3

Second moment of area; $I_x = N \times b \times d^3 / 12 = \mathbf{296.79}$ in^4

$I_y = d \times (N \times b)^3 / 12 = \mathbf{70.24}$ in^4

Adjustment factors

Load duration factor - Table 2.3.2; C_D = **1.00**

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

Size factor for bending - Table 4A; C_{Fb} = **1.10**

Size factor for tension - Table 4A; C_{Ft} = **1.10**

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

Flat use factor - Table 4A; C_{fu} = **1.20**

Incising factor for modulus of elasticity - Table 4.3.8; C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9; C_r = **1.15**

Bearing area factor - eq.3.10-2; 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; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b = \mathbf{625}$ lb/in^2

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{B\_max} / (N \times b \times L_b) = \mathbf{154}$ lb/in^2

f_{c_perp} / F_{c_perp}' = **0.246**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = \mathbf{1139}$ lb/in^2

Actual bending stress; $f_b = M / S_x = \mathbf{1035}$ lb/in^2

f_b / F_b' = **0.909**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_i = \mathbf{180}$ lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A) = \mathbf{100}$ lb/in^2

f_v / F_v' = **0.554**

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

Bending deflection; $\delta_{b\_s1} =$ **0.134** in

Shear deflection; $\delta_{v\_s1} =$ **0.019** in

Total deflection; $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **0.153** in

$\delta_a / \delta_{adm} =$ **0.533**

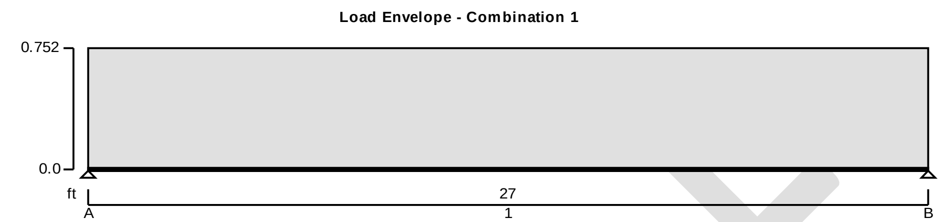
PASS - Design deflection is less than total deflection

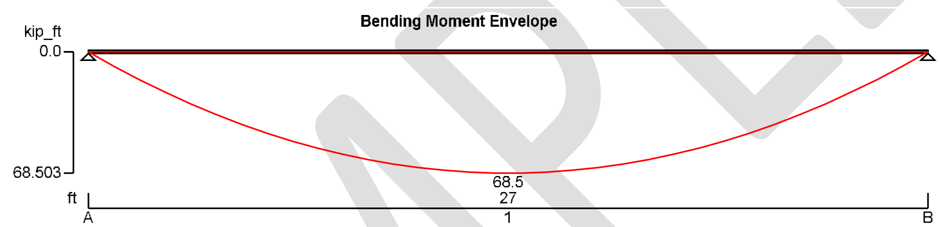
GARAGE DOOR HEADER

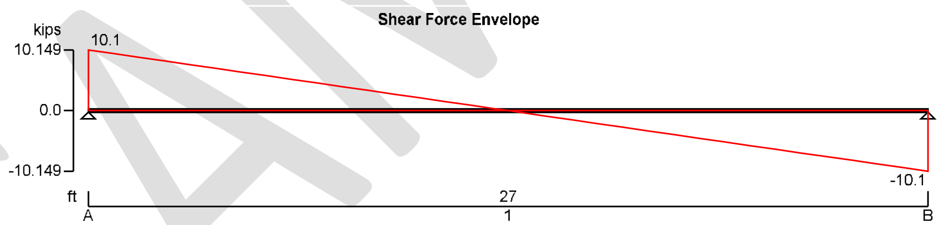
STRUCTURAL COMPOSITE LUMBER BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.04







Applied loading

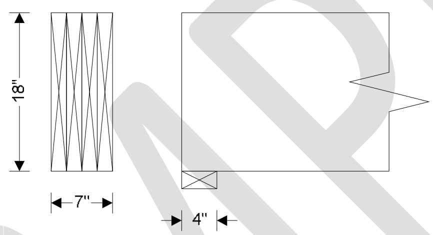
Beam loads

	Dead self weight of beam × 1	
Span 1 loads		
ROOF DEAD - GREEN ROOF	Dead UDL 390 lb/ft from 0.00 in to 324.00 in	
ROOF SNOW	Live UDL 325 lb/ft from 0.00 in to 324.00 in	
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} = **68503** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **68503** lb_ft	
Maximum shear;	F_{max} = **10149** lb;	F_{min} = **-10149** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **10149** lb	
Total load on member;	W_{tot} = **20297** lb	
Reaction at support A;	R_{A_max} = **10149** lb;	R_{A_min} = **10149** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **5761** lb	
Unfactored live load reaction at support A;	R_{A_Live} = **4388** lb	
Reaction at support B;	R_{B_max} = **10149** lb;	R_{B_min} = **10149** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **5761** lb	
Unfactored live load reaction at support B;	R_{B_Live} = **4388** lb	



Composite section details

Breadth of composite section;	b = **1.75** in
Depth of composite section;	d = **18** in
Number of composite sections in member;	N = **4**
Overall breadth of composite member;	b_b = N × b = **7** in

Reference design values for structural composite lumber

Composite type and grade;	Microllam LVL, 1.9E-2600Fb grade
Bending parallel to grain;	F_b = **2600** lb/in^2
Tension parallel to grain;	F_t = **1555** lb/in^2
Compression parallel to grain;	F_c = **2510** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **750** lb/in^2
Shear parallel to grain;	F_v = **285** lb/in^2
Modulus of elasticity;	E = **1900000** lb/in^2
Mean shear modulus;	G_{def} = E / 16 = **118750** lb/in^2
Average density;	ρ = **42** lb/ft^3

Member details

Service condition;	**Dry**
Length of bearing;	L_b = **4** in
Load duration;	**Ten years**

Section properties

Cross sectional area of member; $A = N \times b \times d =$ **126.00** in^2

Section modulus; $S_x = N \times b \times d^2 / 6 =$ **378.00** in^3

$S_y = d \times (N \times b)^2 / 6 =$ **147.00** in^3

Second moment of area; $I_x = N \times b \times d^3 / 12 =$ **3402.00** in^4

$I_y = d \times (N \times b)^3 / 12 =$ **514.50** in^4

Adjustment factors

Load duration factor - Table 2.3.2; $C_D =$ **1.00**

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

Size factor for bending; $C_{Fb} = (12 \text{ in} / \max(d, 3.5 \text{ in}))^{0.136} =$ **0.95**

Size factor for shear; $C_{Fv} = (12 \text{ in} / d)^{0.136} =$ **0.95**

Repetitive member factor - cl.8.3.7; $C_r =$ **1.00**

Length factor; $C_{Len} =$ **1.00**

Bearing area factor - eq.3.10-2; $C_b = (L_b + 0.375 \text{ in}) / L_b =$ **1.09**

Depth-to-breadth ratio; $d / (N \times b) =$ **2.57**

- 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; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_b =$ **820** lb/in^2

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b) =$ **362** lb/in^2

$f_{c\_perp} / F_{c\_perp}' =$ **0.442**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_r =$ **2461** lb/in^2

Actual bending stress; $f_b = M / S_x =$ **2175** lb/in^2

$f_b / F_b' =$ **0.884**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_{Fv} =$ **270** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A) =$ **121** lb/in^2

$f_v / F_v' =$ **0.448**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection; $E' = E \times C_M \times C_t =$ **1900000** lb/in^2

Design deflection; $\delta_{adm} = 0.004 \times L_{s1} =$ **1.296** in

Bending deflection; $\delta_{b\_s1} =$ **1.391** in

Shear deflection; $\delta_{v\_s1} =$ **0.066** in

Total deflection; $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **1.457** in

$\delta_a / \delta_{adm} =$ **1.124**

FAIL - Design deflection exceeds total deflection

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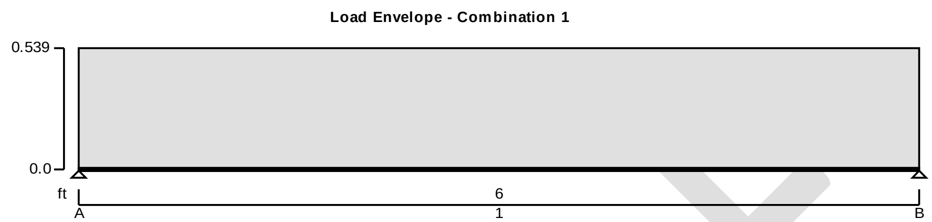
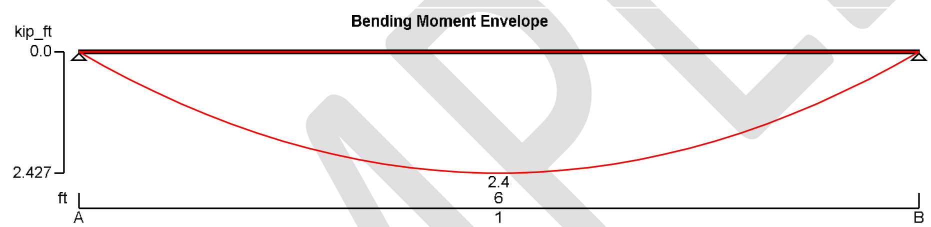
6x6 HEADER

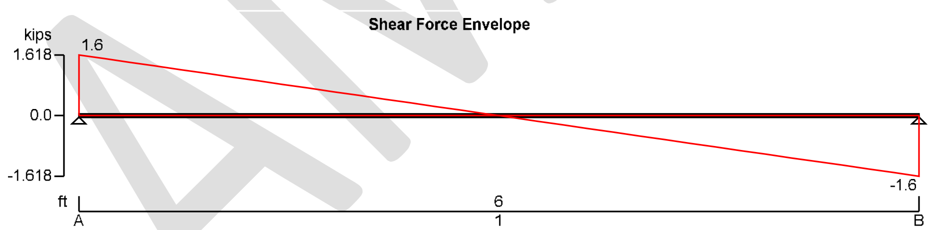
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.04







Applied loading

Beam loads

Dead self weight of beam × 1

Span 1 loads

ROOF SNOW LOAD — Live UDL 225 lb/ft from 0.00 in to 72.00 in

ROOF DEAD LOAD WITH GREEN ROOF — Dead UDL 307 lb/ft from 0.00 in to 72.00 in

Load combinations

Load combination 1 — Support A — Dead × 1.00

Live × 1.00

Span 1 — Dead × 1.00

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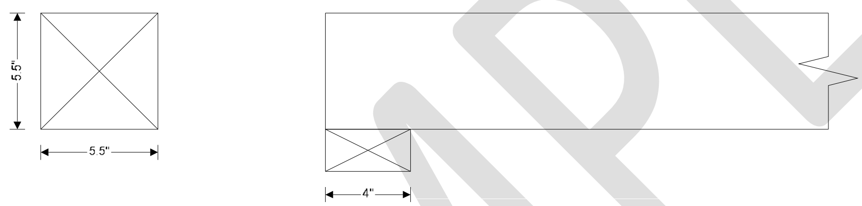
		Live × 1.00
	Support B	Dead × 1.00
		Live × 1.00

Analysis results

Maximum moment;	M_{max} = **2427** lb_ft;	M_{min} = **0** lb_ft
Design moment;	$M = max(abs(M_{max}),abs(M_{min}))$ = **2427** lb_ft	
Maximum shear;	F_{max} = **1618** lb;	F_{min} = **-1618** lb
Design shear;	$F = max(abs(F_{max}),abs(F_{min}))$ = **1618** lb	
Total load on member;	W_{tot} = **3236** lb	
Reaction at support A;	R_{A_max} = **1618** lb;	R_{A_min} = **1618** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **943** lb	
Unfactored live load reaction at support A;	R_{A_Live} = **675** lb	
Reaction at support B;	R_{B_max} = **1618** lb;	R_{B_min} = **1618** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **943** lb	
Unfactored live load reaction at support B;	R_{B_Live} = **675** 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} = **6** in
Dressed depth of sections;	d = **5.5** in
Number of sections in member;	N = **1**
Overall breadth of member;	$b_b = N \times b$ = **5.5** in

Table 4D - Reference design values for visually graded timbers (5"x 5" and larger)

Species, grade and size classification;	Douglas Fir-Larch, No.1 grade, Posts and timbers
Bending parallel to grain;	F_b = **1400** lb/in^2
Tension parallel to grain;	F_t = **925** lb/in^2
Compression parallel to grain;	F_c = **1500** 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 = **1600000** lb/in^2
Mean shear modulus;	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition;	**Dry**
Length of bearing;	L_b = **4** in
Load duration;	**Ten years**

Section properties

Cross sectional area of member; $A = N \times b \times d = \mathbf{30.25}$ in^2

Section modulus; $S_x = N \times b \times d^2 / 6 = \mathbf{27.73}$ in^3

$S_y = d \times (N \times b)^2 / 6 = \mathbf{27.73}$ in^3

Second moment of area; $I_x = N \times b \times d^3 / 12 = \mathbf{76.26}$ in^4

$I_y = d \times (N \times b)^3 / 12 = \mathbf{76.26}$ in^4

Adjustment factors

Load duration factor - Table 2.3.2; $C_D = \mathbf{1.00}$

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

Size factor for bending - Table 4D; $C_{Fb} = \mathbf{1.00}$

Size factor for tension - Table 4D; $C_{Ft} = \mathbf{1.00}$

Size factor for compression - Table 4D; $C_{Fc} = \mathbf{1.00}$

Flat use factor - Table 4D; $C_{fu} = \mathbf{1.00}$

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 - eq.3.10-2; $C_b = (L_b + 0.375 \text{ in}) / L_b = \mathbf{1.09}$

Depth-to-breadth ratio; $d_{nom} / (N \times b_{nom}) = \mathbf{1.00}$

- 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; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b = \mathbf{443}$ lb/in^2

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b) = \mathbf{74}$ lb/in^2

$f_{c\_perp} / F_{c\_perp}' = \mathbf{0.166}$

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = \mathbf{1400}$ lb/in^2

Actual bending stress; $f_b = M / S_x = \mathbf{1050}$ lb/in^2

$f_b / F_b' = \mathbf{0.750}$

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_i = \mathbf{150}$ lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A) = \mathbf{80}$ lb/in^2

$f_v / F_v' = \mathbf{0.535}$

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; $\delta_{adm} = 0.003 \times L_{s1} = \mathbf{0.216}$ in

Bending deflection; $\delta_{b\_s1} = \mathbf{0.129}$ in

Shear deflection; δ_{v_s1} = **0.012** in

Total deflection; $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **0.140** in

δ_a / δ_{adm} = **0.650**

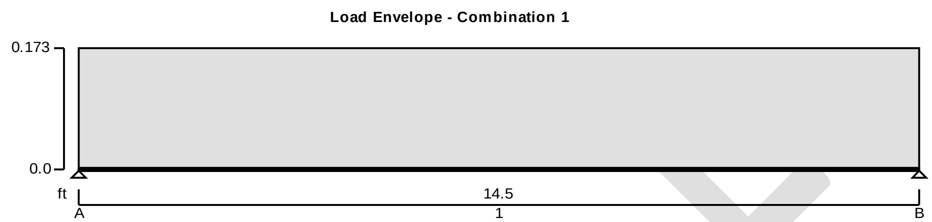
PASS - Design deflection is less than total deflection

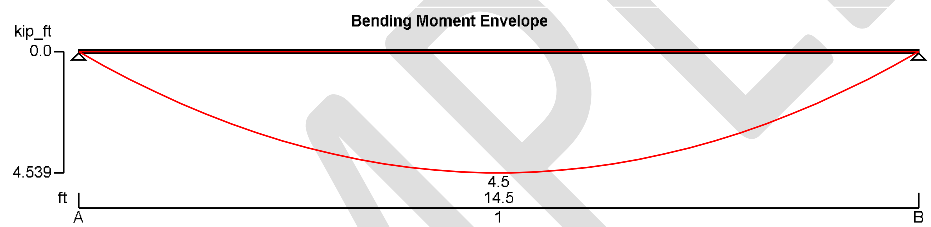
PORCH HEADER

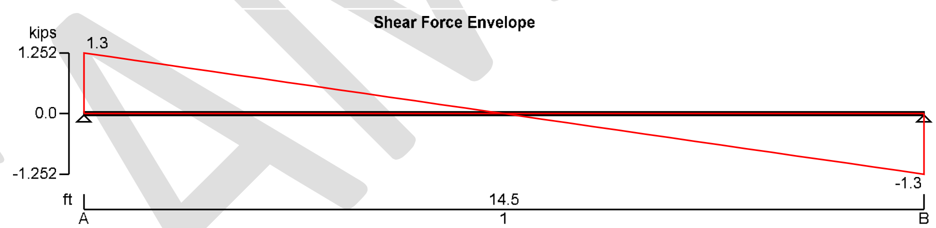
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.04







Applied loading

Beam loads

	Dead self weight of beam × 1

Span 1 loads	
ROOF DEAD - GREEN ROOF	Dead UDL 40 lb/ft from 0.00 in to 174.00 in
ROOF SNOW	Live UDL 120 lb/ft from 0.00 in to 174.00 in

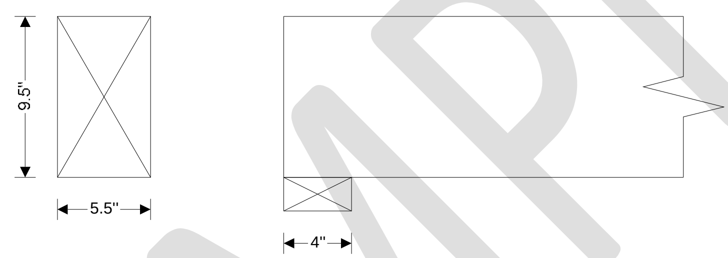
Load combinations

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} = **4539** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **4539** lb_ft	
Maximum shear;	F_{max} = **1252** lb;	F_{min} = **-1252** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **1252** lb	
Total load on member;	W_{tot} = **2504** lb	
Reaction at support A;	R_{A_max} = **1252** lb;	R_{A_min} = **1252** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **382** lb	
Unfactored live load reaction at support A;	R_{A_Live} = **870** lb	
Reaction at support B;	R_{B_max} = **1252** lb;	R_{B_min} = **1252** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **382** lb	
Unfactored live load reaction at support B;	R_{B_Live} = **870** 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

Table 4D - Reference design values for visually graded timbers (5"x 5" and larger)

Species, grade and size classification;	Douglas Fir-Larch, Dense No.2 grade, Beams and stringers
Bending parallel to grain;	F_b = **1000** lb/in^2
Tension parallel to grain;	F_t = **500** lb/in^2
Compression parallel to grain;	F_c = **700** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **730** lb/in^2
Shear parallel to grain;	F_v = **170** lb/in^2
Modulus of elasticity;	E = **1400000** lb/in^2
Mean shear modulus;	G_{def} = E / 16 = **87500** lb/in^2

Member details

Service condition;	**Dry**
Length of bearing;	L_b = **4** in

Load duration; **Ten years**

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; $C_D =$ **1.00**

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 - eq.3.10-2; $C_b = (L_b + 0.375 \text{ in}) / L_b =$ **1.09**

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; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b =$ **798** lb/in^2

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b) =$ **57** lb/in^2

$f_{c\_perp} / F_{c\_perp}' =$ **0.071**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r =$ **1000** lb/in^2

Actual bending stress; $f_b = M / S_x =$ **658** lb/in^2

$f_b / F_b' =$ **0.658**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_i =$ **170** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A) =$ **36** lb/in^2

$f_v / F_v' =$ **0.211**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection; $E' = E \times C_{ME} \times C_t \times C_{iE} =$ **1400000** lb/in^2

Design deflection; $\delta_{adm} = 0.004 \times L_{s1} =$ **0.696** in

Bending deflection; δ_{b_s1} = **0.312** in

Shear deflection; δ_{v_s1} = **0.014** in

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

δ_a / δ_{adm} = **0.469**

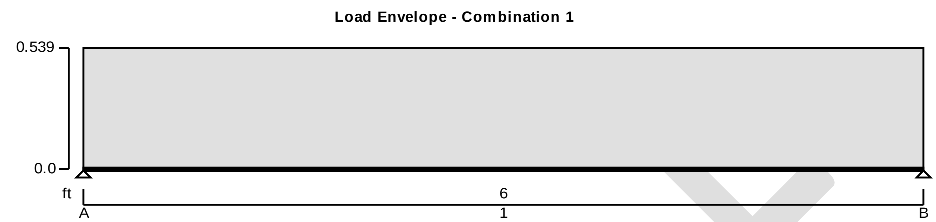
PASS - Design deflection is less than total deflection

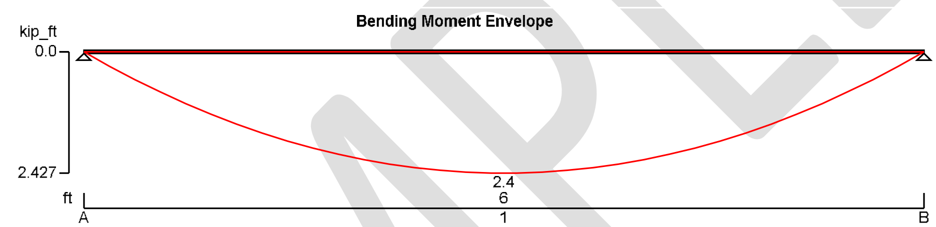
REAR SLIDING DOOR HEADER

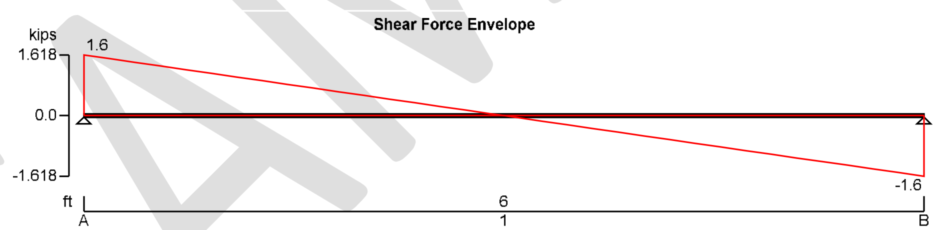
STRUCTURAL WOOD BEAM ANALYSIS & DESIGN (NDS 2005)

In accordance with the ASD method

TEDDS calculation version 1.5.04







Applied loading

Beam loads

Dead self weight of beam × 1

Span 1 loads

ROOF SNOW LOAD	Live UDL 225 lb/ft from 0.00 in to 72.00 in
ROOF DEAD LOAD WITH GREEN ROOF	Dead UDL 307 lb/ft from 0.00 in to 72.00 in

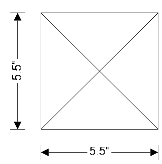
Load combinations

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} = **2427** lb_ft;	M_{min} = **0** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **2427** lb_ft	
Maximum shear;	F_{max} = **1618** lb;	F_{min} = **-1618** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **1618** lb	
Total load on member;	W_{tot} = **3236** lb	
Reaction at support A;	R_{A_max} = **1618** lb;	R_{A_min} = **1618** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **943** lb	
Unfactored live load reaction at support A;	R_{A_Live} = **675** lb	
Reaction at support B;	R_{B_max} = **1618** lb;	R_{B_min} = **1618** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **943** lb	
Unfactored live load reaction at support B;	R_{B_Live} = **675** 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} = **6** in
Dressed depth of sections;	d = **5.5** in
Number of sections in member;	N = **1**
Overall breadth of member;	b_b = N × b = **5.5** in

Table 4D - Reference design values for visually graded timbers (5"x 5" and larger)

Species, grade and size classification;	Douglas Fir-Larch, No.1 grade, Posts and timbers
Bending parallel to grain;	F_b = **1400** lb/in^2
Tension parallel to grain;	F_t = **925** lb/in^2
Compression parallel to grain;	F_c = **1500** 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 = **1600000** lb/in^2
Mean shear modulus;	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition;	**Dry**
Length of bearing;	L_b = **4** in
Load duration;	**Ten years**

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Section properties

Cross sectional area of member; $A = N \times b \times d = \mathbf{30.25}\ in^2$

Section modulus; $S_x = N \times b \times d^2 / 6 = \mathbf{27.73}\ in^3$

$S_y = d \times (N \times b)^2 / 6 = \mathbf{27.73}\ in^3$

Second moment of area; $I_x = N \times b \times d^3 / 12 = \mathbf{76.26}\ in^4$

$I_y = d \times (N \times b)^3 / 12 = \mathbf{76.26}\ in^4$

Adjustment factors

Load duration factor - Table 2.3.2; $C_D = \mathbf{1.00}$

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

Size factor for bending - Table 4D; $C_{Fb} = \mathbf{1.00}$

Size factor for tension - Table 4D; $C_{Ft} = \mathbf{1.00}$

Size factor for compression - Table 4D; $C_{Fc} = \mathbf{1.00}$

Flat use factor - Table 4D; $C_{fu} = \mathbf{1.00}$

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 - eq.3.10-2; $C_b = (L_b + 0.375\ in) / L_b = \mathbf{1.09}$

Depth-to-breadth ratio; $d_{nom} / (N \times b_{nom}) = \mathbf{1.00}$

- 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; $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_i \times C_b = \mathbf{443}\ lb/in^2$

Applied compression stress perpendicular to grain; $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b) = \mathbf{74}\ lb/in^2$

$f_{c\_perp} / F_{c\_perp}' = \mathbf{0.166}$

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = \mathbf{1400}\ lb/in^2$

Actual bending stress; $f_b = M / S_x = \mathbf{1050}\ lb/in^2$

$f_b / F_b' = \mathbf{0.750}$

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t \times C_i = \mathbf{150}\ lb/in^2$

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A) = \mathbf{80}\ lb/in^2$

$f_v / F_v' = \mathbf{0.535}$

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; $\delta_{adm} = 0.003 \times L_{s1} = \mathbf{0.216}\ in$

Bending deflection; $\delta_{b\_s1} = \mathbf{0.129}\ in$

Shear deflection; δ_{v_s1} = **0.012** in

Total deflection; $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **0.140** in

δ_a / δ_{adm} = **0.650**

PASS - Design deflection is less than total deflection

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FOOTING DESIGN

FOOTING SIMPLY TOOK REATIONS FOR BEAMS AND DIVIDED BY ALLOWABLE SOIL BEARING PRESSURE - NO OVERTURNING IS APPLIED TO ANY FOOTINGS AND UPLIFT IS MINIMAL