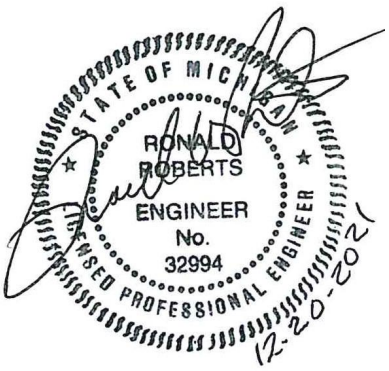


Ronald Roberts Associates, Inc.
Consulting Engineers

STRUCTURAL DESIGN CALCULATIONS

PROJECT: **SCOOTERS COFFEE**

LOCATION: **1490 WEST ANN ARBOR ROAD**
PLYMOUTH, MI 48170

BUILDING CODE: **2015 MICHIGAN BUILDING CODE**
(2015 INTERNATIONAL BUILDING CODE)

DESIGN LOADS:

115 MPH	**DESIGN WIND SPEED (ULTIMATE)**
20 PSF	**ROOF DEAD LOAD**
20 PSF	**ROOF LIVE LOAD**
20 PSF	**GROUND SNOW LOAD**

DATE: **DECEMBER 20, 2021**

2948 N. Stemmons Freeway ▪ Dallas, TX 75247-6103 ▪ Phone: 214-637-6299 ▪ rara@rara.net

Ronald Roberts Associates, Inc. Consulting Engineers

PROJECT: SCOOTERS COFFEE JOB NO.: 21308

SUBJECT: DESIGN LOADS DATE: 12/20/2021

BY: CMB

ALL DESIGN LOADS PER THE 2015 INTERNATIONAL BUILDING CODE AND ASCE 7-10.

- ROOF LIVE LOAD (TABLE 4.3-1)
  - ORDINARY FLAT, PITCHED, AND CURVED ROOFS: **20 PSF**

- ROOF DEAD LOAD

ROOFING (BITUMINOUS):	1.5	PSF
ROOF DECK (3/4" PLYWOOD):	2.4	PSF
INSULATION (3" RIGID):	4.5	PSF
WOOD JOIST:	4.0	PSF
HVAC ALLOWANCE:	4.0	PSF
CEILING SYSTEM, LIGHTING, ETC.:	3.6	PSF
∑=	**20**	**PSF**

- GROUND SNOW LOAD, P_g
  - **P_g = 20 PSF**

- MAIN WIND-FORCE RESISTING SYSTEM (SIMPLE DIAPHRAGM, LOW-RISE BUILDING) PER TABLE 28.5-1 OF ASCE 7-10

STEP 1: DETERMINE RISK CATEGORY OF BUILDING OR OTHER STRUCTURE

RISK CATEGORY II (TABLE 1.5-1)

STEP 2: DETERMINE THE BASIC WIND SPEED, V, FOR APPLICABLE RISK CATEGORY

V_{ULT} = 115 MPH, $V_{ASD} = \sqrt{0.6}V_{ULT} = \sqrt{0.6}(115) =$ 89 MPH (FIGURE 26.5-1A)

STEP 3: DETERMINE WIND LOAD PARAMETERS

EXPOSURE CATEGORY B (SECTION 26.7)

K_{zt} = 1.0 (SECTION 26.8)

STEP 4: ENTER FIGURE TO DETERMINE WIND PRESSURES FOR h=30 FT, P_{S30} (FIGURE 28.6-1) (ROOF ANGLE 0 TO 5°)

P_{S30} =	+21.0	PSF	ZONE A
=	-10.9	PSF	ZONE B
=	+13.9	PSF	ZONE C
=	-6.5	PSF	ZONE D
=	-25.2	PSF	ZONE E
=	-14.3	PSF	ZONE F
=	-17.5	PSF	ZONE G
=	-11.1	PSF	ZONE H
=	-35.3	PSF	OVERHANG, ZONE E
=	-27.6	PSF	OVERHANG, ZONE G

STEP 5: ENTER FIGURE TO DETERMINE ADJUSTMENT FOR BUILDING HEIGHT AND EXPOSURE, λ

λ = 1.00 (H = 13.6 FT, EXPOSURE B) – FIGURE 28.6-1

STEP 6: DETERMINE ADJUSTED WIND PRESSURES, P_S

$P_S = \lambda K_{zt} P_{s30}$ (EQUATION 28.6-1)

P_{S30} =	+21.0	PSF	ZONE A
=	-10.9	PSF	ZONE B
=	+13.9	PSF	ZONE C
=	-6.5	PSF	ZONE D
=	-25.2	PSF	ZONE E
=	-14.3	PSF	ZONE F
=	-17.5	PSF	ZONE G
=	-11.1	PSF	ZONE H
=	-35.3	PSF	OVERHANG, ZONE E
=	-27.6	PSF	OVERHANG, ZONE G

PER SECTION 28.6.4 OF ASCE 7-10, THE LOAD EFFECTS OF THE DESIGN WIND PRESSURES FROM SECTION 28.6.3 SHALL NOT BE LESS THAN A MINIMUM LOAD DEFINED BY ASSUMING THE PRESSURES, P_S, FOR ZONES A AND C EQUAL TO +16 PSF, ZONES B AND D EQUAL TO +8 PSF, WHILE ASSUMING P_S FOR ZONES E, F, G, AND H ARE EQUAL TO 0 PSF.

MINIMUM WIND PRESSURES, P_S

$P_S = \lambda K_{zt} P_{s30}$ (EQUATION 28.6-1)

P_{S30} =	+16.0	PSF	ZONE A
=	+8.0	PSF	ZONE B
=	+16.0	PSF	ZONE C
=	+8.0	PSF	ZONE D
=	0	PSF	ZONE E
=	0	PSF	ZONE F
=	0	PSF	ZONE G
=	0	PSF	ZONE H

- COMPONENTS & CLADDING (SIMPLIFIED METHOD) PER TABLE 30.5-1 OF ASCE 7-10

STEP 1: DETERMINE RISK CATEGORY OF BUILDING OR OTHER STRUCTURE

RISK CATEGORY II (TABLE 1.5-1)

STEP 2: DETERMINE THE BASIC WIND SPEED, V, FOR APPLICABLE RISK CATEGORY

V_{ULT} = 115 MPH, $V_{ASD} = \sqrt{0.6}V_{ULT} = \sqrt{0.6}(115) = 89$ MPH (FIGURE 26.5-1A)

STEP 3: DETERMINE WIND LOAD PARAMETERS

EXPOSURE CATEGORY B (SECTION 26.7)

K_{zt} = 1.0 (SECTION 26.8)

STEP 4: ENTER FIGURE TO DETERMINE WIND PRESSURES AT h=30 FT, P_{NET30} (FIGURE 30.5-1) (ROOF ANGLE 0 TO 7°)

P_{NET30} = +7.7 PSF ZONE 1, EFFECTIVE AREA = 100 FT^2
-21.8 PSF

= +7.7 PSF ZONE 2, EFFECTIVE AREA = 100 FT^2
-25.8 PSF

= +7.7 PSF ZONE 3, EFFECTIVE AREA = 100 FT^2
-25.8 PSF

= +22.7 PSF WALL, ZONE 4, EFFECTIVE AREA = 20 FT^2
-24.7 PSF

= +22.7 PSF WALL, ZONE 5, EFFECTIVE AREA = 20 FT^2
-29.7 PSF

STEP 5: ENTER FIGURE TO DETERMINE ADJUSTMENT FOR BUILDING HEIGHT AND EXPOSURE, λ

λ = 1.00 (H = 13.6 FT, EXPOSURE B) – FIGURE 30.5-1

STEP 6: DETERMINE ADJUSTED WIND PRESSURES, P_{NET}

$P_{NET} = \lambda K_{zt} P_{NET30}$ (EQUATION 30.4-1)

P_{NET} = +7.7 PSF ZONE 1, EFFECTIVE AREA = 100 FT^2
-21.8 PSF

= +7.7 PSF ZONE 2, EFFECTIVE AREA = 100 FT^2
-25.8 PSF

= +7.7 PSF ZONE 3, EFFECTIVE AREA = 100 FT^2
-25.8 PSF

= +22.7 PSF WALL, ZONE 4, EFFECTIVE AREA = 20 FT^2
-24.7 PSF

= +22.7 PSF WALL, ZONE 5, EFFECTIVE AREA = 20 FT^2
-29.7 PSF

PER SECTION 30.2.2 OF ASCE 7-10, THE DESIGN WIND PRESSURE FOR COMPONENTS AND CLADDING OF BUILDINGS SHALL NOT BE LESS THAN A NET PRESSURE OF 16 PSF ACTING IN EITHER DIRECTION NORMAL TO THE SURFACE.

- MAIN WIND-FORCE RESISTING SYSTEM (PARAPETS) PER SECTION 28.4.2 OF ASCE 7-10

$P_P = q_P (GC_{PN})$ (EQUATION 28.4-2)

$q_P = 0.00256K_ZK_{Zt}K_dV^2$ (EQUATION 28.3-1)

$K_Z =$ 0.70 (H =19 FT, EXPOSURE B, TABLE 28.3-1)
$K_{Zt} =$ 1.0 (SECTION 26.8.2)
$K_d =$ 0.85 (TABLE 26.6-1)
$V_{ULT} =$ 115 MPH (FIGURE 26.5-1A)

$q_P = 0.00256(0.70)(1.0)(0.85)(115)^2 = 20.2$ PSF

$P_P =$ 20.2 PSF (+1.5) = +30.3 PSF (WINDWARD PARAPET)
$P_P =$ 20.2 PSF (-1.0) = -20.2 PSF (LEEWARD PARAPET)

- COMPONENTS & CLADDING (PARAPETS) PER TABLE 30.9-1 OF ASCE 7-10

STEP 1: DETERMINE RISK CATEGORY OF BUILDING OR OTHER STRUCTURE

RISK CATEGORY II (TABLE 1.5-1)

STEP 2: DETERMINE THE BASIC WIND SPEED, V, FOR APPLICABLE RISK CATEGORY

V_{ULT} = 115 MPH, $V_{ASD} = \sqrt{0.6}V_{ULT} = \sqrt{0.6}(115) = 89$ MPH (FIGURE 26.5-1A)

STEP 3: DETERMINE WIND LOAD PARAMETERS

K_d = 0.85 (TABLE 26.6-1)

EXPOSURE CATEGORY B (SECTION 26.7)
K_{zt} = 1.0 (SECTION 26.8)

ENCLOSED BUILDING (SECTION 26.10)

INTERNAL PRESSURE COEFFICIENT, GC_{pi} = ±0.18 (TABLE 26.11-1)

STEP 4: DETERMINE VELOCITY PRESSURE EXPOSURE COEFFICIENT

K_h = 0.70 (H = 19 FT, EXPOSURE B, TABLE 30.3-1)

STEP 5: DETERMINE VELOCITY PRESSURE q_p

$$q_P = 0.00256K_hK_{Zt}K_dV^2 \text{ (EQUATION 30.3-1)}$$

$$q_P = 0.00256(0.70)(1.0)(0.85)(115)^2 = 20.2 \text{ PSF}$$

STEP 6: DETERMINE EXTERNAL PRESSURE COEFFICIENT FOR WALL AND ROOF SURFACES ADJACENT TO PARAPET, (GC_P)

GC_P = +0.9 (WALL, ZONE 5, EFFECTIVE AREA = 20 FT^2)
= -1.3 (WALL, ZONE 5, EFFECTIVE AREA = 20 FT^2)

= -1.1 (ROOF, ZONE 3, EFFECTIVE AREA = 100 FT^2)

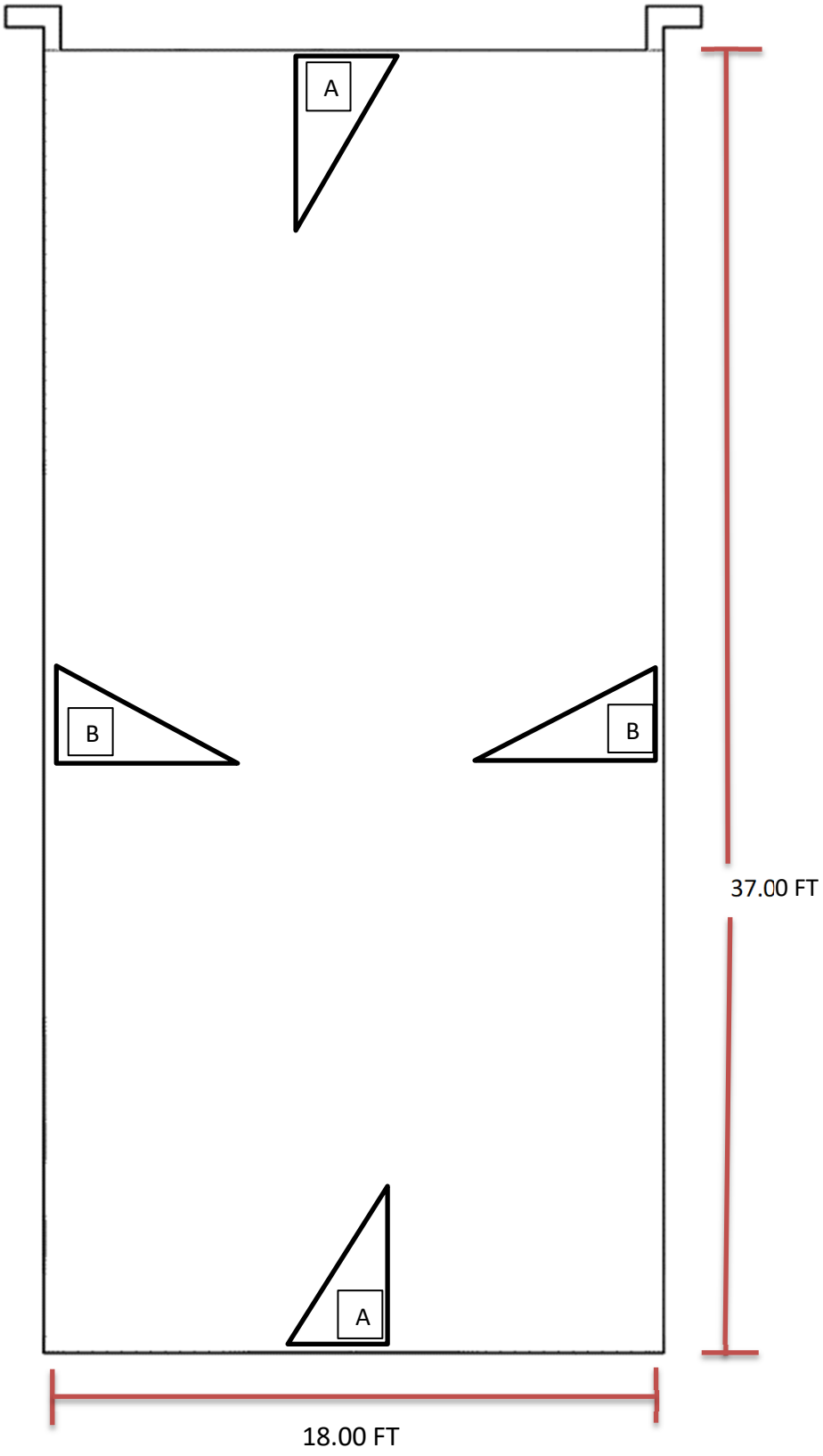
STEP 7: CALCULATE WIND PRESSURE, P

LOAD CASE A: +0.9-(-1.1) = +2.0
$P = q_P(GC_P - GC_{Pi})$ (EQUATION 30.9-1)
P = 20.2 PSF(+2.0 + 0.18)
P = 44.0 PSF

LOAD CASE B: +0.9-(-1.3) = +2.2
P = 20.2 PSF(+2.2 + 0.18)
P = 48.1 PSF

B.O. DECK: 12.85 FT
T. O. PARAPET: 18.25 FT

PARAPET HEIGHT: 5.40 FT



B.O. DECK: 13.60 FT
T. O. PARAPET: 19.00 FT

PARAPET HEIGHT: 5.40 FT

GROUND SNOW LOAD (P_g) : 20.0 PSF (ASCE 7-10: 7.2)

FLAT ROOF SNOW LOAD (ASCE 7-10: 7.3)

$P_f = .7C_eC_tI_sP_g$ Equation 7.3-1

C_e 1.0
C_t = 1.0
I_s = 1.0
P_g = 20.0 PSF

P_f = 14.0 PSF

MINIMUM SNOW LOAD FOR LOW- SLOPE ROOFS (ASCE7-10: 7.3.4)

$P_m = 20 * I_s$ for Ground Snow > 20
$P_m = I_sP_g$ for Ground Snow < 20

I_s = 1.0
P_g = 20.0 PSF

P_m = 20.0 PSF

SNOW DENSITY

$\gamma = .013P_g+14$ but no more than 30 pcf (ASCE 7-10 : 7.7-1)

P_g = 20.0 PSF

γ= 14.26 PCF

BASE SNOW HEIGHT

$h_b = {}^{P_f}/_{\gamma}$

P_f = 20.0 PSF
γ= 14.26 PCF

h_b = 1.40 FT

SNOW DRIFT A

FULL DRIFT

$$h_d = .43\sqrt[3]{l_u}\sqrt[4]{P_g + 10} - 1.5$$ (ASCE 7-10 : FIG. 7-9)

l_u = 37.00 FT if less than 20 FT use 20 FT

P_g = 20.0 PSF

h_d = 1.85 FT

LESS THAN PARAPET HEIGHT: 5.40 FT
THEREFORE FULL DRIFT POSSIBLE TRUE

MAXIMUM INTENSITY OF DRIFT SURCHARGE LOAD

$$P_d = h_d\gamma$$

h_d =1.85 FT
γ = 14.26 PCF

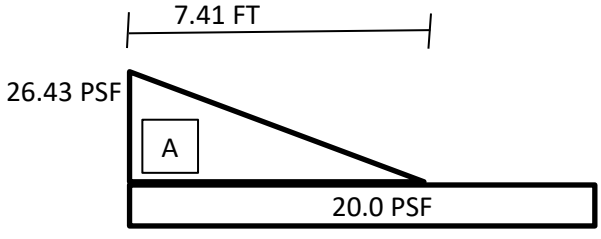
P_d = 26.43 PSF

DRIFT LENGTH

$$w = 4h_d$$

h_d = 1.85 FT

w= 7.41 FT



SNOW DRIFT B

FULL DRIFT

$$h_d = .43\sqrt[3]{l_u}\sqrt[4]{P_g + 10} - 1.5$$ (ASCE 7-10 : FIG. 7-9)

l_u = 20.00 FT 20 if less than 20 FT use 20 FT

P_g = 20.0 PSF

h_d = 1.23 FT

LESS THAN PARAPET HEIGHT: 5.40 FT
THEREFORE FULL DRIFT POSSIBLE TRUE

MAXIMUM INTENSITY OF DRIFT SURCHARGE LOAD

$$P_d = h_d\gamma$$

h_d =1.23 FT
γ = 14.26 PCF

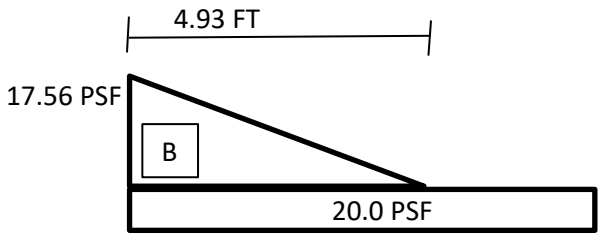
P_d = 17.56 PSF

DRIFT LENGTH

$$w = 4h_d$$

h_d = 1.23 FT

w= 4.93 FT



- HEADER (A)

ROOF DEAD LOAD:	20	PSF	TRIBUTARY WIDTH (ROOF) = 8'-0"
ROOF LIVE LOAD:	20	PSF	TRIBUTARY WIDTH (WALL) = 9'-1"
FLAT ROOF SNOW LOAD:	20	PSF	
SNOW DRIFT A: LOAD	17.0	PSF	= 26.4 PSF-(2.63 FT(26.4 PSF/7.42 FT))
FOR 4.79 FT	0	PSF	7.42 FT – 2.63 FT = 4.79 FT
SNOW DRIFT B LOAD:	43.3	PLF	= 17.6 PSF (4.92 FT)/ 2
WALL LOAD:	15	PSF	

SPAN = 6'-6"

USE (3) 2x10 W/ ½" PLYWOOD SPACERS*

- HEADER (B)

ROOF DEAD LOAD:	20	PSF	TRIBUTARY WIDTH (ROOF) = 1'-0"
ROOF LIVE LOAD:	20	PSF	TRIBUTARY WIDTH (WALL) = 9'-1"
FLAT ROOF SNOW LOAD:	20	PSF	
SNOW DRIFT A LOAD:	26.4	PLF	= 26.4 PSF (1ft)
WALL LOAD:	15	PSF	

SPAN = 6'-0"

USE (3) 2x8 W/ ½" PLYWOOD SPACERS*

- HEADER (C)

ROOF DEAD LOAD:	20	PSF	TRIBUTARY WIDTH (ROOF) = 1'-0"
ROOF LIVE LOAD:	20	PSF	TRIBUTARY WIDTH (WALL) = 9'-1"
FLAT ROOF SNOW LOAD:	20	PSF	
SNOW DRIFT A LOAD:	26.4	PLF	= 26.4 PSF (1ft)
WALL LOAD:	15	PSF	

SPAN = 3'-4"

USE (3) 2x6 W/ ½" PLYWOOD SPACERS*

*REFER TO ATTACHED ENERCALC OUTPUT FOR DESIGN DETAILS

Wood Beam

Lic. # : KW-06006725 — RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: HEADER A

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Load Resistance Factor D
Load Combination : ASCE 7-10

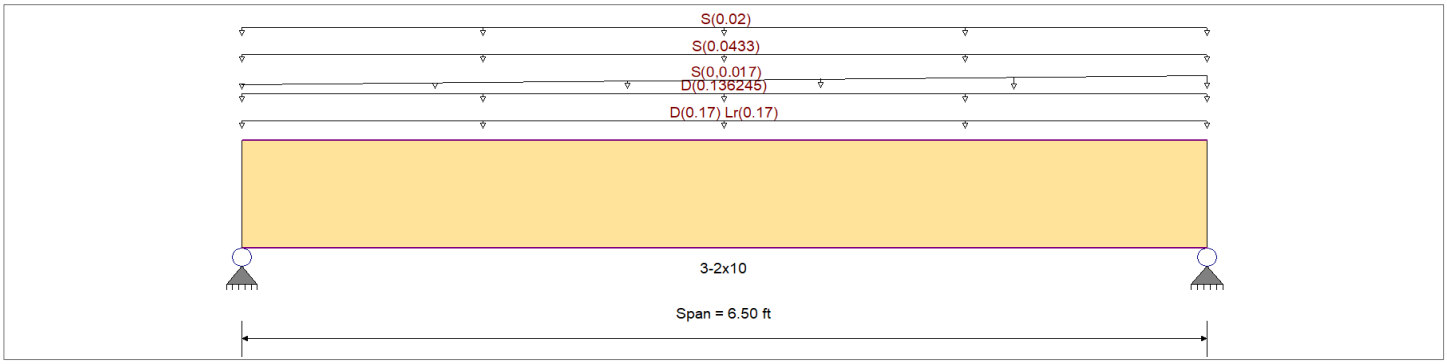
Wood Species : Southern Pine
Wood Grade : No.2: 2"-4" Thick: 10" Wide

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	800.0 psi
Fb -	800.0 psi
Fc - Prll	1,300.0 psi
Fc - Perp	565.0 psi
Fv	175.0 psi
Ft	475.0 psi

E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	34.330 pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.020, Lr = 0.020 ksf, Tributary Width = 8.50 ft, (ROOF LOAD)
Uniform Load : D = 0.0150 ksf, Tributary Width = 9.083 ft, (WALL LOAD)
Varying Uniform Load : S= 0.0->0.0170 k/ft, Extent = 0.0 -->> 6.50 ft, Trib Width = 1.0 ft, (SNOW DRIFT A)

Uniform Load : S = 0.04330 , Tributary Width = 1.0 ft, (SNOW DRIFT B)
Uniform Load : S = 0.020 , Tributary Width = 1.0 ft, (FLAT ROOF SNOW LOAD)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.465 : 1	Maximum Shear Stress Ratio	=	0.193 : 1
Section used for this span		3-2x10	Section used for this span		3-2x10
fb: Actual	=	643.31 psi	fv: Actual	=	58.47 psi
Fb: Allowable	=	1,382.40 psi	Fv: Allowable	=	302.40 psi
Load Combination		+1.20D+1.60Lr+0.50L+1.60H	Load Combination		+1.20D+1.60Lr+0.50L+1.60H
Location of maximum on span	=	3.250 ft	Location of maximum on span	=	5.741 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.017 in	Ratio =	4719 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.047 in	Ratio =	1650 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	λ	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values Mu	fb	Fb	Shear Values Vu	fv	Fv
+1.40D+1.60H													0.00	0.00	0.00	0.00
Length = 6.50 ft	1	0.422	0.175	0.60	1.000	1.00	1.00	1.00	1.00	1.00	2.34	437.14	1036.80	1.10	39.73	226.80
+1.20D+0.50Lr+1.60L+1.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.50 ft	1	0.332	0.138	0.80	1.000	1.00	1.00	1.00	1.00	1.00	2.45	458.64	1382.40	1.16	41.69	302.40
+1.20D+1.60L+0.50S+1.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.50 ft	1	0.297	0.123	0.80	1.000	1.00	1.00	1.00	1.00	1.00	2.19	410.15	1382.40	1.04	37.34	302.40
+1.20D+1.60Lr+0.50L+1.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.50 ft	1	0.465	0.193	0.80	1.000	1.00	1.00	1.00	1.00	1.00	3.44	643.31	1382.40	1.62	58.47	302.40

Wood Beam

Lic. # : KW-06006725 RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: HEADER A

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	λ	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values Mu	fb	Fb	Shear Values Vu	fv	Fv
+1.20D+1.60Lr+0.50W+1.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.50 ft	1	0.465	0.193	0.80	1.000	1.00	1.00	1.00	1.00	1.00	3.44	643.31	1382.40	1.62	58.47	302.40
+1.20D+0.50L+1.60S+1.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.50 ft	1	0.353	0.147	0.80	1.000	1.00	1.00	1.00	1.00	1.00	2.61	488.15	1382.40	1.24	44.57	302.40
+1.20D+1.60S+0.50W+1.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.50 ft	1	0.353	0.147	0.80	1.000	1.00	1.00	1.00	1.00	1.00	2.61	488.15	1382.40	1.24	44.57	302.40
+1.20D+0.50Lr+0.50L+W+1.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.50 ft	1	0.265	0.110	1.00	1.000	1.00	1.00	1.00	1.00	1.00	2.45	458.64	1728.00	1.16	41.69	378.00
+1.20D+0.50L+0.50S+W+1.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.50 ft	1	0.237	0.099	1.00	1.000	1.00	1.00	1.00	1.00	1.00	2.19	410.15	1728.00	1.04	37.34	378.00
+1.20D+0.50L+0.20S+E+1.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.50 ft	1	0.225	0.094	1.00	1.000	1.00	1.00	1.00	1.00	1.00	2.08	388.87	1728.00	0.98	35.37	378.00
+0.90D+W+0.90H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.50 ft	1	0.163	0.068	1.00	1.000	1.00	1.00	1.00	1.00	1.00	1.50	281.02	1728.00	0.71	25.54	378.00
+0.90D+E+0.90H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.50 ft	1	0.163	0.068	1.00	1.000	1.00	1.00	1.00	1.00	1.00	1.50	281.02	1728.00	0.71	25.54	378.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr+H	1	0.0473	3.274		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.580	1.580
Overall MINimum	0.224	0.243
+D+H	1.028	1.028
+D+L+H	1.028	1.028
+D+Lr+H	1.580	1.580
+D+S+H	1.252	1.270
+D+0.750Lr+0.750L+H	1.442	1.442
+D+0.750L+0.750S+H	1.196	1.209
+D+0.60W+H	1.028	1.028
+D+0.70E+H	1.028	1.028
+D+0.750Lr+0.750L+0.450W+H	1.442	1.442
+D+0.750L+0.750S+0.450W+H	1.196	1.209
+D+0.750L+0.750S+0.5250E+H	1.196	1.209
+0.60D+0.60W+0.60H	0.617	0.617
+0.60D+0.70E+0.60H	0.617	0.617
D Only	1.028	1.028
Lr Only	0.553	0.553
S Only	0.224	0.243
H Only		

Wood Beam

Lic. # : KW-06006725 — RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: HEADER B

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

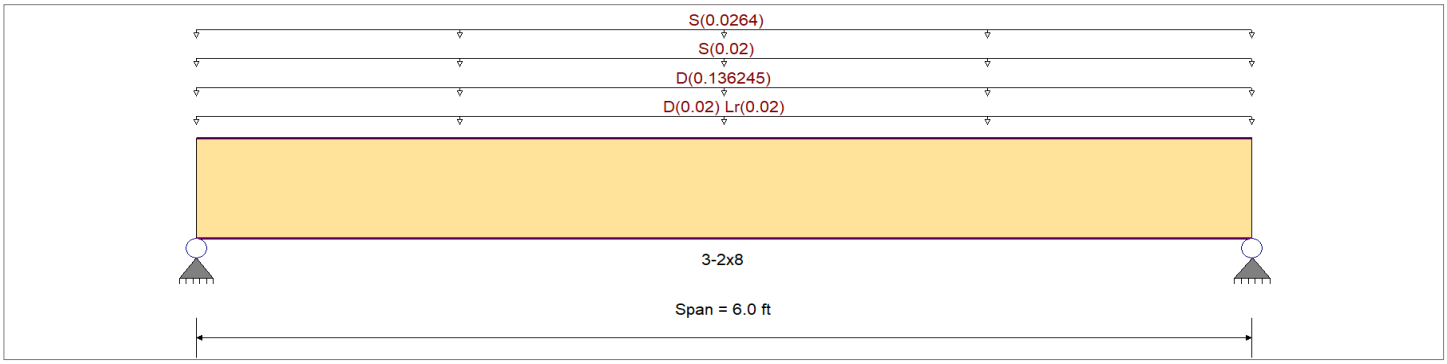
Wood Species : Southern Pine
Wood Grade : No.2: 2"-4" Thick: 8" Wide

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	925.0 psi
Fb -	925.0 psi
Fc - Prll	1,350.0 psi
Fc - Perp	565.0 psi
Fv	175.0 psi
Ft	550.0 psi

E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	34.330 pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.020, Lr = 0.020 , Tributary Width = 1.0 ft, (ROOF LOAD)
Uniform Load : D = 0.0150 ksf, Tributary Width = 9.083 ft, (WALL LOAD)
Uniform Load : S = 0.020 , Tributary Width = 1.0 ft, (FLAR ROOF SNOW LOAD)
Uniform Load : S = 0.02640 , Tributary Width = 1.0 ft, (SNOW DRIFT A LOAD)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.271 : 1	Maximum Shear Stress Ratio	=	0.116 : 1
Section used for this span		3-2x8	Section used for this span		3-2x8
fb: Actual	=	288.24 psi	fv: Actual	=	23.30 psi
Fb: Allowable	=	1,063.75 psi	Fv: Allowable	=	201.25 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	3.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.007 in	Ratio =	10584 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.031 in	Ratio =	2333 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.270	0.115	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.74	224.68	832.50	0.40	18.17	157.50
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.243	0.104	1.00	1.000	1.00	1.00	1.00	1.00	1.00	0.74	224.68	925.00	0.40	18.17	175.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.218	0.093	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.83	252.07	1156.25	0.44	20.38	218.75
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.271	0.116	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.95	288.24	1063.75	0.51	23.30	201.25
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.212	0.091	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.81	245.23	1156.25	0.43	19.83	218.75

Wood Beam

Lic. # : KW-06006725 RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: HEADER B

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.256	0.109	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.89	272.35	1063.75	0.48	22.02	201.25
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.152	0.065	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.74	224.68	1480.00	0.40	18.17	280.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.152	0.065	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.74	224.68	1480.00	0.40	18.17	280.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.166	0.071	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.81	245.23	1480.00	0.43	19.83	280.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.184	0.079	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.89	272.35	1480.00	0.48	22.02	280.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.184	0.079	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.89	272.35	1480.00	0.48	22.02	280.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.091	0.039	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.44	134.81	1480.00	0.24	10.90	280.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.091	0.039	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.44	134.81	1480.00	0.24	10.90	280.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.0308	3.022		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.631	0.631
Overall MINimum	0.139	0.139
+D+H	0.492	0.492
+D+L+H	0.492	0.492
+D+Lr+H	0.552	0.552
+D+S+H	0.631	0.631
+D+0.750Lr+0.750L+H	0.537	0.537
+D+0.750L+0.750S+H	0.596	0.596
+D+0.60W+H	0.492	0.492
+D+0.70E+H	0.492	0.492
+D+0.750Lr+0.750L+0.450W+H	0.537	0.537
+D+0.750L+0.750S+0.450W+H	0.596	0.596
+D+0.750L+0.750S+0.5250E+H	0.596	0.596
+0.60D+0.60W+0.60H	0.295	0.295
+0.60D+0.70E+0.60H	0.295	0.295
D Only	0.492	0.492
Lr Only	0.060	0.060
S Only	0.139	0.139
H Only		

Wood Beam

Lic. # : KW-06006725 RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: HEADER C

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

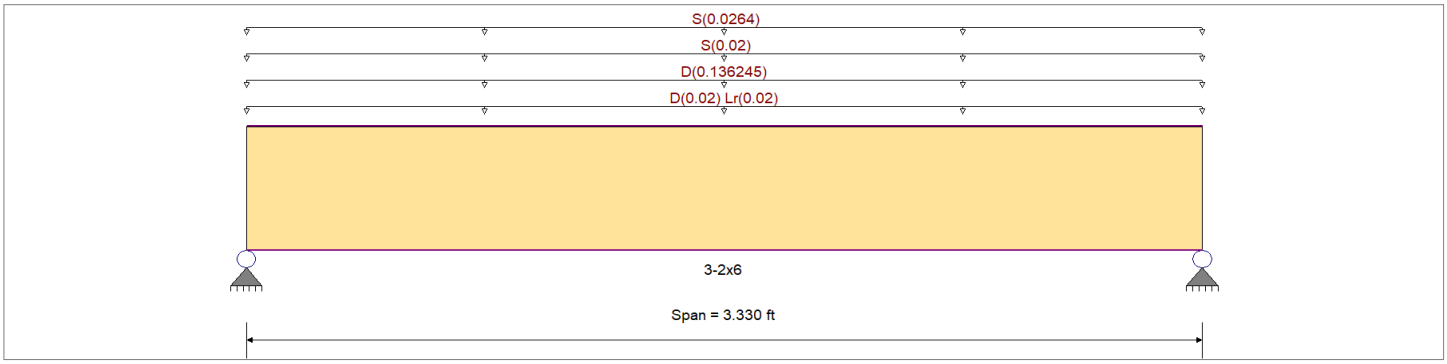
Wood Species : Southern Pine
Wood Grade : No.2: 2"-4" Thick: 5"-6" Wide

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,400.0 psi
Fc - Perp	565.0 psi
Fv	175.0 psi
Ft	600.0 psi

E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	34.330 pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.020, Lr = 0.020 , Tributary Width = 1.0 ft, (ROOF LOAD)
Uniform Load : D = 0.0150 ksf, Tributary Width = 9.083 ft, (WALL LOAD)
Uniform Load : S = 0.020 , Tributary Width = 1.0 ft, (FLAT ROOF SNOW LOAD)
Uniform Load : S = 0.02640 , Tributary Width = 1.0 ft, (SNOW DRIFT A LOAD)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.133 : 1	Maximum Shear Stress Ratio	=	0.076 : 1
Section used for this span		3-2x6	Section used for this span		3-2x6
fb: Actual	=	152.90 psi	fv: Actual	=	15.36 psi
Fb: Allowable	=	1,150.00 psi	Fv: Allowable	=	201.25 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	1.665 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.001 in	Ratio =	27031 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.007 in	Ratio =	6014 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 3.330 ft	1	0.132	0.076	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.22	118.88	900.00	0.20	11.94	157.50
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.330 ft	1	0.119	0.068	1.00	1.000	1.00	1.00	1.00	1.00	1.00	0.22	118.88	1000.00	0.20	11.94	175.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.330 ft	1	0.107	0.061	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.25	133.54	1250.00	0.22	13.42	218.75
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.330 ft	1	0.133	0.076	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.29	152.90	1150.00	0.25	15.36	201.25
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.330 ft	1	0.104	0.060	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.25	129.87	1250.00	0.22	13.05	218.75

Wood Beam

Lic. # : KW-06006725

RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: HEADER C

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.330 ft	1	0.126	0.072	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.27	144.39	1150.00	0.24	14.51	201.25
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.330 ft	1	0.074	0.043	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.22	118.88	1600.00	0.20	11.94	280.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.330 ft	1	0.074	0.043	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.22	118.88	1600.00	0.20	11.94	280.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.330 ft	1	0.081	0.047	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.25	129.87	1600.00	0.22	13.05	280.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.330 ft	1	0.090	0.052	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.27	144.39	1600.00	0.24	14.51	280.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.330 ft	1	0.090	0.052	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.27	144.39	1600.00	0.24	14.51	280.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.330 ft	1	0.045	0.026	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.13	71.33	1600.00	0.12	7.17	280.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.330 ft	1	0.045	0.026	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.13	71.33	1600.00	0.12	7.17	280.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.0066	1.677		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.347	0.347
Overall MINimum	0.077	0.077
+D+H	0.270	0.270
+D+L+H	0.270	0.270
+D+Lr+H	0.303	0.303
+D+S+H	0.347	0.347
+D+0.750Lr+0.750L+H	0.295	0.295
+D+0.750L+0.750S+H	0.328	0.328
+D+0.60W+H	0.270	0.270
+D+0.70E+H	0.270	0.270
+D+0.750Lr+0.750L+0.450W+H	0.295	0.295
+D+0.750L+0.750S+0.450W+H	0.328	0.328
+D+0.750L+0.750S+0.5250E+H	0.328	0.328
+0.60D+0.60W+0.60H	0.162	0.162
+0.60D+0.70E+0.60H	0.162	0.162
D Only	0.270	0.270
Lr Only	0.033	0.033
S Only	0.077	0.077
H Only		

- ROOF JOIST: SNOW DRIFT A, 1st PARALLEL TO WALL

ROOF DEAD LOAD:	20	PSF	TRIBUTARY WIDTH (ROOF) = 1’-4”
ROOF LIVE LOAD:	20	PSF	
SNOW LOAD:	20	PSF	
SNOW DRIFT A: LOAD	26.4	PSF	

SPAN = 17’-0”

USE: (2)2X12*

- ROOF JOIST: SNOW DRIFT A & B 5th PARALLEL TO WALL

ROOF DEAD LOAD:	20	PSF	TRIBUTARY WIDTH (ROOF) = 1’-4”
ROOF LIVE LOAD:	20	PSF	
SNOW LOAD:	20	PSF	
SNOW DRIFT A: LOAD	2.67	PSF	
SNOW DRIFT B LOAD:	17.6	PSF	4.42’ EACH END
	2.67	PSF	

SPAN = 17’-0”

USE: (2)2X12*

- ROOF JOIST: SNOW DRIFT B

ROOF DEAD LOAD:	20	PSF	TRIBUTARY WIDTH (ROOF) = 1’-4”
ROOF LIVE LOAD:	20	PSF	
SNOW LOAD:	20	PSF	
SNOW DRIFT B LOAD:	17.6	PSF	4.92’ EACH END
	0	PSF	

SPAN = 17’-0”

USE: 2X12*

- ROOF JOIST: SNOW DRIFT B & MECH

ROOF DEAD LOAD:	20	PSF	TRIBUTARY WIDTH (ROOF) = 1’-4”
ROOF LIVE LOAD:	20	PSF	
SNOW LOAD:	20	PSF	
SNOW DRIFT B LOAD:	17.6	PSF	4.92’ EACH END
	0	PSF	
MECH LOAD:	140	LBS	@ 8’-5’’

SPAN = 17’-0”

USE: (2)2X12*

*REFER TO ATTACHED ENERCALC OUTPUT FOR DESIGN DETAILS

** REFER TO ATTACHED SIMPSON BASE PLATE OUTPUT FOR DESIGN DETAILS

Wood Beam

Lic. # : KW-06006725 RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: ROOF JOIST: SNOW DRIFT A, 1ST PARALLEL TO WALL

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Southern Pine
Wood Grade : No.2: 2"-4" Thick: 12" Wide

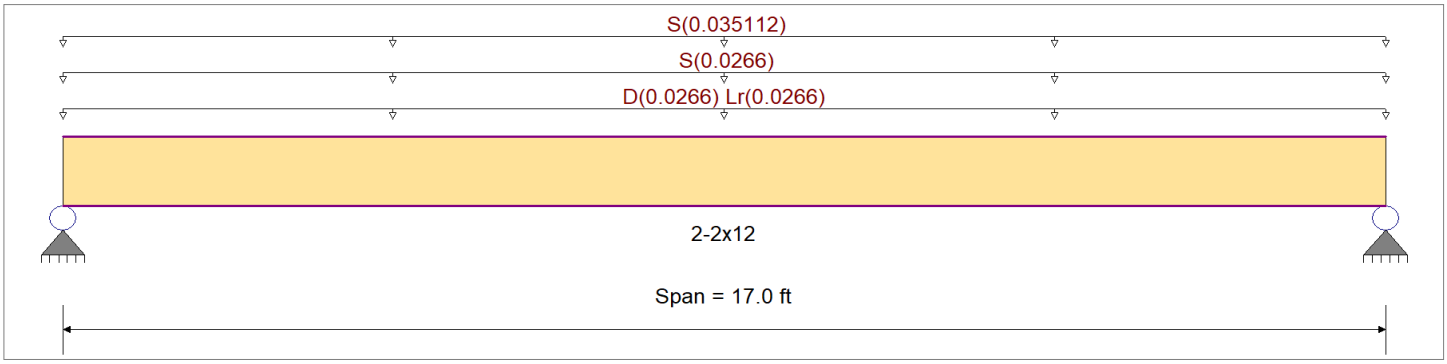
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	750.0 psi
Fb -	750.0 psi
Fc - Prll	1,250.0 psi
Fc - Perp	565.0 psi
Fv	175.0 psi
Ft	450.0 psi

E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	34.330 pcf

Repetitive Member Stress Increase



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.020, Lr = 0.020 ksf, Tributary Width = 1.330 ft, (ROOF LOAD)
Uniform Load : S = 0.020 ksf, Tributary Width = 1.330 ft, (FLAT ROOF SNOW LOAD)
Uniform Load : S = 0.02640 ksf, Tributary Width = 1.330 ft, (SNOW DRIFT A LOAD)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.665 : 1	Maximum Shear Stress Ratio	=	0.161 : 1
Section used for this span		2-2x12	Section used for this span		2-2x12
fb: Actual	=	660.09 psi	fv: Actual	=	32.42 psi
Fb: Allowable	=	991.88 psi	Fv: Allowable	=	201.25 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	8.500 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.234 in	Ratio =	871 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.365 in	Ratio =	558 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
+D+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.306	0.074	0.90							1.25	237.34	776.25	0.26	11.66	157.50
+D+L+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.275	0.067	1.00							1.25	237.34	862.50	0.26	11.66	175.00
+D+Lr+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.389	0.094	1.25							2.21	419.56	1078.13	0.46	20.60	218.75
+D+S+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.665	0.161	1.15							3.48	660.09	991.88	0.73	32.42	201.25
+D+0.750Lr+0.750L+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.347	0.084	1.25							1.97	374.00	1078.13	0.41	18.37	218.75
+D+0.750L+0.750S+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

Lic. # : KW-06006725

RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: ROOF JOIST: SNOW DRIFT A, 1ST PARALLEL TO WALL

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 17.0 ft	1	0.559	0.135	1.15	1.000	1.00	1.15	1.00	1.00	1.00	2.92	554.40	991.88	0.61	27.23	201.25
+D+0.60W+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.172	0.042	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.25	237.34	1380.00	0.26	11.66	280.00
+D+0.70E+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.172	0.042	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.25	237.34	1380.00	0.26	11.66	280.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.271	0.066	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.97	374.00	1380.00	0.41	18.37	280.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.402	0.097	1.60	1.000	1.00	1.15	1.00	1.00	1.00	2.92	554.40	1380.00	0.61	27.23	280.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.402	0.097	1.60	1.000	1.00	1.15	1.00	1.00	1.00	2.92	554.40	1380.00	0.61	27.23	280.00
+0.60D+0.60W+0.60H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.103	0.025	1.60	1.000	1.00	1.15	1.00	1.00	1.00	0.75	142.40	1380.00	0.16	6.99	280.00
+0.60D+0.70E+0.60H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.103	0.025	1.60	1.000	1.00	1.15	1.00	1.00	1.00	0.75	142.40	1380.00	0.16	6.99	280.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.3655	8.562		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.819	0.819
Overall MINimum	0.525	0.525
+D+H	0.294	0.294
+D+L+H	0.294	0.294
+D+Lr+H	0.521	0.521
+D+S+H	0.819	0.819
+D+0.750Lr+0.750L+H	0.464	0.464
+D+0.750L+0.750S+H	0.688	0.688
+D+0.60W+H	0.294	0.294
+D+0.70E+H	0.294	0.294
+D+0.750Lr+0.750L+0.450W+H	0.464	0.464
+D+0.750L+0.750S+0.450W+H	0.688	0.688
+D+0.750L+0.750S+0.5250E+H	0.688	0.688
+0.60D+0.60W+0.60H	0.177	0.177
+0.60D+0.70E+0.60H	0.177	0.177
D Only	0.294	0.294
Lr Only	0.226	0.226
S Only	0.525	0.525
H Only		

Wood Beam

Lic. # : KW-06006725 RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: ROOF JOIST: SNOW DRIFT A & B, 5TH PARALLEL TO WALL

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Southern Pine
Wood Grade : No.2: 2"-4" Thick: 12" Wide

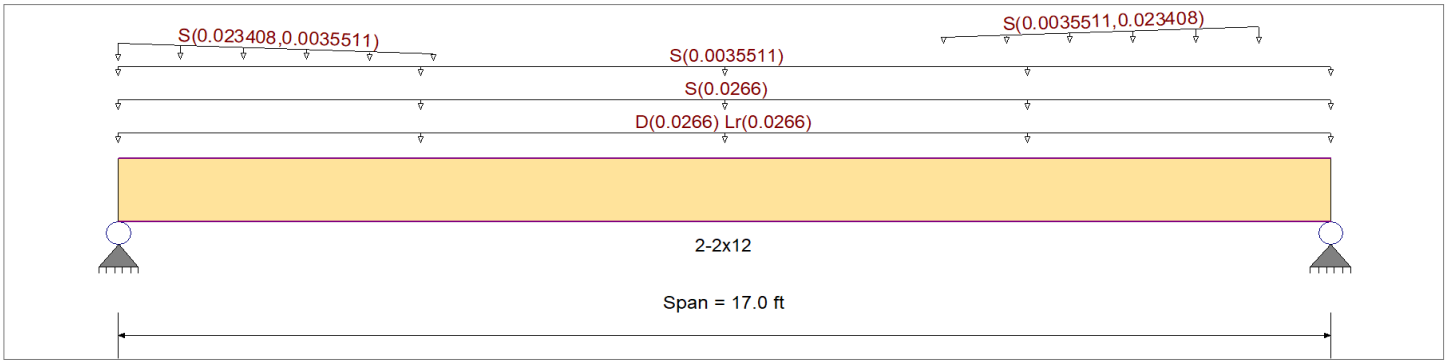
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	750.0 psi
Fb -	750.0 psi
Fc - Prll	1,250.0 psi
Fc - Perp	565.0 psi
Fv	175.0 psi
Ft	450.0 psi

E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	34.330 pcf

Repetitive Member Stress Increase



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.020, Lr = 0.020 ksf, Tributary Width = 1.330 ft, (ROOF LOAD)
Uniform Load : S = 0.020 ksf, Tributary Width = 1.330 ft, (FLAT ROOF SNOW LOAD)
Uniform Load : S = 0.002670 ksf, Tributary Width = 1.330 ft, (SNOW DRIFT A LOAD)
Varying Uniform Load : S= 0.01760->0.002670 ksf, Extent = 0.0 -->> 4.420 ft, Trib Width = 1.330 ft, (SNOW DRIFT B LOAD 1)

Varying Uniform Load : S= 0.002670->0.01760 ksf, Extent = 11.580 -->> 16.0 ft, Trib Width = 1.330 ft, (SNOW DRIFT B LOAD 2)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.472 : 1	Maximum Shear Stress Ratio	=	0.121 : 1
Section used for this span		2-2x12	Section used for this span		2-2x12
fb: Actual	=	468.39 psi	fv: Actual	=	24.29 psi
Fb: Allowable	=	991.88 psi	Fv: Allowable	=	201.25 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	8.562 ft	Location of maximum on span	=	16.069 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.130 in	Ratio =	1570 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.261 in	Ratio =	780 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
+D+H													0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.306	0.074	0.90	1.000	1.00	1.15	1.00	1.00	1.00	1.25	237.34	776.25	0.26	11.66	157.50
+D+L+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.275	0.067	1.00	1.000	1.00	1.15	1.00	1.00	1.00	1.25	237.34	862.50	0.26	11.66	175.00
+D+Lr+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.389	0.094	1.25	1.000	1.00	1.15	1.00	1.00	1.00	2.21	419.56	1078.13	0.46	20.60	218.75

Wood Beam

Lic. # : KW-06006725 RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: ROOF JOIST: SNOW DRIFT A & B, 5TH PARALLEL TO WALL

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+S+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.472	0.121	1.15	1.000	1.00	1.15	1.00	1.00	1.00	2.47	468.39	991.88	0.55	24.29	201.25
+D+0.750Lr+0.750L+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.347	0.084	1.25	1.000	1.00	1.15	1.00	1.00	1.00	1.97	374.00	1078.13	0.41	18.37	218.75
+D+0.750L+0.750S+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.414	0.105	1.15	1.000	1.00	1.15	1.00	1.00	1.00	2.17	410.62	991.88	0.48	21.13	201.25
+D+0.60W+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.172	0.042	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.25	237.34	1380.00	0.26	11.66	280.00
+D+0.70E+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.172	0.042	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.25	237.34	1380.00	0.26	11.66	280.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.271	0.066	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.97	374.00	1380.00	0.41	18.37	280.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.298	0.075	1.60	1.000	1.00	1.15	1.00	1.00	1.00	2.17	410.62	1380.00	0.48	21.13	280.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.298	0.075	1.60	1.000	1.00	1.15	1.00	1.00	1.00	2.17	410.62	1380.00	0.48	21.13	280.00
+0.60D+0.60W+0.60H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.103	0.025	1.60	1.000	1.00	1.15	1.00	1.00	1.00	0.75	142.40	1380.00	0.16	6.99	280.00
+0.60D+0.70E+0.60H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.103	0.025	1.60	1.000	1.00	1.15	1.00	1.00	1.00	0.75	142.40	1380.00	0.16	6.99	280.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.2613	8.562		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.614	0.607
Overall MINimum	0.319	0.312
+D+H	0.294	0.294
+D+L+H	0.294	0.294
+D+Lr+H	0.521	0.521
+D+S+H	0.614	0.607
+D+0.750Lr+0.750L+H	0.464	0.464
+D+0.750L+0.750S+H	0.534	0.529
+D+0.60W+H	0.294	0.294
+D+0.70E+H	0.294	0.294
+D+0.750Lr+0.750L+0.450W+H	0.464	0.464
+D+0.750L+0.750S+0.450W+H	0.534	0.529
+D+0.750L+0.750S+0.5250E+H	0.534	0.529
+0.60D+0.60W+0.60H	0.177	0.177
+0.60D+0.70E+0.60H	0.177	0.177
D Only	0.294	0.294
Lr Only	0.226	0.226
S Only	0.319	0.312
H Only		

Wood Beam

Lic. # : KW-06006725 RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: ROOF JOIST SNOW DRIFT B

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Southern Pine
Wood Grade : No.2: 2"-4" Thick: 12" Wide

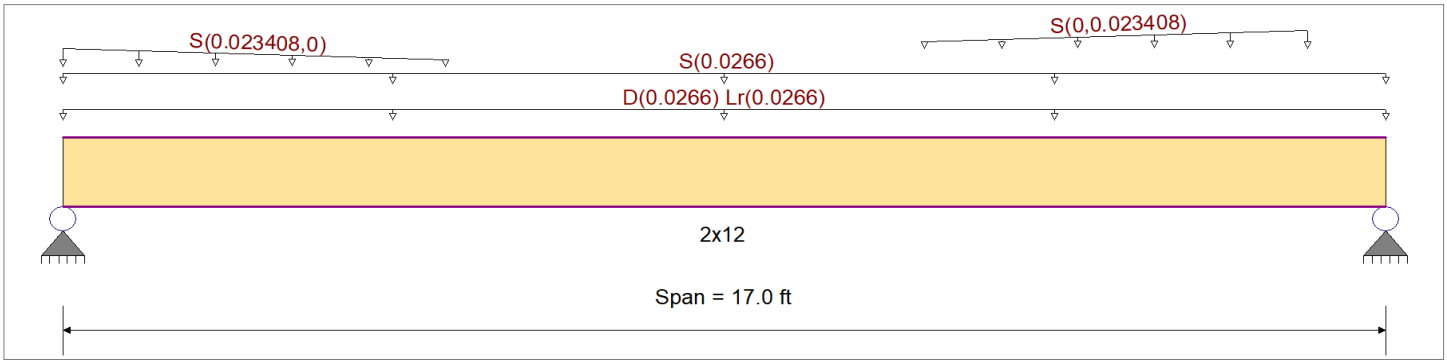
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	750.0 psi
Fb -	750.0 psi
Fc - Prll	1,250.0 psi
Fc - Perp	565.0 psi
Fv	175.0 psi
Ft	450.0 psi

E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	34.330 pcf

Repetitive Member Stress Increase



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.020, Lr = 0.020 ksf, Tributary Width = 1.330 ft, (ROOF LOAD)
Uniform Load : S = 0.020 ksf, Tributary Width = 1.330 ft, (FLAT ROOF SNOW LOAD)
Varying Uniform Load : S= 0.01760->0.0 ksf, Extent = 0.0 -->> 4.920 ft, Trib Width = 1.330 ft, (SNOW DRIFT B LOAD 1)

Varying Uniform Load : S= 0.0->0.01760 ksf, Extent = 11.080 -->> 16.0 ft, Trib Width = 1.330 ft, (SNOW DRIFT B LOAD 2)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.838 : 1	Maximum Shear Stress Ratio	=	0.215 : 1
Section used for this span		2x12	Section used for this span		2x12
fb: Actual	=	830.77 psi	fv: Actual	=	43.32 psi
Fb: Allowable	=	991.88 psi	Fv: Allowable	=	201.25 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	8.562 ft	Location of maximum on span	=	16.069 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.231 in	Ratio =	881 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.464 in	Ratio =	439 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
+D+H													0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.540	0.131	0.90	1.000	1.00	1.15	1.00	1.00	1.00	1.11	419.56	776.25	0.23	20.60	157.50
+D+L+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.486	0.118	1.00	1.000	1.00	1.15	1.00	1.00	1.00	1.11	419.56	862.50	0.23	20.60	175.00
+D+Lr+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.727	0.176	1.25	1.000	1.00	1.15	1.00	1.00	1.00	2.07	784.00	1078.13	0.43	38.50	218.75
+D+S+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

Lic. # : KW-06006725 RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: ROOF JOIST SNOW DRIFT B

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 17.0 ft	1	0.838	0.215	1.15	1.000	1.00	1.15	1.00	1.00	1.00	2.19	830.77	991.88	0.49	43.32	201.25
+D+0.750Lr+0.750L+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.643	0.156	1.25	1.000	1.00	1.15	1.00	1.00	1.00	1.83	692.89	1078.13	0.38	34.03	218.75
+D+0.750L+0.750S+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.734	0.187	1.15	1.000	1.00	1.15	1.00	1.00	1.00	1.92	727.96	991.88	0.42	37.64	201.25
+D+0.60W+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.304	0.074	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.11	419.56	1380.00	0.23	20.60	280.00
+D+0.70E+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.304	0.074	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.11	419.56	1380.00	0.23	20.60	280.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.502	0.122	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.83	692.89	1380.00	0.38	34.03	280.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.528	0.134	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.92	727.96	1380.00	0.42	37.64	280.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.528	0.134	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.92	727.96	1380.00	0.42	37.64	280.00
+0.60D+0.60W+0.60H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.182	0.044	1.60	1.000	1.00	1.15	1.00	1.00	1.00	0.66	251.74	1380.00	0.14	12.36	280.00
+0.60D+0.70E+0.60H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.182	0.044	1.60	1.000	1.00	1.15	1.00	1.00	1.00	0.66	251.74	1380.00	0.14	12.36	280.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.4638	8.562		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.547	0.541
Overall MINimum	0.287	0.280
+D+H	0.260	0.260
+D+L+H	0.260	0.260
+D+Lr+H	0.486	0.486
+D+S+H	0.547	0.541
+D+0.750Lr+0.750L+H	0.430	0.430
+D+0.750L+0.750S+H	0.476	0.471
+D+0.60W+H	0.260	0.260
+D+0.70E+H	0.260	0.260
+D+0.750Lr+0.750L+0.450W+H	0.430	0.430
+D+0.750L+0.750S+0.450W+H	0.476	0.471
+D+0.750L+0.750S+0.5250E+H	0.476	0.471
+0.60D+0.60W+0.60H	0.156	0.156
+0.60D+0.70E+0.60H	0.156	0.156
D Only	0.260	0.260
Lr Only	0.226	0.226
S Only	0.287	0.280
H Only		

Wood Beam

Lic. # : KW-06006725 RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: ROOF JOIST SNOW DRIFT B AND MECH

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Southern Pine
Wood Grade : No.2: 2"-4" Thick: 12" Wide

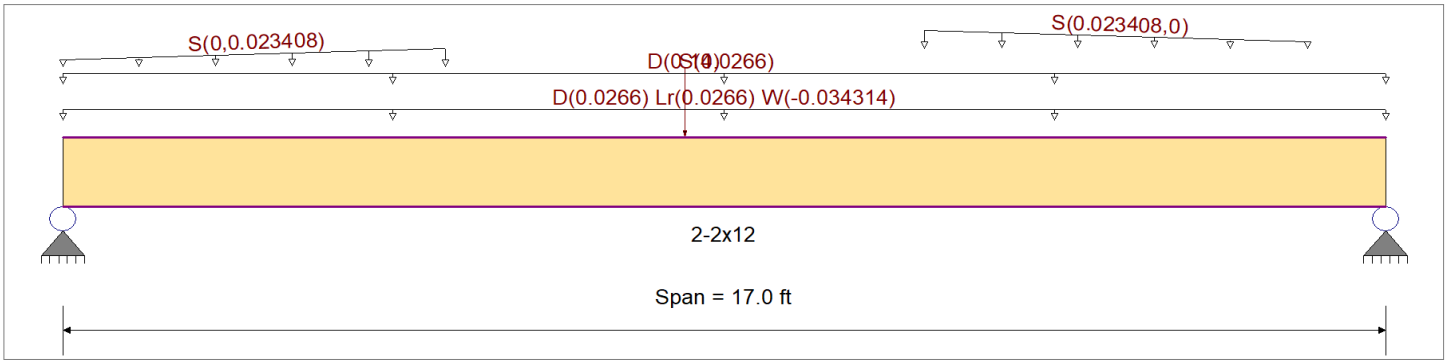
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	750.0 psi
Fb -	750.0 psi
Fc - Prll	1,250.0 psi
Fc - Perp	565.0 psi
Fv	175.0 psi
Ft	450.0 psi

E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	34.330 pcf

Repetitive Member Stress Increase



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.020, Lr = 0.020, W = -0.02580 ksf, Tributary Width = 1.330 ft, (ROOF LOAD)
Uniform Load : S = 0.020 ksf, Tributary Width = 1.330 ft, (FLAT ROOF SNOW LOAD)
Varying Uniform Load : S= 0.0->0.01760 ksf, Extent = 0.0 -->> 4.920 ft, Trib Width = 1.330 ft, (SNOW DRIFT B LOAD 1)

Varying Uniform Load : S= 0.01760->0.0 ksf, Extent = 11.080 -->> 16.0 ft, Trib Width = 1.330 ft, (SNOW DRIFT B LOAD 2)

Point Load : D = 0.140 k @ 8.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.576 : 1	Maximum Shear Stress Ratio	=	0.132 : 1
Section used for this span		2-2x12	Section used for this span		2-2x12
fb: Actual	=	571.48 psi	fv: Actual	=	26.52 psi
Fb: Allowable	=	991.88 psi	Fv: Allowable	=	201.25 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	8.004 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.126 in	Ratio =	1619 >=360
Max Upward Transient Deflection	-0.130 in	Ratio =	1567 >=360
Max Downward Total Deflection	0.307 in	Ratio =	664 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.449	0.095	0.90	1.000	1.00	1.15	1.00	1.00	1.00	1.84	348.92	776.25	0.34	14.95	157.50
+D+L+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.405	0.085	1.00	1.000	1.00	1.15	1.00	1.00	1.00	1.84	348.92	862.50	0.34	14.95	175.00
+D+Lr+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.492	0.109	1.25	1.000	1.00	1.15	1.00	1.00	1.00	2.80	530.52	1078.13	0.54	23.90	218.75

Wood Beam

Lic. # : KW-06006725 RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: ROOF JOIST SNOW DRIFT B AND MECH

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+S+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.576	0.132	1.15	1.000	1.00	1.15	1.00	1.00	1.00	3.01	571.48	991.88	0.60	26.52	201.25
+D+0.750Lr+0.750L+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.450	0.099	1.25	1.000	1.00	1.15	1.00	1.00	1.00	2.56	485.12	1078.13	0.49	21.66	218.75
+D+0.750L+0.750S+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.520	0.117	1.15	1.000	1.00	1.15	1.00	1.00	1.00	2.72	515.84	991.88	0.53	23.62	201.25
+D+0.60W+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.151	0.029	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.10	208.37	1380.00	0.18	8.02	280.00
+D+0.70E+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.253	0.053	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.84	348.92	1380.00	0.34	14.95	280.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.275	0.059	1.60	1.000	1.00	1.15	1.00	1.00	1.00	2.00	379.70	1380.00	0.37	16.47	280.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.297	0.066	1.60	1.000	1.00	1.15	1.00	1.00	1.00	2.16	410.42	1380.00	0.41	18.43	280.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.374	0.084	1.60	1.000	1.00	1.15	1.00	1.00	1.00	2.72	515.84	1380.00	0.53	23.62	280.00
+0.60D+0.60W+0.60H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.050	0.007	1.60	1.000	1.00	1.15	1.00	1.00	1.00	0.36	68.80	1380.00	0.05	2.04	280.00
+0.60D+0.70E+0.60H					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 17.0 ft	1	0.152	0.032	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.10	209.35	1380.00	0.20	8.97	280.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.3071	8.500		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.656	0.641
Overall MINimum	0.287	0.280
+D+H	0.369	0.360
+D+L+H	0.369	0.360
+D+Lr+H	0.595	0.586
+D+S+H	0.656	0.641
+D+0.750Lr+0.750L+H	0.538	0.530
+D+0.750L+0.750S+H	0.584	0.571
+D+0.60W+H	0.194	0.185
+D+0.70E+H	0.369	0.360
+D+0.750Lr+0.750L+0.450W+H	0.407	0.399
+D+0.750L+0.750S+0.450W+H	0.453	0.439
+D+0.750L+0.750S+0.5250E+H	0.584	0.571
+0.60D+0.60W+0.60H	0.046	0.041
+0.60D+0.70E+0.60H	0.221	0.216
D Only	0.369	0.360
Lr Only	0.226	0.226
S Only	0.287	0.280
W Only	-0.292	-0.292
H Only		

- GRADE BEAM (WORST CASE)

ROOF DEAD LOAD:	20	PSF	TRIBUTARY WIDTH (ROOF) = 9'-0"
ROOF LIVE LOAD:	25	PSF	TRIBUTARY WIDTH (WALL) = 19'-0"
WALL LOAD:	20	PSF	
GRADE BEAM WIDTH:	1'-6"		

TOTAL LOAD = (20+25) PSF (9.0') + (20) PSF (19.0') = 700 PLF

ALLOWABLE = 1'-6" (2,000) = 3,000 PLF > 740 PLF **OK**

USE 1'-6" x X'-X" GRADE BEAM W/ (2) #5 TOP, BOT. & INT. W/ #3 STIRRUPS AT 18" O.C.

*REFER TO ATTACHED ENERCALC OUTPUT FOR THESE REACTIONS

- WOOD STUDS: LOAD CASE A

HEIGHT: 19’-0” TRIBUTARY WIDTH (SPACING) = 16” O.C.
LATERAL WIND LOAD:

PARAPET:	44.0 PSF (LOAD CASE A C&C PARAPET: ULT)	13.5 FT TO 19 FT
MAIN BLDG:	29.7 PSF (WALL ZONE 5 C&C: ULT.)	0 FT TO 13.5 FT

USE 2X6 STUDS @ 16” O.C. *

- WOOD STUDS: LOAD CASE B

HEIGHT: 19’-0” TRIBUTARY WIDTH (SPACING) = 16” O.C.
LATERAL WIND LOAD:

PARAPET:	48.1 PSF (LOAD CASE B C&C PARAPET: ULT)	13.5 FT TO 19 FT
MAIN BLDG:	29.7 PSF (WALL ZONE 5 C&C: ULT.)	0 FT TO 13.5 FT

USE 2X6 STUDS @ 16” O.C. *

- WOOD STUDS: LOAD CASE A

HEIGHT: 18’-3” TRIBUTARY WIDTH (SPACING) = 16” O.C.
LATERAL WIND LOAD:

PARAPET:	44.0 PSF (LOAD CASE A C&C PARAPET: ULT)	13 FT TO 18.25 FT
MAIN BLDG:	29.7 PSF (WALL ZONE 5 C&C: ULT.)	0 FT TO 13 FT

USE 2X6 STUDS @ 16” O.C. *

- WOOD STUDS: LOAD CASE B

HEIGHT: 18’-3” TRIBUTARY WIDTH (SPACING) = 16” O.C.
LATERAL WIND LOAD:

PARAPET:	48.1 PSF (LOAD CASE B C&C PARAPET: ULT)	13 FT TO 18.25 FT
MAIN BLDG:	29.7 PSF (WALL ZONE 5 C&C: ULT.)	0 FT TO 13 FT

USE 2X6 STUDS @ 16” O.C. *

*REFER TO ATTACHED ENERCALC OUTPUT FOR THESE REACTIONS

Wood Beam

DESCRIPTION: Load Case A: Wood Stud: 19'-0"

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Southern Pine
Wood Grade : No.2: 2"-4" Thick: 5"-6" Wide

Beam Bracing : Beam bracing is defined as a set spacing over all spans

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,400.0 psi
Fc - Perp	565.0 psi
Fv	175.0 psi
Ft	600.0 psi

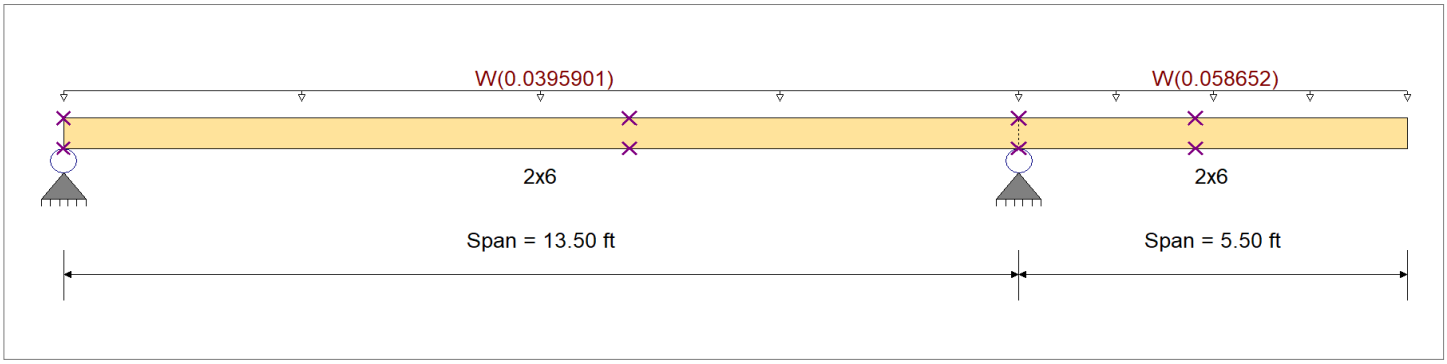
E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	34.330 pcf

Repetitive Member Stress Increase

Unbraced Lengths

First Brace starts at 8.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 8.0 ft



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1
Uniform Load : W = 0.02970 ksf, Tributary Width = 1.333 ft
Load for Span Number 2
Uniform Load : W = 0.0440 ksf, Tributary Width = 1.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.539 : 1	Maximum Shear Stress Ratio	=	0.130 : 1
Section used for this span		2x6	Section used for this span		2x6
fb: Actual	=	844.59 psi	fv: Actual	=	36.32 psi
Fb: Allowable	=	1,566.77 psi	Fv: Allowable	=	280.00 psi
Load Combination		+0.60W	Load Combination		+0.60W
Location of maximum on span	=	13.500 ft	Location of maximum on span	=	13.500 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.375 in	Ratio =	352 >=120
Max Upward Transient Deflection	0.000 in	Ratio =	0 <120
Max Downward Total Deflection	0.225 in	Ratio =	586 >=120
Max Upward Total Deflection	-0.000 in	Ratio =	15572 >=120

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
													0.00	0.00	0.00	0.00
Length = 7.994 ft	1			0.90	1.000	1.00	1.15	1.00	1.00	0.91			941.90	0.00	0.00	157.50
Length = 5.506 ft	1			0.90	1.000	1.00	1.15	1.00	1.00	0.95			987.58	0.00	0.00	157.50
Length = 2.489 ft	2			0.90	1.000	1.00	1.15	1.00	1.00	0.98			1018.02	0.00	0.00	157.50
Length = 3.011 ft	2			0.90	1.000	1.00	1.15	1.00	1.00	0.98			1013.16	0.00	0.00	157.50
+0.60W					1.000	1.00	1.15	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 7.994 ft	1	0.383	0.079	1.60	1.000	1.00	1.15	1.00	1.00	0.69	0.31	488.28	1274.66	0.12	21.98	280.00
Length = 5.506 ft	1	0.539	0.130	1.60	1.000	1.00	1.15	1.00	1.00	0.85	0.53	844.59	1566.77	0.20	36.32	280.00

Wood Beam

DESCRIPTION: Load Case A: Wood Stud: 19'-0"

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 2.489 ft	**2**	0.477	0.130	1.60	1.000	1.00	1.15	1.00	1.00	0.96	0.53	844.59	1771.69	0.19	36.32	280.00
Length = 3.011 ft	**2**	0.145	0.130	1.60	1.000	1.00	1.15	1.00	1.00	0.95	0.16	253.16	1745.80	0.11	36.32	280.00
+0.450W					1.000	1.00	1.15	1.00	1.00	0.95			0.00	0.00	0.00	0.00
Length = 7.994 ft	**1**	0.287	0.059	1.60	1.000	1.00	1.15	1.00	1.00	0.69	0.23	366.21	1274.66	0.09	16.49	280.00
Length = 5.506 ft	**1**	0.404	0.097	1.60	1.000	1.00	1.15	1.00	1.00	0.85	0.40	633.44	1566.77	0.15	27.24	280.00
Length = 2.489 ft	**2**	0.358	0.097	1.60	1.000	1.00	1.15	1.00	1.00	0.96	0.40	633.44	1771.69	0.15	27.24	280.00
Length = 3.011 ft	**2**	0.109	0.097	1.60	1.000	1.00	1.15	1.00	1.00	0.95	0.12	189.87	1745.80	0.08	27.24	280.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.4366	5.732		0.0000	0.000
W Only	2	0.3749	5.500	W Only	-0.0002	0.092

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	0.202	0.656	
Overall MINimum	0.202	0.656	
+0.60W	0.121	0.393	
+0.450W	0.091	0.295	
W Only	0.202	0.656	

Wood Beam

Lic. # : KW-06006725

DESCRIPTION: Load Case B: Wood Stud: 19'-0"

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Southern Pine
Wood Grade : No.2: 2"-4" Thick: 5"-6" Wide

Beam Bracing : Beam bracing is defined as a set spacing over all spans

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,400.0 psi
Fc - Perp	565.0 psi
Fv	175.0 psi
Ft	600.0 psi

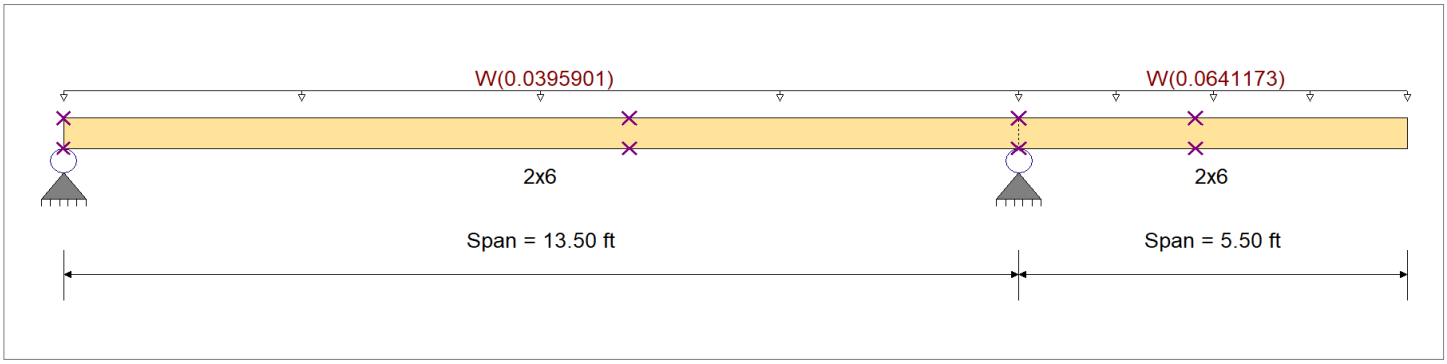
E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	34.330 pcf

Repetitive Member Stress Increase

Unbraced Lengths

First Brace starts at 8.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 8.0 ft



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1
Uniform Load : W = 0.02970 ksf, Tributary Width = 1.333 ft
Load for Span Number 2
Uniform Load : W = 0.04810 ksf, Tributary Width = 1.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.589 : 1	Maximum Shear Stress Ratio	=	0.127 : 1
Section used for this span		2x6	Section used for this span		2x6
fb: Actual	=	923.29 psi	fv: Actual	=	35.46 psi
Fb: Allowable	=	1,566.77 psi	Fv: Allowable	=	280.00 psi
Load Combination		+0.60W	Load Combination		+0.60W
Location of maximum on span	=	13.500 ft	Location of maximum on span	=	13.500 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.533 in	Ratio =	246 >=120
Max Upward Transient Deflection	0.000 in	Ratio =	0 <120
Max Downward Total Deflection	0.320 in	Ratio =	412 >=120
Max Upward Total Deflection	0.000 in	Ratio =	0 <120

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
													0.00	0.00	0.00	0.00
Length = 7.994 ft	1			0.90	1.000	1.00	1.15	1.00	1.00	0.91			941.90	0.00	0.00	157.50
Length = 5.506 ft	1			0.90	1.000	1.00	1.15	1.00	1.00	0.95			987.58	0.00	0.00	157.50
Length = 2.489 ft	2			0.90	1.000	1.00	1.15	1.00	1.00	0.98			1018.02	0.00	0.00	157.50
Length = 3.011 ft	2			0.90	1.000	1.00	1.15	1.00	1.00	0.98			1013.16	0.00	0.00	157.50
+0.60W					1.000	1.00	1.15	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 7.994 ft	1	0.360	0.069	1.60	1.000	1.00	1.15	1.00	1.00	0.69	0.29	459.06	1274.66	0.11	19.36	280.00
Length = 5.506 ft	1	0.589	0.127	1.60	1.000	1.00	1.15	1.00	1.00	0.85	0.58	923.29	1566.77	0.20	35.46	280.00

Wood Beam

DESCRIPTION: Load Case B: Wood Stud: 19'-0"

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
Length = 2.489 ft	**2**	0.521	0.127	1.60	1.000	1.00	1.15	1.00	1.00	0.96	0.58	923.29	1771.69	0.20	35.46	280.00
Length = 3.011 ft	**2**	0.159	0.127	1.60	1.000	1.00	1.15	1.00	1.00	0.95	0.17	276.75	1745.80	0.12	35.46	280.00
+0.450W					1.000	1.00	1.15	1.00	1.00	0.95			0.00	0.00	0.00	0.00
Length = 7.994 ft	**1**	0.270	0.052	1.60	1.000	1.00	1.15	1.00	1.00	0.69	0.22	344.30	1274.66	0.08	14.52	280.00
Length = 5.506 ft	**1**	0.442	0.095	1.60	1.000	1.00	1.15	1.00	1.00	0.85	0.44	692.47	1566.77	0.15	26.60	280.00
Length = 2.489 ft	**2**	0.391	0.095	1.60	1.000	1.00	1.15	1.00	1.00	0.96	0.44	692.47	1771.69	0.15	26.60	280.00
Length = 3.011 ft	**2**	0.119	0.095	1.60	1.000	1.00	1.15	1.00	1.00	0.95	0.13	207.56	1745.80	0.09	26.60	280.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.3848	5.581		0.0000	0.000
W Only	2	0.5333	5.500		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	0.195	0.692	
Overall MINimum	0.195	0.692	
+0.60W	0.117	0.415	
+0.450W	0.088	0.311	
W Only	0.195	0.692	

Wood Beam

Lic. # : KW-06006725 RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: Load Case A: Wood Stud: 18'-3"

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Southern Pine
Wood Grade : No.2: 2"-4" Thick: 5"-6" Wide

Beam Bracing : Beam bracing is defined as a set spacing over all spans

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,400.0 psi
Fc - Perp	565.0 psi
Fv	175.0 psi
Ft	600.0 psi

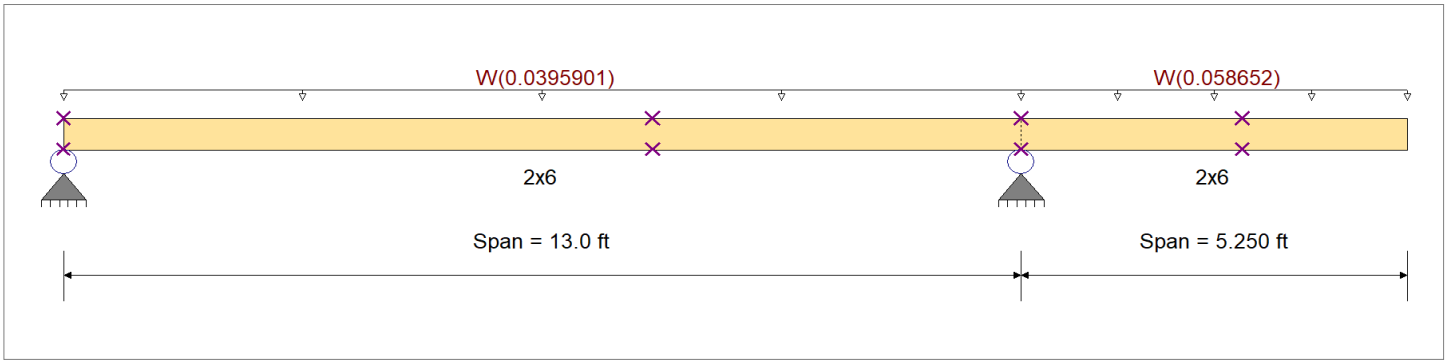
E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	34.330 pcf

Repetitive Member Stress Increase

Unbraced Lengths

First Brace starts at 8.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 8.0 ft



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1
Uniform Load : W = 0.02970 ksf, Tributary Width = 1.333 ft
Load for Span Number 2
Uniform Load : W = 0.0440 ksf, Tributary Width = 1.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.477 : 1	Maximum Shear Stress Ratio	=	0.118 : 1
Section used for this span		2x6	Section used for this span		2x6
fb: Actual	=	769.55 psi	fv: Actual	=	32.97 psi
Fb: Allowable	=	1,613.04 psi	Fv: Allowable	=	280.00 psi
Load Combination		+0.60W	Load Combination		+0.60W
Location of maximum on span	=	13.000 ft	Location of maximum on span	=	12.564 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.384 in	Ratio =	406 >=120
Max Upward Transient Deflection	0.000 in	Ratio =	0 <120
Max Downward Total Deflection	0.230 in	Ratio =	677 >=120
Max Upward Total Deflection	-0.000 in	Ratio =	316862 >=120

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
													0.00	0.00	0.00	0.00
Length = 7.989 ft	1			0.90	1.000	1.00	1.15	1.00	1.00	0.91			942.04	0.00	0.00	157.50
Length = 5.011 ft	1			0.90	1.000	1.00	1.15	1.00	1.00	0.96			993.61	0.00	0.00	157.50
Length = 2.992 ft	2			0.90	1.000	1.00	1.15	1.00	1.00	0.98			1013.35	0.00	0.00	157.50
Length = 2.258 ft	2			0.90	1.000	1.00	1.15	1.00	1.00	0.99			1019.99	0.00	0.00	157.50
+0.60W					1.000	1.00	1.15	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 7.989 ft	1	0.359	0.069	1.60	1.000	1.00	1.15	1.00	1.00	0.69	0.29	457.96	1275.33	0.11	19.41	280.00
Length = 5.011 ft	1	0.477	0.118	1.60	1.000	1.00	1.15	1.00	1.00	0.88	0.48	769.55	1613.04	0.18	32.97	280.00

Wood Beam

Lic. # : KW-06006725 RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: Load Case A: Wood Stud: 18'-3"

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 2.992 ft	**2**	0.441	0.118	1.60	1.000	1.00	1.15	1.00	1.00	0.95	0.48	769.55	1746.88	0.17	32.97	280.00
Length = 2.258 ft	**2**	0.080	0.118	1.60	1.000	1.00	1.15	1.00	1.00	0.97	0.09	142.40	1781.31	0.08	32.97	280.00
+0.450W					1.000	1.00	1.15	1.00	1.00	0.97			0.00	0.00	0.00	0.00
Length = 7.989 ft	**1**	0.269	0.052	1.60	1.000	1.00	1.15	1.00	1.00	0.69	0.22	343.47	1275.33	0.08	14.56	280.00
Length = 5.011 ft	**1**	0.358	0.088	1.60	1.000	1.00	1.15	1.00	1.00	0.88	0.36	577.16	1613.04	0.14	24.73	280.00
Length = 2.992 ft	**2**	0.330	0.088	1.60	1.000	1.00	1.15	1.00	1.00	0.95	0.36	577.16	1746.88	0.13	24.73	280.00
Length = 2.258 ft	**2**	0.060	0.088	1.60	1.000	1.00	1.15	1.00	1.00	0.97	0.07	106.80	1781.31	0.06	24.73	280.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.3838	5.592		0.0000	0.000
W Only	2	0.2915	5.250	W Only	-0.0007	0.176

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	0.195	0.627	
Overall MINimum	0.195	0.627	
+0.60W	0.117	0.376	
+0.450W	0.088	0.282	
W Only	0.195	0.627	

Wood Beam

Lic. # : KW-06006725 — RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: Load Case B: Wood Stud: 18'-3"

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Southern Pine
Wood Grade : No.2: 2"-4" Thick: 5"-6" Wide

Beam Bracing : Beam bracing is defined as a set spacing over all spans

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,400.0 psi
Fc - Perp	565.0 psi
Fv	175.0 psi
Ft	600.0 psi

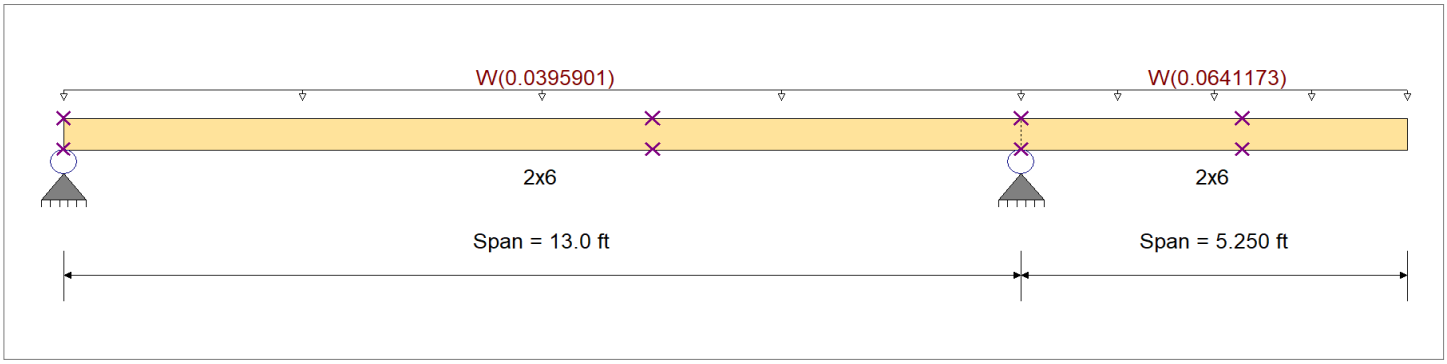
E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	34.330 pcf

Repetitive Member Stress Increase

Unbraced Lengths

First Brace starts at 8.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 8.0 ft



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1
Uniform Load : W = 0.02970 ksf, Tributary Width = 1.333 ft
Load for Span Number 2
Uniform Load : W = 0.04810 ksf, Tributary Width = 1.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.522 : 1	Maximum Shear Stress Ratio	=	0.120 : 1
Section used for this span		2x6	Section used for this span		2x6
fb: Actual	=	841.26 psi	fv: Actual	=	33.64 psi
Fb: Allowable	=	1,613.04 psi	Fv: Allowable	=	280.00 psi
Load Combination		+0.60W	Load Combination		+0.60W
Location of maximum on span	=	13.000 ft	Location of maximum on span	=	13.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.424 in	Ratio =	296 >=120
Max Upward Transient Deflection	0.000 in	Ratio =	0 <120
Max Downward Total Deflection	0.254 in	Ratio =	494 >=120
Max Upward Total Deflection	0.000 in	Ratio =	0 <120

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
													0.00	0.00	0.00	0.00
Length = 7.989 ft	1			0.90	1.000	1.00	1.15	1.00	1.00	0.91			942.04	0.00	0.00	157.50
Length = 5.011 ft	1			0.90	1.000	1.00	1.15	1.00	1.00	0.96			993.61	0.00	0.00	157.50
Length = 2.992 ft	2			0.90	1.000	1.00	1.15	1.00	1.00	0.98			1013.35	0.00	0.00	157.50
Length = 2.258 ft	2			0.90	1.000	1.00	1.15	1.00	1.00	0.99			1019.99	0.00	0.00	157.50
+0.60W					1.000	1.00	1.15	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 7.989 ft	1	0.338	0.067	1.60	1.000	1.00	1.15	1.00	1.00	0.69	0.27	431.17	1275.33	0.10	18.78	280.00
Length = 5.011 ft	1	0.522	0.120	1.60	1.000	1.00	1.15	1.00	1.00	0.88	0.53	841.26	1613.04	0.19	33.64	280.00

Wood Beam

Lic. # : KW-06006725 RONALD A. ROBERTS ASSOCIATES

DESCRIPTION: Load Case B: Wood Stud: 18'-3"

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 2.992 ft	**2**	0.482	0.120	1.60	1.000	1.00	1.15	1.00	1.00	0.95	0.53	841.26	1746.88	0.19	33.64	280.00
Length = 2.258 ft	**2**	0.087	0.120	1.60	1.000	1.00	1.15	1.00	1.00	0.97	0.10	155.67	1781.31	0.09	33.64	280.00
+0.450W					1.000	1.00	1.15	1.00	1.00	0.97			0.00	0.00	0.00	0.00
Length = 7.989 ft	**1**	0.254	0.050	1.60	1.000	1.00	1.15	1.00	1.00	0.69	0.20	323.38	1275.33	0.08	14.08	280.00
Length = 5.011 ft	**1**	0.391	0.090	1.60	1.000	1.00	1.15	1.00	1.00	0.88	0.40	630.95	1613.04	0.14	25.23	280.00
Length = 2.992 ft	**2**	0.361	0.090	1.60	1.000	1.00	1.15	1.00	1.00	0.95	0.40	630.95	1746.88	0.14	25.23	280.00
Length = 2.258 ft	**2**	0.066	0.090	1.60	1.000	1.00	1.15	1.00	1.00	0.97	0.07	116.75	1781.31	0.07	25.23	280.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.3399	5.374		0.0000	0.000
W Only	2	0.4238	5.250		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	0.189	0.662	
Overall MINimum	0.189	0.662	
+0.60W	0.114	0.397	
+0.450W	0.085	0.298	
W Only	0.189	0.662	

USE SDS25412 SCREWS

- MINIMUM EDGE DISTANCE: C_Δ = 0.5 FOR 2D AND C_Δ = 1.0 FOR 4D (TABLE 12.5.1A, NDS)

  D = SCREW DIAMETER = 0.25”
  0.25” x 2 = 0.5”
  0.2” x 4 = 1.0”

  IF INSTALLED IN THE MIDDLE OF THE 2X EDGE DISTANCE = 0.75 INCHES OR 3D. INTEREPLATING BETWEEN 2D AND 4D, C_Δ = 0.75 INCHES

- SPACING REQUIREMENTS: 4D FOR C_Δ = 1.0 (TABLE 12.5.1B, NDS) = 4 (0.25”) = 1”

- LOADED EDGE 4D = 1.0 INCH (CHECK PER TABLE 12.5.1C)

- MINIMUM PENETRATION: 8D = 8 x 0.25’’ = 2 INCHES
  SDS25412 = 4 ½’’ – 1 ½’’ = 3’’ > 2 INCHES **OK**

- SHEAR CAPACITY OF SINGLE SDS25412 SCREW

  $Z' = Z(C_D)(C_m)(C_E)(C_g)(C_\Delta)$

  Z = 350 LBS (PER ESR-2236)
  C_D = 1.0
  C_m = 1.0
  C_E = 1.0
  C_g = 1.0
  C_Δ = 0.75

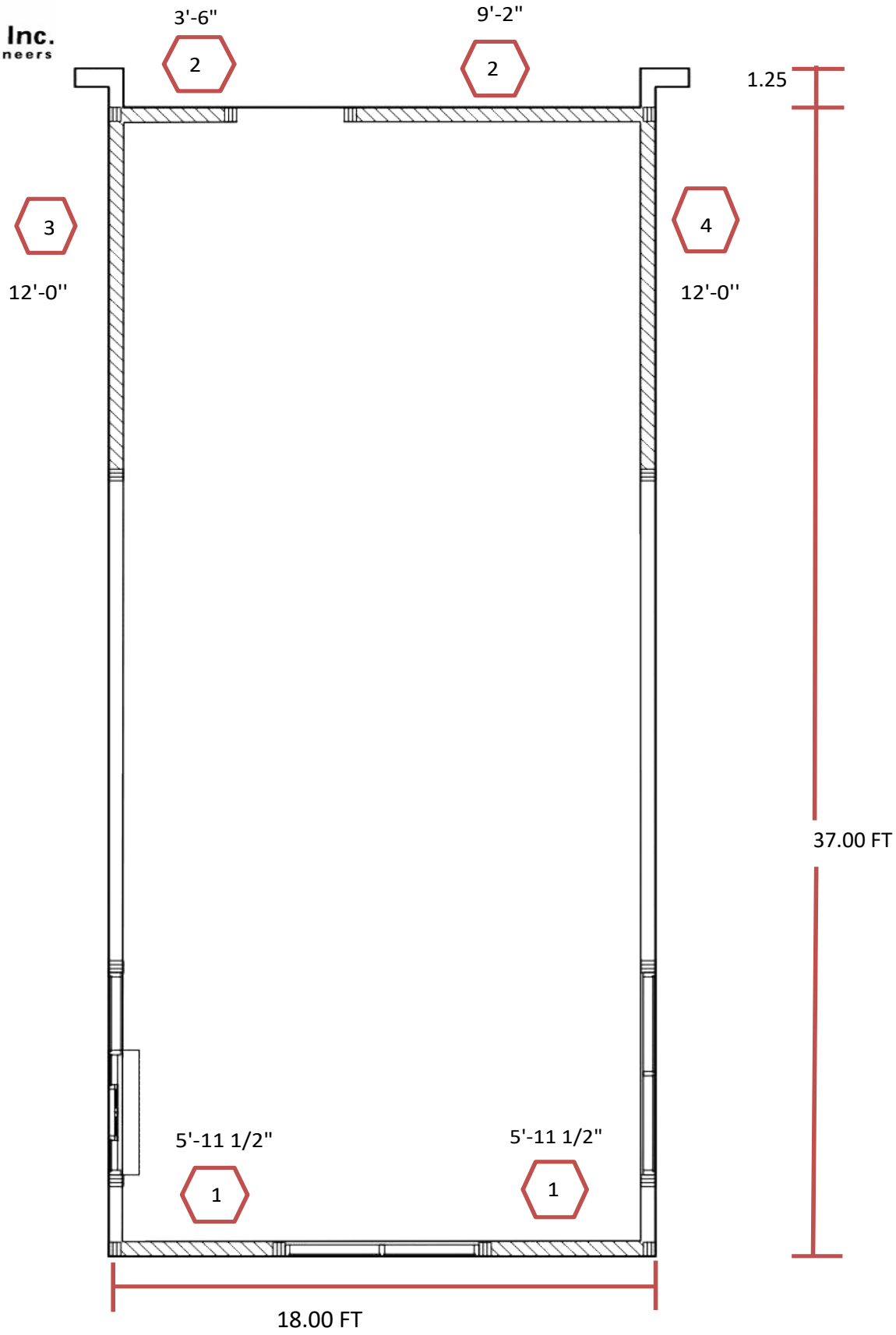
  Z’ = 350 LBS(1.0)(1.0)(1.0)(1.0)(0.75) = 262 LBS

- REQUIRED LOAD = 884 LBS (REFER TO WORST CASE JOIST CALCULATION)

- REQUIRED NUMBER OF SCREWS

  (884 LBS)/(262 LBS/SCREW) = 3.37 = 4 SCREWS MINIMUM
  (5 SCREWS USED IN TYPICAL DESIGN)
  USE: (5) SDS25412 SCREWS/STUD

*REFER TO ATTACHED ENERCALC OUTPUT FOR THESE REACTIONS


3'-6"
9'-2"
2
2
1.25
3
4
12'-0"
12'-0"
37.00 FT
5'-11 1/2"
5'-11 1/2"
1
1
18.00 FT

EDGE DISTANCE:

a = 10% LEAST HORIZONTAL DIMENSION OR .4 DECK HEIGHT BUT NOT LESS THAN 3 FT

		a
LEAST HORIZONTAL DIMENSION:	18.00'	1.8 FT
DECK HEIGHT:	13.00'	5.2 FT
	USE:	5.2 FT

AVERAGE TOP OF PARAPET : 18.50 FT

	Weights of Materials Per ASCE 7-10 Table C3-1						
EXTERIOR WALLS:			LINEAR FEET		HEIGHT		
2X6 STUD	2.20 PSF		112.50 FT	X	18.5 FT	=	2081.3 SF
INSULATION	0.70 PSF						
5/8" GYP	2.50 PSF						
1/2 PLYWOOD SHEATHING	1.60 PSF						2081.3 SF
ADHERED STONE	10.00 PSF						
2" EIFS	1.50 PSF						
TOTAL:	18.50 PSF	X	2081.3 SF	=	38503 LBS		
INTERIOR WALLS:					ESTIMATED WALL	=	50.0 FT
2X6 STUD WALLS	2.20 PSF				HEIGHT	=	12.0 FT
5/8" GYP BOARD X 2 SIDES	5.50 PSF						600.0 SF
TOTAL:	7.70 PSF	X	600.0 FT	=	4,620 LBS		
ROOF:			WIDTH		LENGTH		
SINGLE PLY MEMBRANE	0.70 PSF		18.0 FT	X	37.0 FT	=	666.0 SF
3/4" PLYWOOD ROOF DECK	2.40 PSF						
MECHANICAL/LIGHTING	4.00 PSF						
MECHANICAL/LIGHTING	4.00 PSF						
RIGID INSULATION	4.15 PSF						
SPRINKLER	2.00 PSF						
GYPSUM CEILING	2.75 PSF						
TOTAL:	20.0 PSF	X	666.0 SF	=	13,320 LBS		
MECHANICAL UNITS:							
TOTAL OF ALL UNITS:	1,000 LBS						

TOTAL WEIGHT: **57443 LBS**

SIMPLIFIED DESIGN PROCEDURE CHECKLIST (12.14.1.1)

1) THE STRUCUTRE SHALL QUALIFY FOR RISK CATEGORY I OR II IN ACCORDANCE WITH ☑
2) THE SITE CLASS, DEFINED IN CHAPTER 20, SHALL NOT BE CLASS E OR F. ☑
3) THE STRUCTURE SHALL NOT EXCEED THREE STORIES ABOVE GRADE PLANE. ☑
4) THE SEISMIC FORCE- RESISTING SYSTEM SHALL BE EITHER A BEARING WALL SYSTEM OR BUILDING FRAME SYSTEM, AS INDICATED IN TABLE 12.14-1. ☑
5) THE STRUCTURE SHALL HAVE AT LEAST TWO LINES OF LATERAL RESISTANCE IN EACH OF THE TWO MAJOR AXIS DIRECTIONS. ☑
6) AT LEAST ONE LINE OF RESISTANCE SHALL BE PROVIDED ON EACH SIDE OF THE CENTER OF MASS IN EACH DIRECTION. ☑
7) FOR STRUCUTRES WITH FLEXIBLE DIAPHRAGMS, OVERHANGS BEYONG THE OUSDIE LINE OF SHEAR WALLS OR BRACED SHALL SATISFY THE FOLLOWING: ☑

$$a \leq d/5$$

a = the distance perpendicular to the forces being considered from the extreme edge of the diaphragm to the line of vertical resistance closest to that edge
d = the depth of the diaprgam parallel to the forces being considered at the line of the vertical resistance closest to the edge.

8) FOR BUILDINGS WITH A DIAPHRAGM THAT IS NOT FLEXIBLE, THE DISTANCE BETWEENT HE CENTER OF RIGIDITY AND THE CENTER OF MASS PARALLEL TO EACH MAJOR AXIS SHALL NOT EXCEED 15 PERCENT OF THE GREATEST WIDTH OF THE n/a
9) LINES OF RESISTANCE OF THE SEISMIC FORCE-RESISTING SYSTEM SHALL BE ORIENTED AT ANGLE OF NO MORE THAN 15 DEGREES FROM ALIGMENT WITH THE MAJOR ORTHOGONAL HORIZONTAL AXES OF THE BUILDING. ☑
10) THE SIMPLIFIED DESIGN PROCEDURE SHALL BE USED FOR EACH MAJOR ORTHOGONAL HORIZONTAL AAXIS DIRECTION OF THE BUILDING. ☑
11) SYSTEM IRREGULARITIES CASUED BY IN-PLANE OR OUT-OF-PLANE OFFSETS OF LATERAL FORCE- RESISTING ELEMENTS SHALL NOT BE PERMITTED. ☑

EXCEPTION: OUT-OF-PLANE AND IN-PLANE OFFSETS OF SHEAR WALLS ARE PERMITTED IN TWO-STORY BUILDINGS OF LIGHT-FRAME CONSTRUCTION PROVIDED THAT THE FRAMING SUPPORTING THE UPPER WALL IS DESINGED FOR SEISMIC FORCE EFFECTS FROM OVERTURNING OF THE WALL AMPLIFIED BY A FACTOR OF 2.5.

12) THE LATERAL LOAD RESISTANCE OF ANY STORY SHALL NOT BE LESS THAN 80 PERCENT OF THE STORY ABOVE. ☑

DETERMINE SEISMIC BASE SHEAR:

$$V = \frac{FS_{ds}W}{R}$$ Equation 12.14-11

F = 1.00 Per ASCE 7-10, 12.14.8.1

Sds = *0.10*

R = 6.50

W = 57,443 LBS FROM CALCULATION ABOVE

R=6.5 (Light framed wood walls sheathed in plywood)

TOTAL BASE SHEAR = 0,875 LBS

DETERMINE SHEAR TO WALLS

SHEARWALL 1

PERCENT TO WALL: 50.00% X 0,875 LBS = 437 LBS

SHEARWALL 2

PERCENT TO WALL: 50.00% X 0,875 LBS = 437 LBS

SHEARWALL 3

PERCENT TO WALL: 50.00% X 0,875 LBS = 437 LBS

SHEARWALL 4

PERCENT TO WALL: 50.00% x 0,875 LBS = 437 LBS

AVERAGE TOP OF SHEAR WALL : 13 FT
TOP OF PARAPET (WORST CASE) 19 FT

MAIN BUILDING WIND PRESSURE :	21.0	PSF	END ZONE (A)
	13.9	PSF	INTERIOR ZONE (C)
PARAPET WIND PRESSURE :	30.3	PSF	(WINDWARD)
	20.2	PSF	(LEEWARD)
	50.5	PSF	TOTAL PARAPET

W = 21.0 PSF (13FT/2) + 50.3 PSF (19FT - 13FT) = 439.5 PLF (ZONE A)

W = 13.9 PSF (13FT/2) + 50.3 PSF (19FT - 13FT) = 393.4 PLF (ZONE C)

DETERMINE SHEAR TO SIDES

SHEARWALL 1
SHEAR (V) = 393.4 PLF (37FT /2- 5.2FT) + 439.5 PLF (5.2 FT) = 7,517 LBS

SHEARWALL 2
SHEAR (V) = 393.4 PLF (37FT /2- 5.2FT) + 439.5 PLF (5.2 FT+1.33FT) = 8,066 LBS

SHEARWALL 3
SHEAR (V) = 393.4 PLF (18FT /2- 5.2FT) + 439.5 PLF (5.2 FT) = 3,780 LBS

SHEARWALL 4
SHEAR (V) = 393.4 PLF (18FT /2- 5.2FT) + 439.5 PLF (5.2 FT) = 3,780 LBS

ULTIMATE VALUES SHOWN

AVERAGE TOP OF SHEAR WALL : 13 FT
TOP OF PARAPET (WORST CASE) : 19 FT

MINIMUM

MAIN BUILDING WIND PRESSURE :	16.0	PSF	END ZONE (A)
	16.0	PSF	INTERIOR ZONE (C)
PARAPET WIND PRESSURE :	30.3	PSF	(WINDWARD)
	20.2	PSF	(LEEWARD)
	50.5	PSF	TOTAL PARAPET

W = 16 PSF (13FT/2) + 50.3 PSF (19FT - 13FT) = 407.0 PLF

DETERMINE SHEAR TO SIDES

SHEARWALL 1
SHEAR (V) = 407.0 PLF (37 FT /2) = 7,530 LBS

SHEARWALL 2
SHEAR (V) = 407.0 PLF (37 FT /2 + 1.33FT) = 8,038 LBS

SHEARWALL 3
SHEAR (V) = 407.0 PLF (18 FT /2) = 3,663 LBS

SHEARWALL 4
SHEAR (V) = 407.0 PLF (18 FT /2) = 3,663 LBS

	SHEAR LOADS			
SHEAR WALL	SEISMIC	WIND PRESSURE	MINIMUM WIND PRESSURE	USE
WALL 1	437 LBS	7,517 LBS	7,530 LBS	7,530 LBS
WALL 2	437 LBS	8,066 LBS	8,038 LBS	8,066 LBS
WALL 3	437 LBS	3,780 LBS	3,663 LBS	3,780 LBS
WALL 4	437 LBS	3,780 LBS	3,663 LBS	3,780 LBS

ULTIMATE VALUES SHOWN

WALL 1

		(ULTIMATE)			(ASD)
SHEAR (V)	=	7,530 LBS	X	.6 =	4518 LBS
NAILING LENGTH (LN)	=	10.9 FT			
ANCHOR LENGTH (LA)	=	8.9 FT			
HEIGHT (H)	=	13.5 FT			

NAILING REQUIREMENTS

$$V/LN = \frac{4518\ PLF}{10.9\ FT} = 414\ LBS$$

USE 10d NAILS @ 6" O.C. (CAPACITY = 435 PLF)

NDS15 : Table 4.3A USING 15/32 PANEL SHEATING

HOLDOWN REQUIREMENTS

$$(VH)/LA = \frac{60989\ LBS\text{-}FT}{8.9\ FT} = 6837\ LBS$$

USE HDU8-SDS2.5 (CAPACITY = 7,870 LBS)

2019 - 2020 SIMPSON CATALOG

ANCHOR REQUIREMENTS (AR)

$$AR = V/LA = \frac{7530\ LBS}{8.9\ FT} = 844\ PLF$$

PER NDS 2015 TABLE 12E: 5/8" BOLT WITH 1.5" THICK DOUGLAS FIR-LARCH SIDE MEMBER ALLOWABLE SHEAR WITH WIND LOAD DURATION FACTOR= 1.6*930 LBS =

5/8" DIA. ANCHOR ALLOW. LOAD: 1488 LBS/ANCHOR

$$\frac{(A.B.ALLOW * 12INCHES/FT)}{AR} = \frac{1488\ LBS * 12\ IN/FT}{844\ LBS/ANCHOR} = 21\ IN$$

USE 5/8" DIAMETER ANCHORS AT 16" O.C

ALLOWABLE VALUES SHOWN

WALL 2

		(ULTIMATE)			(ASD)
SHEAR (V)	=	8,066 LBS	X	.6 =	4,840 LBS
NAILING LENGTH (LN)	=	12.7 FT			
ANCHOR LENGTH (LA)	=	10.7 FT			
HEIGHT (H)	=	13.0 FT			

NAILING REQUIREMENTS

$$V/LN = \frac{4840\ PLF}{12.7\ FT} = 382\ LBS$$

USE 10d NAILS @ 6" O.C. (CAPACITY = 435 PLF)

NDS15 : Table 4.3A USING 15/32 PANEL SHEATING

HOLDOWN REQUIREMENTS

$$(VH)/LA = \frac{62917\ LBS\text{-}FT}{10.7\ FT} = 5897\ LBS$$

USE HDU8-SDS2.5 (CAPACITY = 7,870 LBS)

2019 - 2020 SIMPSON CATALOG

ANCHOR REQUIREMENTS (AR)

$$AR = V/LA = \frac{8066\ LBS}{10.7\ FT} = 756\ PLF$$

PER NDS 2015 TABLE 12E: 5/8" BOLT WITH 1.5" THICK DOUGLAS FIR-LARCH SIDE MEMBER ALLOWABLE SHEAR WITH WIND LOAD DURATION FACTOR= 1.6x930 LBS =

5/8" DIA. ANCHOR ALLOW. LOAD: 1488 LBS/ANCHOR

$$\frac{(A.B.ALLOW * 12INCHES/FT)}{AR} = \frac{1488\ LBS * 12\ IN/FT}{756\ LBS/ANCHOR} = 24\ IN$$

USE 5/8" DIAMETER ANCHORS AT 24" O.C.

ALLOWABLE VALUES SHOWN

WALL 3

		(ULTIMATE)			(ASD)
SHEAR (V)	=	3,780 LBS	X	.6 =	2,268 LBS
NAILING LENGTH (LN)	=	12.0 FT			
ANCHOR LENGTH (LA)	=	11.0 FT			
HEIGHT (H)	=	12.5 FT			

NAILING REQUIREMENTS

$$V/LN = \frac{2268\ PLF}{12.0\ FT} = 189\ LBS$$

USE 10d NAILS @ 6" O.C. (CAPACITY = 435 PLF)

NDS15 : Table 4.3A USING 15/32 PANEL SHEATING

HOLDOWN REQUIREMENTS

$$(VH)/LA = \frac{28351\ LBS\text{-}FT}{11.0\ FT} = 2577\ LBS$$

USE HDU2-SDS2.5 (CAPACITY = 3,075 LBS)

2019 - 2020 SIMPSON CATALOG

ANCHOR REQUIREMENTS (AR)

$$AR = V/LA = \frac{3780\ LBS}{11.0\ FT} = 344\ PLF$$

PER NDS 2015 TABLE 12E: 5/8" BOLT WITH 1.5" THICK DOUGLAS FIR-LARCH SIDE MEMBER ALLOWABLE SHEAR WITH WIND LOAD DURATION FACTOR= 1.6x930 LBS =

5/8" DIA. ANCHOR ALLOW. LOAD: 1488 LBS/ANCHOR

$$\frac{(A.B.ALLOW * 12INCHES/FT)}{AR} = \frac{1488\ LBS * 12\ IN/FT}{344\ LBS/ANCHOR} = 52\ IN$$

USE 5/8" DIAMETER ANCHORS AT 32" O.C.

ALLOWABLE VALUES SHOWN

WALL 4

		(ULTIMATE)			(ASD)
SHEAR (V)	=	3,780 LBS	X	.6 =	2,268 LBS
NAILING LENGTH (LN)	=	12.0 FT			
ANCHOR LENGTH (LA)	=	11.0 FT			
HEIGHT (H)	=	12.5 FT			

NAILING REQUIREMENTS

$$V/LN = \frac{2268\ PLF}{12.0\ FT} = 189\ LBS$$

USE 10d NAILS @ 6" O.C. (CAPACITY = 435 PLF)

NDS15 : Table 4.3A USING 15/32 PANEL SHEATING

HOLDOWN REQUIREMENTS

$$(VH)/LA = \frac{28351\ LBS\text{-}FT}{11.0\ FT} = 2577\ LBS$$

USE HDU2-SDS2.5 (CAPACITY = 3,075 LBS)

2019 - 2020 SIMPSON CATALOG

ANCHOR REQUIREMENTS (AR)

$$AR = V/LA = \frac{3780\ LBS}{11.0\ FT} = 344\ PLF$$

PER NDS 2015 TABLE 12E: 5/8" BOLT WITH 1.5" THICK DOUGLAS FIR-LARCH SIDE MEMBER ALLOWABLE SHEAR WITH WIND LOAD DURATION FACTOR= 1.6x930 LBS =

5/8" DIA. ANCHOR ALLOW. LOAD: 1488 LBS/ANCHOR

$$\frac{(A.B.ALLOW * 12INCHES/FT)}{AR} = \frac{1488\ LBS * 12\ IN/FT}{344\ LBS/ANCHOR} = 52\ IN$$

USE 5/8" DIAMETER ANCHORS AT 32" O.C.

ALLOWABLE VALUES SHOWN

DIAPHARGM

WALL 1 &2: (WORST CASE)	8,066 LBS/	18 FT =	448.13	PLF	
					CASE 1: LRFD : 640 PLF
WALL 3 & 4	3,780 LBS/	37 FT =	102.17	PLF	
					CASE 3 LRFD : 480 PLF

USE 10d NAILS @ 6 O.C. AT DIAPHRAGM BOUNDARIES (UN-BLOCKED)

CHORD

Chord Along Wall 1 and 2

$$\left(\frac{3{,}780 \text{ LBS} + 3{,}780 \text{ LBS}}{8 \text{ X } 37 \text{ FT}}\right) \times 18 \text{ FT} = 460 \text{ LBS} \quad (.6) = 275.8 \text{ LBS}$$

USE 2X8 (MIN) LEDGER

FT = 575 TALLOW = 575*10.875 = 6,253 LBS > 460 LBS

Chord Along Wall 3 and 4

$$\left(\frac{8{,}066 \text{ LBS} + 7{,}517 \text{ LBS}}{8 \text{ X } 18 \text{ FT}}\right) \times 37 \text{ FT} = 4{,}004 \text{ LBS} \quad (.6) = 2402 \text{ LBS}$$

USE 2X12 (MIN) LEDGER

FT = 575 TALLOW = 575*16.875 = 9,703 LBS > 2,402 LBS

USE SIMPSON MSTI48 STRAP @ SPLICE

CAPACITY : 5,070 LBS

ULTIMATE VALUES SHOWN