



San Mina Engineering, Inc.

October 16, 2025

www.sanminaeng.com

24310 Moulton Pkwy, Ste. O-105, Laguna Hills, CA 92637

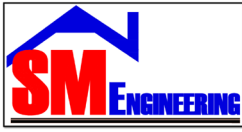
PROJECT NO. 25-10281

Phone (949) 446-1417 & (949) 536-9645

NEW BUILD STRUCTURAL ANALYSIS

FOR

**CITY OF UPLAND WATER DISTRICT
SE CORNER OF E 20TH ST & FLOWER CT
UPLAND, CA
APN 1044-091-22**



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Maintenance Building.....	
Trash Enclosure	

**E 20th St,
Upland, CA 91784**

STRUCTURAL CALCULATIONS

Main Building

Date :10/14/25

Rev. : 0

CALCULATION INDEX

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GRAVITY LOADS

ROOF LOADS

DEAD LOAD

Roofing Shingles Asphalt (1/4 in)	2.00	psf
Sheathing 15/32" Plywood	2.81	psf
Framing 2x12 @ 16" O.C. DF-Larch	3.15	psf
Drywall Finish	3.00	psf
Insulation Fiber Glass Loose	0.56	psf
Miscellaneous	1.48	psf

ROOF DEAD LOAD TOTAL	13.00	psf
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315 ROOF LIVE LOAD TOTAL	20.00	psf
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OPEN DECK

DEAD LOAD

Flooring Hardwood (1 in)	3.00	psf
Sheathing 3/4" T&G	2.25	psf
Framing 2x12 @ 16" O.C. DF-Larch	3.15	psf
Drywall Finish	3.00	psf
Miscellaneous	1.60	psf

DECK DEAD LOAD TOTAL	13.00	psf
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DECK LIVE LOAD TOTAL	100.00	psf
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CEILING LOADS

DEAD LOAD

Framing 2x 8 @ 16" O.C. DF-Larch	2.10	psf
Drywall Finish	3.00	psf
Miscellaneous	1.00	psf

CEILING DEAD LOAD TOTAL	6.00	psf
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CEILING LIVE LOAD TOTAL	20.00	psf
--------------------------------	--------------	------------

EXTERIOR WALL

Wall Plaster (1 in) on Metal Lath	10.00	psf
Sheathing 15/32" Plywood	2.81	psf
Wall Wood Studs 2x6 @ 16" O.C. DF-Larch	1.58	psf
Insulation Fiber Glass Loose	0.50	psf
Drywall Finish	3.00	psf
EXTERIOR WALL LOAD	18.00	psf

INTERIOR WALL

Drywall Finish	3.00	psf
Wall Wood Studs 2x4 @ 16" O.C. DF-Larch	1.05	psf
Sheathing 15/32" Plywood	2.81	psf
Drywall Finish	3.00	psf
INTERIOR WALL LOAD	10.00	psf

BUILDING WEIGHT

DESIGN CRITERIA

Code Used : 2022 CBC
Building Code : 2021 IBC
LACity Requirement : Yes

Concrete F'c : 2,500 psi
Steel Fy : 60,000 psi

SOILS REPORT

NO SOILS REPORT

Bearing Capacity : 1,500 psf

SEISMIC DESIGN PARAMETERS

Short Period, Fa : 1.2 0.2 sec Fig 1613.5(1)
Long Period, Fv : 1.5 1.0 sec Fig 1613.5(2)
Response Spectral : 198.00% 0.2 sec Fig 1613.5(3)
Response Spectral : 67.00% 1.0 sec Fig 1613.5(4)

Latitude : 34.135827
Longitude : -117.646592
Site Class : D

BUILDING SUMMARY

Level	Height ft	Elevation	Unit Weight psf	
			Dead	Live
ROOF	8.50	8.50	13.00	20.00

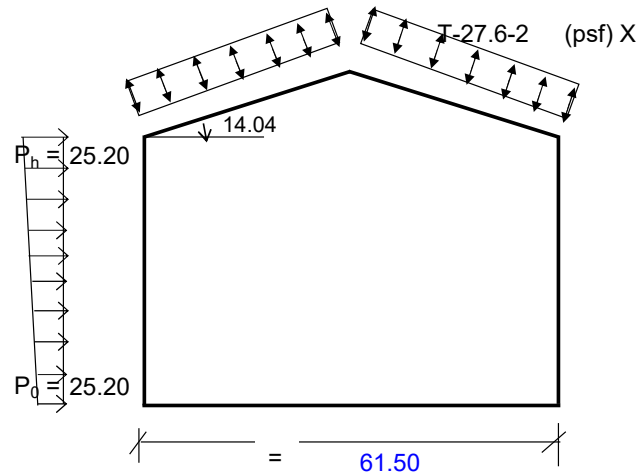
Wall Type	Weight psf
Exterior	18.00
Interior	10.00

BUILDING WEIGHT

ROOF

Area Type	L (ft)	W (ft)	H (ft)	Area (ft^2)	Dead Load (psf)	Weight (kips)
Roof	61.50	115.00		5,468.8	13.00	71.11
Ext. Wall	123.00	230.00	4.25		18.00	27.00
Int. Wall	246.00	172.50	4.25		10.00	17.79
TOTAL ROOF AREA / LOAD				5,468.8		115.90

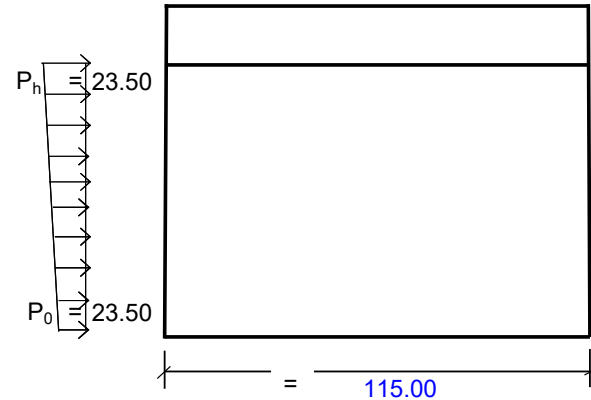
27.5 Directional Procedure, Part 2: Enclosed Simple Diaphragm Buildings With $H \leq 160$ Ft



Example will work for $V = 110, 115$ and 120 MPH only.

Wind Normal to Ridge

Roof Exp adjust factor $T-27.6-2 = 1$



Wind Parallel to Ridge

Roof Height $h = 15$ feet
 Exposure **C**
 Roof Pitch = **3.00 :12**
 Topography factor $K_{zt} = 1.00$
 Building & Structure Risk Category = **II, standard**
 Wind Speed $V = 110$ mph
 Wind Normal to Ridge, $L/B = 0.535$ use **0.5**
 Since $.2 \leq L/B \leq 5.0 = \text{Yes}$
 and $h \leq 60 = \text{Yes}$
 Building meet Class I condition ? Yes $\Leftarrow$
 Since $.5 \leq L/B \leq 2.0 = \text{Yes}$
 and $60 \leq h \leq 160 = \text{No}$
 Fundamental period = **0.500 sec**
 More than $h/75$ sec. NG
 Building meet Class II condition ? No

From T-27.6-1, $V = 110$ MPH Exp C, $L/B = 0.5$ and 15 FT. HT

$$P_h = 25.20 K_{zt}$$

$$P_o = 25.20 K_{zt}$$

T-27.6-2 $V = 110$ Exp C, $h = 15$ Roof slope = 3:12

$$\text{Zone 1} = -23.5$$

Adjust factor T-27.6-2, = 1.00

Design roof pressure for zone 1 = -23.50 PSF

Wind Base Shear = 26.08 kips

26.8.2, Figure 26.8-1

IBC T-1604.5

Fig. 26.5-1A, MRI = 700 yrs

Wind Parallel to Ridge, $L/B = 1.87$ use **1.9**

Since $.2 \leq L/B \leq 5.0 = \text{Yes}$

and $h \leq 60 = \text{Yes}$

Building meet Class I condition ? Yes $\Leftarrow$

Since $.5 \leq L/B \leq 2.0 = \text{Yes}$

and $60 \leq h \leq 160 = \text{No}$

Fundamental period = **0.500 sec**

More than $h/75$ sec. NG

Building meet Class II condition ? No

From T-27.6-1, $V = 110$ MPH Exp C, $L/B = 1.9$ and 15 FT. HT

$$P_h = 23.50 K_{zt}$$

$$P_o = 23.50 K_{zt}$$

Wind Base Shear = 13.01 kips

ASCE Hazards Report

Standard: ASCE/SEI 7-22

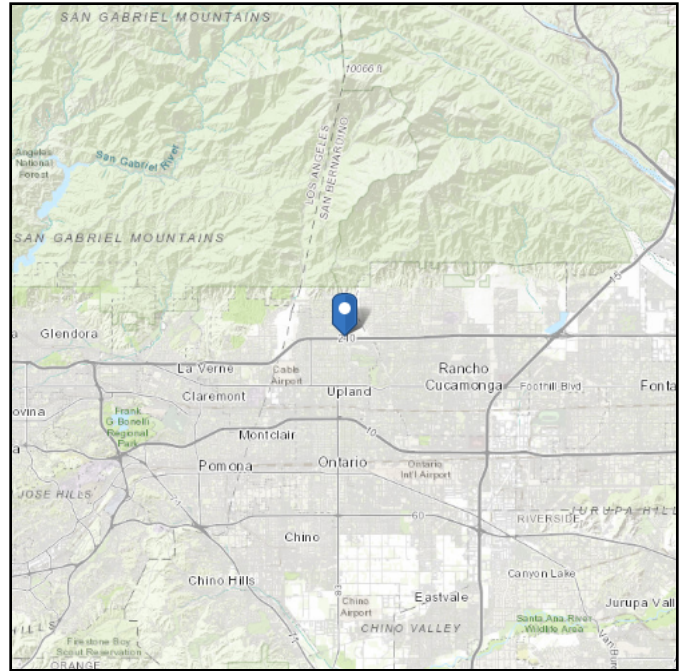
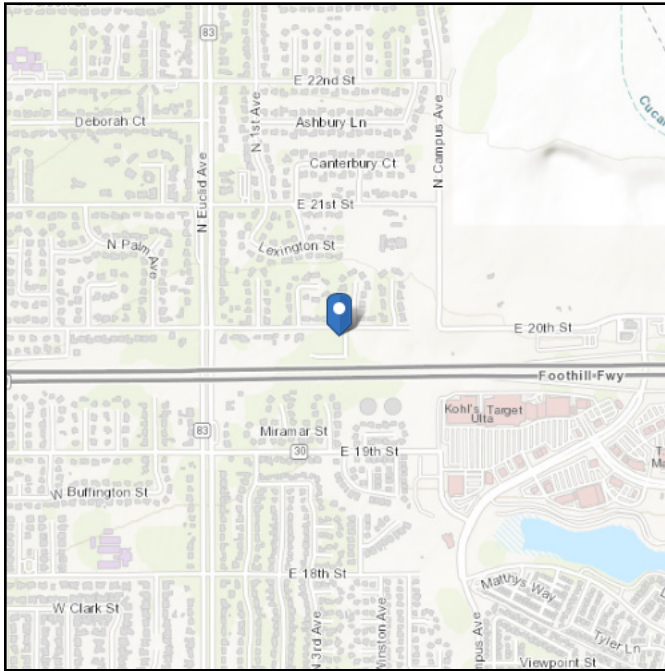
Risk Category: II

Soil Class: Default

Latitude: 34.135827

Longitude: -117.646592

Elevation: 1648.2100926574194 ft
(NAVD 88)



Wind

Results:

Wind Speed	95 Vmph
10-year MRI	66 Vmph
25-year MRI	72 Vmph
50-year MRI	77 Vmph
100-year MRI	81 Vmph
300-year MRI	89 Vmph
700-year MRI	95 Vmph
1,700-year MRI	102 Vmph
3,000-year MRI	106 Vmph
10,000-year MRI	115 Vmph
100,000-year MRI	133 Vmph
1,000,000-year MRI	151 Vmph

Data Source: ASCE/SEI 7-22, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2

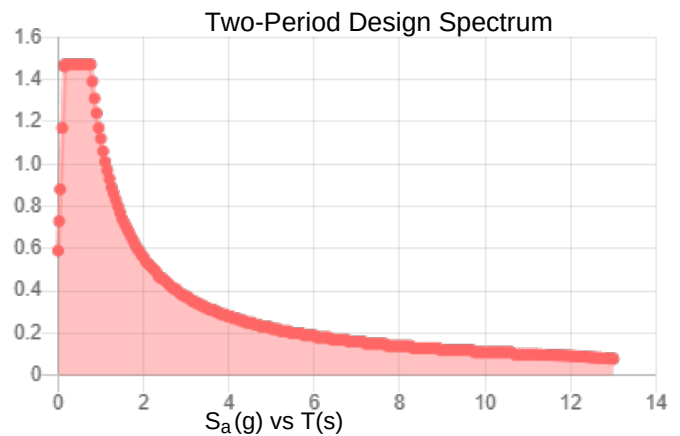
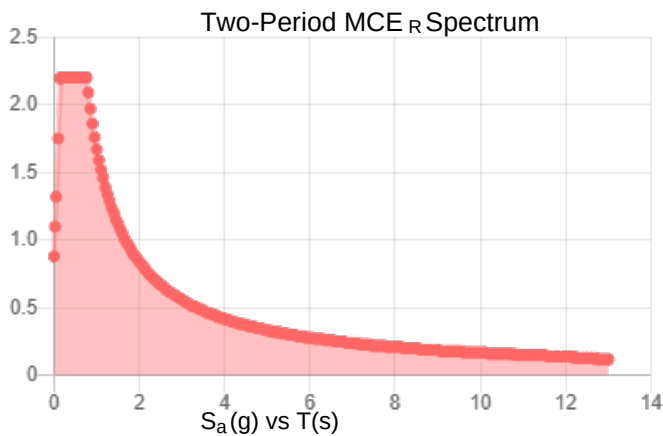
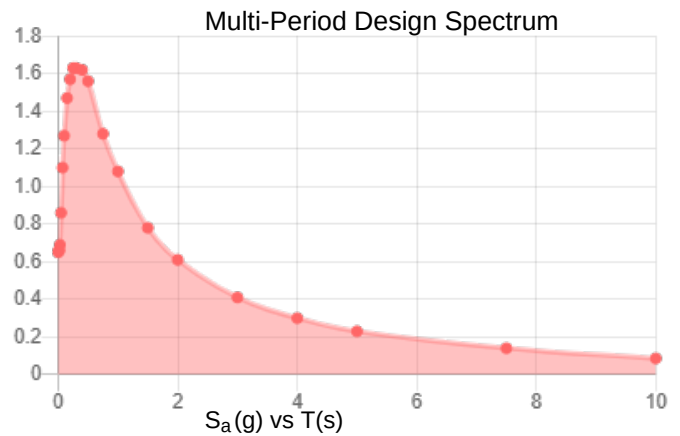
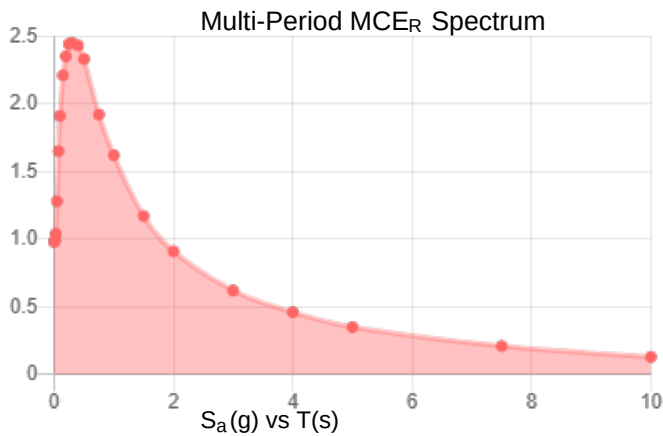
Date Accessed: Fri Aug 15 2025

Site Soil Class: Default

Results:

PGA _M :	0.85	T _L :	12
S _{MS} :	2.2	S _S :	1.98
S _{M1} :	1.67	S ₁ :	0.67
S _{DS} :	1.47	V _{S30} :	260
S _{D1} :	1.12		

Seismic Design Category: D



MCE_R Vertical Response Spectrum
Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum
Vertical ground motion data has not yet been made available by USGS.

IBC2021 (1613), ASCE 7-16 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

Response Spectral Acc. (0.2 sec) $S_s = 1.980\text{ g}$

Figure 22-1,22-3,22-5,22-6

Response Spectral Acc. (1.0 sec) $S_1 = 0.670\text{ g}$

Figure 22-2,22-4,22-5,22-6

Soil Site Class

Table 20-3-1, Default = D

Site Coefficient $F_a = 1.200$

Table 11.4-1

Site Coefficient $F_v = 1.500$

Table 11.4-2

Max Considered Earthquake Acc. $S_{MS} = F_a \cdot S_s = 2.376\text{ g}$ (11.4-1)

Max Considered Earthquake Acc. $S_{M1} = F_v \cdot S_1 = 1.005\text{ g}$ (11.4-2)

@ 5% Damped Design $S_{DS} = 2/3(S_{MS}) = 1.584\text{ g}$ (11.4-3)

$S_{D1} = 2/3(S_{M1}) = 0.670\text{ g}$ (11.4-4)

Building Risk Categories

Table 1604.5

Design Category Consideration:

with dist. between seismic resisting system >40ft

Seismic Design Category for 0.1sec D

Table 11.6-1

Seismic Design Category for 1.0sec D

Table 11.6-2

$S_1 < .75\text{ g}$ NA

Section 11.6

Since $T_a < .8T_s$ (see below), SDC = D

Control (exception of Section 11.6 does not apply)

IBC, Seismic Design Category D

IRC, Seismic Design Category = E

T-R301.2.2.1.1

12.8 Equivalent lateral force procedure

A. BEARING WALL SYSTEMS

T-12.2

Seismic Force Resisting Systems

15. Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance or steel

$C_t = 0.02$

$x = 0.75$

T-12.8-2

Building ht. $H_n = 35$

ft

Unfinished Building Height (ft) = 65

$C_u = 1.400$

for S_{D1} of 0.670g

Table 12.8-1

Approx Fundamental period, $T_a = C_t(h_n)^x = 0.288$

12.8-7

$T_L = 6.000\text{ Sec}$

Calculated T shall not exceed $\leq C_u \cdot T_a = 0.403$

Use T = sec.

$0.8T_s = 0.8(S_{D1}/S_{DS}) = 0.338 < T_a$

Is structure Regular & ≤ 5 stories?

12.8.1.3

Response Spectral Acc. (0.2 sec) $S_s = 1.980\text{ g}$

$F_a = 1.20$

@ 5% Damped Design $S_{DS} = 2/3(F_a \cdot S_s) = 1.584\text{ g}$ (11.4-3)

Response Modification Coef. $R = 6.5$

Table-12.2-1

Over Strength Factor $\Omega_o = 2.5$

foot note g

Importance factor $I = 1$

T 1.5-2

Seismic Base Shear $V = C_s W$

$$C_s = \frac{S_{DS}}{R/I} = 0.244 \quad (12.8-2)$$

$$\text{or need not to exceed, } C_s = \frac{S_{D1}}{(R/I) \cdot T} = 0.256 \quad \text{For } T \leq T_L \quad (12.8-3)$$

$$\text{or } C_s = \frac{S_{D1} T_L}{T^2 (R/I)} \quad \text{N/A} \quad \text{For } T > T_L \quad (12.8-4)$$

C_s shall not be less than $= 0.044 S_{DS} \cdot I \geq 0.1 = 0.070$ (12.8-5)

Min $C_s = 0.5 S_1 / R = 0.052$ For $S_1 \geq 0.6\text{ g}$ (12.8-6)

Use $C_s = 0.244$

Design base shear $V = 0.244 W$ Control

12.14 Simplified Seismic base shear

15. Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance or steel

T-12.14

@ 5% Damped Design $S_{DS} = 1.584$

SDC = D

Limitations: P

$F = 1.2$

For three story building

$R = 6.5$

$$V = \frac{F S_{DS} (W)}{R} = 0.292 W$$

(12.14-11)

LATERAL FORCE DISTRIBUTION

SEISMIC BASE SHEAR

Importance Factor, I ASCE7-16 Table 11.5-1 I = 1.00
EQ Acc. @ 5% Damped Design ASCE7-16 EQ 11.4-3 SDS = 1.58
Vult = 2021 IBC SHEET = 0.244 W
Vwork = Vult / 1.4 = 0.174 W Working Stress Level
Vwork = 0.174 * 115.9 kips = 20.2 kips Seismic Base Shear
p = 1.30 ==> Vwork = 26.2 kips Seismic Base Shear

WIND BASE SHEAR

Vult = MWFR < 160 ft = 26.1 kips
Vwork = Vult * 0.6 = 15.6 kips

SEISMIC BASE SHEAR GOVERNS

DESIGN BASE SHEAR	=	26.2 kips
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BASE SHEAR VERTICAL DISTRIBUTION

Floor Type	Height hx ft	Floor Area ft ²	Weight wx kips	wx*hx kips-ft	Fx % wx*hx Σ (wx*hx)	Fx (V-Ft) Fx% kips	Story V Ft+ΣFi kips	Fx Fl Area psf
ROOF	8.50	5,469	115.90	985	100.0%	26.23	26.23	4.80
		Total	115.90	985	100.0%	26.23		

DIAPHRAGM FORCE DISTRIBUTION

Floor Type	Weight		Story V Ft+ΣFi kips	Fpx V*wp/Σw kips	Min Fpx ¹ kips	Max Fpx ² kips	Fpx Design kips
	wp kips	Σwi kips					
ROOF	102.40	102.40	26.23	26.23	32.44	64.88	32.44

1 Fpx min = 0.2*SDs*I*wp (for Flexible Diaphragm)

ASCE7-16 12.10.1.1

2 Fpx max = 0.4*SDs*I*wp (for Flexible Diaphragm)

ASCE7-16 12.10.1.1

SHEARWALL DESIGN FORCES

Level	Fl. Area	Wall Ht.	Stry Shear kips	Base ² Shear %	Shear psf
	ft ²	ft			
ROOF	5,469	8.50	26.23	100.0%	4.80

1 Redundancy Factor Used Per ASCE7-16 12.3.4.2

TABLES REFERENCED IN SHEARWALL DESIGN SPREADSHEET

Table 2306.4.1 Allowable Shear (plf) for Wood Shearwall Panels For Wind or Seismic Loading

Mark	# sides	Sheathing	Edge Nailing	A35/LTP 4 Simpson	Sill Plate @ Framed Floor	Anchor @ Foundation	V _{allow} (100%) (plf)	V _{allow} (75%) (plf)
1	1	15/32"	10d @6"	@16"	3/8" dia Lag Screw @32"OC	5/8" dia A.B. @48"OC	340	255
2	1	15/32"	10d @4"	@8"	3/8" dia Lag Screw @16"OC	5/8" dia A.B. @32"OC	510	380
3	1	15/32"	10d @3"	@12"	3/8" dia Lag Screw @12"OC	5/8" dia A.B. @24"OC	665	500
4	1	15/32"	10d @2"	@8"	3/8" dia Lag Screw @9"OC	5/8" dia A.B. @16"OC	870	650
5	2	15/32"	10d @4"	@8"	3/8" dia Lag Screw @9"OC	5/8" dia A.B. @16"OC	1,020	765
6	2	15/32"	10d @3"	@4"	3/8" dia Lag Screw @9"OC	5/8" dia A.B. @12"OC	1,370	1,030
7	2	15/32"	10d @2"	@4"	3/8" dia Lag Screw @4"OC	5/8" dia A.B. @16"OC	1,740	1,305

NOTES:

1. ALL SHEATHING TO BE DOUGLAS FIR-LARCH STRUCT. I.
2. ALL FIELD NAILING TO INTERMEDIATE FRAMING TO BE AT 12" O.C.
3. FRAMING AT ADJOINING PANEL EDGES SHALL BE 3" (INCH) NOMINAL OR THICKER (i.e. MIN 3x VERTICAL STUDS AND BLOCKING) NAILS SHALL BE STAGGERED IN TWO LINES ALONG PANEL EDGES WHERE NAILS ARE SPACED 2" INCHES ON CENTER OR WHEN 10d COMMON NAILS SPACED 3" O.C. PENETRATE FRAMING MORE THAN 1 5/8".
4. WHERE PLYWOOD APPLIES ON BOTH FACES OF A WALL AND NAIL SPACING IS LESS THAN 6" INCHES ON CENTER ON EITHER SIDE. PANEL JOINTS SHALL HAVE MIN 3x (VERTICAL STUDS & BLOCKING) AND NAILS SHALL BE STAGGERED. OFFSET PANEL JOINTS TO FALL ON DIFFERENT FRAMING MEMBER WHERE PLYWOOD IS PLACED AT BOTH FACES.
5. NAILS SHALL BE PLACED NOT LESS THAN 1/2" INCH FROM PANEL EDGES.
6. FOR HOLD-DOWNS AT THE END OF THE SHEAR WALL, SEE PLANS AND SCHEDULE. PROVIDE 3"x3"x1/4" WASHER PLATE AT ALL SILL PLATE ANCHOR BOLTS AND AT ALL HOLD-DOWN BOLTS CONNECTED TO VERTICAL POSTS.
7. USE WELDED STUD BOLTS OF SAME SIZE & SPACING AS OF SILL PLATE ANCHOR BOLTS, WHERE STEEL BEAM OCCURS UNDER SHEAR WALL.
8. PROVIDE DOUBLE PARALLAM BLOCKING TO ACCOMODATE 2 ROWS OF SIMPSON SDS SCREWS.
9. FOUNDATION SILL PLATE SHALL BE 3x. U.N.O.
10. SIMPSON SDS 3/8"x7" WOOD SCREWS. TYP. USE |2"x4|4" SDS INTO NAILER AT STEEL BEAMS.
11. WHERE 2-2x SILL PLATES ARE USED, SCREW LOWER PLATE TO FRAMING BELOW. DRILL HOLE IN UPPER SILL PLATE FOR SCREW HEAD SO THAT UPPER PLATE SITS FLAT ON LOW SILL PLATE.

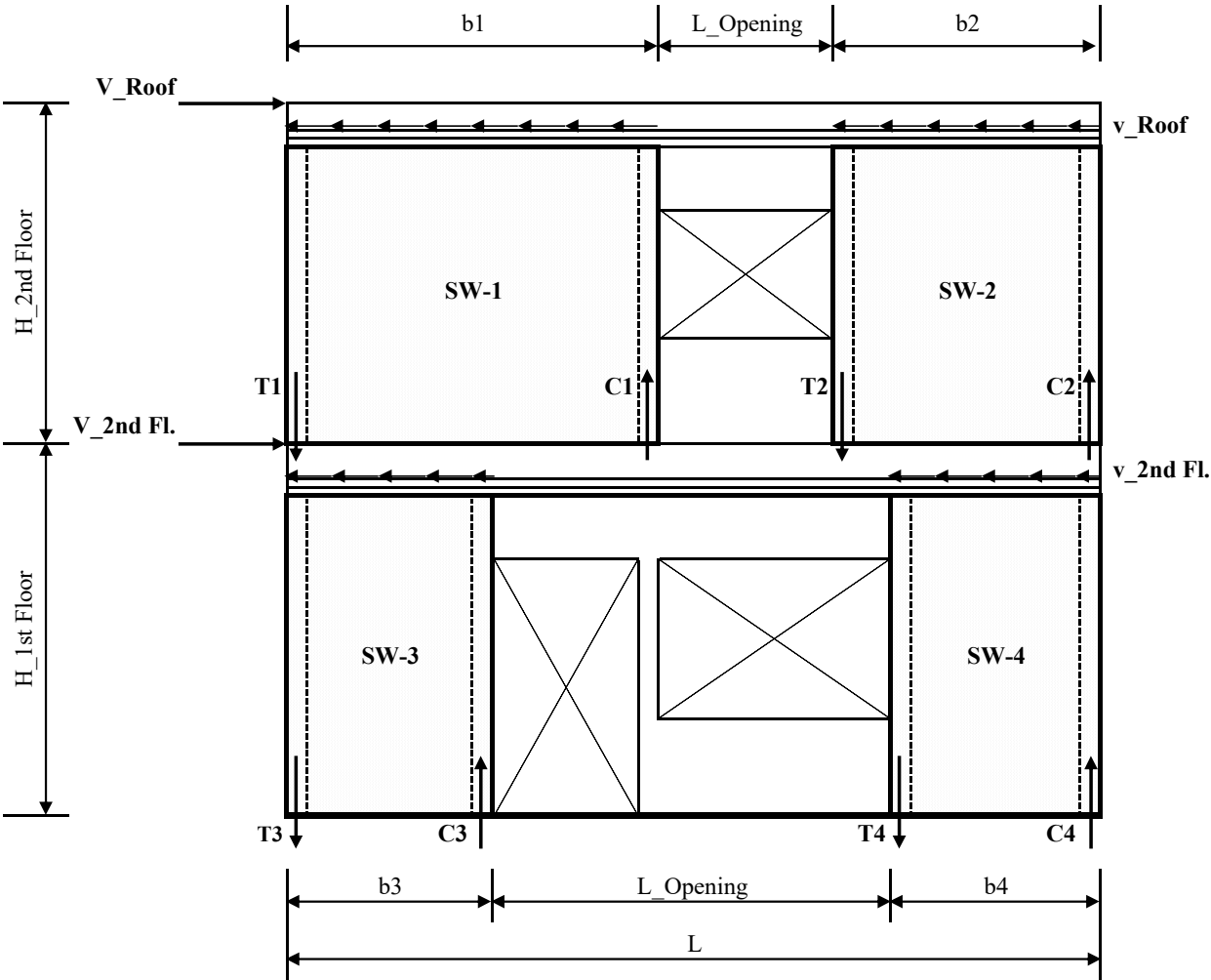
TABLES REFERENCED IN SHEARWALL DESIGN SPREADSHEET (HOLDOWN)

Holdown Capacity

HD Type	Max Uplift Force (lbs)			Post Size
	75%	Full	Used	
HDU2	2,306	3,075	2,306	4x4
HDU4	3,424	4,565	3,424	4x4
HDU5	4,253	5,670	4,253	4x4
HDU8	5,903	7,870	5,903	4x6
HDU11	7,151	9,535	7,151	4x6
HDU14	10,793	14,390	10,793	4x8

NOTES:

1. USE THREADED ROD @ EXISTING FOOTING ONLY. SEE DETAIL 16/S4 LARR# 25279.
2. USE SSTB @ NEW FOOTING ONLY PER LARR# 25248.
3. PROVIDE MIN. POST SIZE AS INDICATED, INCREASE SIZE TO MATCH WALL THICKNESS.
4. ALL HOLDOWN TO BE DOUGLAS FIR-LARCH No. 1.



AT ROOF LEVEL

Total Shearwall Length	ft	=	Sum (b1 + b2 + ..)
Total Shear (V_Roof)	lbs	=	Shear Trib Width * Line Trib Length (L) * Roof Unit Shear (psf)
Unit Shear (v_Roof)	plf	=	V_Roof / Sum(b1 + b2 + ..)
SW-1 Dead Load	lbs	=	Roof_Dead_Laod + Ceiling_Dead_Load + 2nd Fl Wall_Selfweight
SW-1 Uplift Force T1	lbs	=	(Upward Force) - (Downward Force)
		=	(v_Roof * H_2nd Fl) - (0.6 - 0.14*Sds) * SW-1 Dead Load /2)
Drag Force at Open	lbs	=	V Roof / Line Trib Length (L) * L Opening

AT SECOND FLOOR

Total Shearwall Length	ft	=	Sum (b3 + b4 + ..)
Total Shear (V_2nd Fl)	lbs	=	Shear Trib Width * Line Trib Length (L) * 2nd Fl Unit Shear (psf) + Roof Total Shear (V_Roof)
Unit Shear (v_2nd Fl)	plf	=	V_2nd Fl / Sum(b3 + b4 + ..)
SW-3 Dead Load	lbs	=	Roof_Dead_Laod + Ceiling_Dead_Load + 2nd Fl_Wall_Selfweight + Floor_Dead_Load + 1st Fl_Wall_Selfweight
SW-3 Uplift Force T3	lbs	=	(Upward Force) - (Downward Force)
Upward Force	lbs	=	v_Roof * (H_2nd Fl + H_1st Fl) + v_2nd Fl * H_1st Fl
Downward Force	lbs	=	(0.6 - 0.14*Sds) * SW-3 Dead Load /2
Drag Force at Open	lbs	=	V 2nd Fl / Line Trib Length (L) * L Opening

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.80 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE A PERFORATED							
ROOF			SW-01	SW-02	SW-03	SW-04	
Total Shearwall Segments	ΣL_i	ft	14.50	7.83	6.67	20.24	
Total Shearwall Length	L_{tot}	ft	19.25	7.83	11.42	26.75	
Percent Full-Height Sheathing		%	75%	100%	58%	76%	
Total Area of Openings	A_o	ft ²	21.38	-	21.38	29.30	
Sheathing Area Ratio	$r = \frac{1}{1 + \frac{A_o}{h \Sigma L_i}}$		0.85	1.00	0.73	0.85	
Shear Capacity Adjustment Factor	$C_o = \left(\frac{r}{3 - 2r} \right) \frac{L_{tot}}{\Sigma L_i}$		0.87	1.00	0.80	0.87	
Shear Trib. Width		ft	12.0				
Line Trib. Length		ft	92.0				
Shortest Panel > H/3.5 and < H/2		ft	3.1				
Line Total Shear	V	lbs	5,294				
Line Unit Shear	$V / (\Sigma L_i \times C_o)$	plf	134	x 4.25 ft / 3.08 ft =	185	pdf	
100% Wood Shearwall 10d	1		v = 340 plf (1-Side 15/32" Str. 1 w/ 10d Common Nails @ 6"-6"-12" O.C.)				
Roof DL Trib. Width		ft	-	-	-	-	
Ceiling DL Trib. Width		ft	-	-	-	-	
Wall Height		ft	8.5	8.5	8.5	8.5	
Wall Dead Load		plf	153.0	153.0	153.0	153.0	
Uplift Force (Demand)	$T = \frac{Vh}{C_o \Sigma L_i}$	lbs	718	911	945	552	
Holdown Type			H DU4	H DU4	H DU4	H DU4	
Uplift Force (Capacity)		lbs	3,424	3,424	3,424	3,424	

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.80 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE D PERFORATED							
ROOF			SW-05	SW-06	SW-07	SW-08	SW-09
Total Shearwall Segments	ΣL_i	ft	8.83	22.08	16.92	4.42	5.83
Total Shearwall Length	L_{tot}	ft	14.83	22.08	16.92	4.42	5.83
Percent Full-Height Sheathing		%	60%	100%	100%	100%	100%
Total Area of Openings	A_o	ft ²	27.00	-	-	-	-
Sheathing Area Ratio	$r = \frac{1}{1 + \frac{A_o}{h \Sigma L_i}}$		0.74	1.00	1.00	1.00	1.00
Shear Capacity Adjustment Factor	$C_o = \left(\frac{r}{3 - 2r} \right) \frac{L_{tot}}{\Sigma L_i}$		0.81	1.00	1.00	1.00	1.00
Shear Trib. Width		ft	21.0				
Line Trib. Length		ft	114.0				
Shortest Panel > H/3.5 and < H/2		ft					
Line Total Shear	V	lbs	11,481				
Line Unit Shear	$V / (\Sigma L_i \times C_o)$	plf	245				
100% Wood Shearwall 10d	2	v = 510 plf (1-Side 15/32" Str. 1 w/ 10d Common Nails @ 4"-6"-12" O.C.)					
Roof DL Trib. Width		ft	-	-	-	-	
Ceiling DL Trib. Width		ft	-	-	-	-	
Wall Height		ft	8.5	8.5	8.5	8.5	8.5
Wall Dead Load		plf	153.0	153.0	153.0	153.0	153.0
Uplift Force (Demand)	$T = \frac{Vh}{C_o \Sigma L_i}$	lbs	1,825	1,441	1,590	1,952	1,911
Holdown Type			HDU4	HDU4	HDU4	HDU4	HDU4
Uplift Force (Capacity)		lbs	3,424	3,424	3,424	3,424	3,424

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.80 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE F							
ROOF		SW-10	SW-11	SW-12	SW-13	SW-11A	
Shearwall Length	ft	9.00	6.08	4.00	3.50	6.67	
Total Shearwall Length	ft	29.25					
Shear Trib. Width	ft	11.0	6.0				
Line Trib. Length	ft	115.0	44.0				
Line Total Length	ft	159.0					
Line Total Shear	lbs	7,332					
Line Unit Shear	plf	251	x 4 ft / 3.5 ft =		304.4	plf	
75% Wood Shearwall 10d		4	v = 653 plf (1-Side 15/32" Str. I w/ 10d Common Nails @ 2'-6"-12" O.C.)				
Drag Length	ft	6.0					
Drag Force $\times \Omega/1.2$	lbs	576					
Strap Type		MST27					
Roof DL Trib. Width	ft	-	-	-	-	-	
Ceiling DL Trib. Width	ft	-	-	-	-	-	
Wall Height	ft	8.5	8.5	8.5	8.5	8.5	
Wall Dead Load	plf	153.0	153.0	153.0	153.0	153.0	
Wall Shear Force	lbs	2,256	1,524	1,003	877	1,672	
Uplift Force (Demand)	lbs	2,327	2,411	2,472	2,486	2,394	
Holdown Type		HDU4	HDU4	HDU4	HDU4	HDU4	
Uplift Force (Capacity)	lbs	3,424	3,424	3,424	3,424	3,424	

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.80 psf

Wall Type Wall Height
Ext. 17.00 ft

LINE F - (HFX)							
ROOF		SW-14	SW-15				
Shearwall Length	ft	1.50	1.50				
Total Shearwall Length	ft	3.00					
Shear Trib. Width	ft	12.0					
Line Trib. Length	ft	40.0					
Line Total Length	ft	40.0					
Line Total Shear	lbs	2,302					
Line Unit Shear	plf	767					
18" HFX on 3,000 psi Conc.		HFX-18x17-HS	v = 1195 lbs/Panel - Width = 18" - Height = 200 1/4"				
Drag Length	ft	6.0					
Drag Force $\times \Omega / 1.2$	lbs	719					
Strap Type		MST27					
Roof DL Trib. Width	ft	-	-	-	-	-	-
Ceiling DL Trib. Width	ft	-	-	-	-	-	-
Wall Height	ft	17.0	17.0				
Wall Dead Load	plf	306.0	306.0				
Wall Shear Force	lbs	1,151	1,151				
Uplift Force (Demand)	lbs	12,957	12,957				
Holdown Type		1 1/8" HS	1 1/8" HS				
Uplift Force (Capacity)	lbs	18,775	18,775				

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.80 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE H-J PERFORATED							
ROOF			SW-16	SW-17			
Total Shearwall Segments	ΣL_i	ft	8.08	14.00			
Total Shearwall Length	L_{tot}	ft	16.17	26.00			
Percent Full-Height Sheathing		%	50%	54%			
Total Area of Openings	A_o	ft ²	35.64	54.00			
Sheathing Area Ratio	$r = \frac{1}{1 + \frac{A_o}{h \Sigma L_i}}$		0.66	0.69			
Shear Capacity Adjustment Factor	$C_o = \left(\frac{r}{3 - 2r} \right) \frac{L_{tot}}{\Sigma L_i}$		0.78	0.79			
Shear Trib. Width		ft	9.0				
Line Trib. Length		ft	44.0				
Shortest Panel > H/3.5 and < H/2		ft	3.7				
Line Total Shear	V	lbs	1,899				
Line Unit Shear	$V / (\Sigma L_i \times C_o)$	plf	110	x 4.25 ft / 3.67 ft =	127	pdf	
100% Wood Shearwall 10d	1		v = 340 plf (1-Side 15/32" Str. 1 w/ 10d Common Nails @ 6"-6"-12" O.C.)				
Roof DL Trib. Width		ft	-	-			
Ceiling DL Trib. Width		ft	-	-			
Wall Height		ft	8.5	8.5			
Wall Dead Load		plf	153.0	153.0			
Uplift Force (Demand)	$T = \frac{Vh}{C_o \Sigma L_i}$	lbs	700	529			
Holdown Type			H DU4	H DU4			
Uplift Force (Capacity)		lbs	3,424	3,424			

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.80 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE 1-2 PERFORATED							
ROOF			SW-18	SW-19			
Total Shearwall Segments	ΣL_i	ft	10.25	8.92			
Total Shearwall Length	L_{tot}	ft	16.25	14.92			
Percent Full-Height Sheathing		%	63%	60%			
Total Area of Openings	A_o	ft ²	27.00	27.00			
Sheathing Area Ratio	$r = \frac{1}{1 + \frac{A_o}{h \Sigma L_i}}$		0.76	0.74			
Shear Capacity Adjustment Factor	$C_o = \left(\frac{r}{3 - 2r} \right) \frac{L_{tot}}{\Sigma L_i}$		0.82	0.81			
Shear Trib. Width		ft	10.0				
Line Trib. Length		ft	40.0				
Shortest Panel > H/3.5 and < H/2		ft					
Line Total Shear	V	lbs	1,918				
Line Unit Shear	$V / (\Sigma L_i \times C_o)$	plf	124				
100% Wood Shearwall 10d	1		v = 340 plf (1-Side 15/32" Str. 1 w/ 10d Common Nails @ 6"-6"-12" O.C.)				
Roof DL Trib. Width		ft	-	-			
Ceiling DL Trib. Width		ft	-	-			
Wall Height		ft	8.5	8.5			
Wall Dead Load		plf	153.0	153.0			
Uplift Force (Demand)	$T = \frac{Vh}{C_o \Sigma L_i}$	lbs	755	794			
Holdown Type			H DU4	H DU4			
Uplift Force (Capacity)		lbs	3,424	3,424			

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.80 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE 3

ROOF		SW-20				
Shearwall Length	ft	16.08				
Total Shearwall Length	ft	16.08				
Shear Trib. Width	ft	20.0				
Line Trib. Length	ft	35.0				
Line Total Length	ft	35.0				
Line Total Shear	lbs	3,357				
Line Unit Shear	plf	209				
75% Wood Shearwall 10d		1	v = 255 plf (1-Side 15/32" Str. I w/ 10d Common Nails @ 6"-6"-12" O.C.)			
Drag Length	ft	6.0				
Drag Force $\times \Omega/1.2$	lbs	1,199				
Strap Type		MST27				
Roof DL Trib. Width	ft	-				
Ceiling DL Trib. Width	ft	-				
Wall Height	ft	8.5				
Wall Dead Load	plf	153.0				
Wall Shear Force	lbs	3,357				
Uplift Force (Demand)	lbs	1,309				
Holdown Type		H DU4				
Uplift Force (Capacity)	lbs	3,424				

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.80 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE 4

ROOF		POST					
Shearwall Length	ft	16.08					
Total Shearwall Length	ft	16.08					
Shear Trib. Width	ft	11.0					
Line Trib. Length	ft	20.0					
Line Total Length	ft	20.0					
Line Total Shear R=2.5	lbs	2,743					
Line Unit Shear	plf						
75% Wood Shearwall 10d							
Drag Length	ft	6.0					
Drag Force $\times \Omega/1.2$	lbs						
Strap Type							
Roof DL Trib. Width	ft	-					
Ceiling DL Trib. Width	ft	-					
Wall Height	ft	8.5					
Wall Dead Load	plf						
Wall Shear Force	lbs						
Uplift Force (Demand)	lbs						
Holdown Type							
Uplift Force (Capacity)	lbs						

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.80 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE 5 PERFORATED							
ROOF			SW-21	SW-22			
Total Shearwall Segments	ΣL_i	ft	11.00	22.08			
Total Shearwall Length	L_{tot}	ft	11.00	24.25			
Percent Full-Height Sheathing		%	100%	91%			
Total Area of Openings	A_o	ft ²	-	9.77			
Sheathing Area Ratio	$r = \frac{1}{1 + \frac{A_o}{h \Sigma L_i}}$		1.00	0.95			
Shear Capacity Adjustment Factor	$C_o = \left(\frac{r}{3 - 2r} \right) \frac{L_{tot}}{\Sigma L_i}$		1.00	0.95			
Shear Trib. Width		ft	12.0	12.0			
Line Trib. Length		ft	58.0	46.0			
Shortest Panel > H/3.5 and < H/2		ft					
Line Total Shear	V	lbs	5,985				
Line Unit Shear	$V / (\Sigma L_i \times C_o)$	plf	190				
100% Wood Shearwall 10d	1		v = 340 plf (1-Side 15/32" Str. 1 w/ 10d Common Nails @ 6"-6"-12" O.C.)				
Roof DL Trib. Width		ft	-	-			
Ceiling DL Trib. Width		ft	-	-			
Wall Height		ft	8.5	8.5			
Wall Dead Load		plf	153.0	153.0			
Uplift Force (Demand)	$T = \frac{Vh}{C_o \Sigma L_i}$	lbs	1,301	980			
Holdown Type			H DU4	H DU4			
Uplift Force (Capacity)		lbs	3,424	3,424			

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.80 psf

Wall Type Wall Height
Ext. 17.00 ft

LINE 6						
ROOF		SW-23	SW-24	SW-25		
Shearwall Length	ft	10.33	8.83	3.92		
Total Shearwall Length	ft	23.08				
Shear Trib. Width	ft	21.0				
Line Trib. Length	ft	46.0				
Line Total Length	ft	46.0				
Line Total Shear	lbs	4,633				
Line Unit Shear	plf	201				
75% Wood Shearwall 10d		3	v = 499 plf (1-Side 15/32" Str. I w/ 10d Common Nails @ 3"-6"-12" O.C.)			
Drag Length	ft	6.0				
Drag Force $\times \Omega/1.2$	lbs	1,259				
Strap Type		MST27				
Roof DL Trib. Width	ft	-	-	-		
Ceiling DL Trib. Width	ft	-	-	-		
Wall Height	ft	-	-	-		
Wall Dead Load	plf	-	-	-		
Wall Shear Force	lbs	2,073	1,772	787		
Uplift Force (Demand)	lbs	3,412	3,412	3,412		
Holdown Type		HDU8	HDU8	HDU8		
Uplift Force (Capacity)	lbs	5,903	5,903	5,903		

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.80 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE 7 (HFX)						
ROOF	SW-26					
Shearwall Length ft	1.75					
Total Shearwall Length ft	1.75					
Shear Trib. Width ft	21.0					
Line Trib. Length ft	17.0					
Line Total Length ft	17.0					
Line Total Shear lbs	1,712					
Line Unit Shear plf	978					
21" HFX on 3,000 psi Conc.	HFX-21x17-HS	v = 1745 lbs/Panel - Width = 21" - Height = 200 1/4"				
Drag Length ft	6.0					
Drag Force $\times \Omega/1.2$ lbs	1,259					
Strap Type	MST27					
Roof DL Trib. Width ft	-					
Ceiling DL Trib. Width ft	-					
Wall Height ft	8.5					
Wall Dead Load plf	153.0					
Wall Shear Force lbs	1,712					
Uplift Force (Demand) lbs	8,265					
Holdown Type	1 1/8" HS					
Uplift Force (Capacity) lbs	23,050					

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.80 psf

Wall Type Wall Height
Ext. 17.00 ft

LINE 7							
ROOF		SW-27	SW-28				
Shearwall Length	ft	12.00	6.92				
Total Shearwall Length	ft	18.92					
Shear Trib. Width	ft	11.0	12.0				
Line Trib. Length	ft	48.0	43.0				
Line Total Length	ft	91.0					
Line Total Shear	lbs	5,007					
Line Unit Shear	plf	265	x 9 ft / 6.92 ft =	325.0	plf		
75% Wood Shearwall 10d		3	v = 499 plf (1-Side 15/32" Str. I w/ 10d Common Nails @ 3"-6"-12" O.C.)				
Drag Length	ft	6.0					
Drag Force $\times \Omega/1.2$	lbs	688					
Strap Type		MST27					
Roof DL Trib. Width	ft	-	-				
Ceiling DL Trib. Width	ft	-	-				
Wall Height	ft	-	-				
Wall Dead Load	plf	-	-				
Wall Shear Force	lbs	3,175	1,831				
Uplift Force (Demand)	lbs	5,526	5,526				
Holdown Type		HDU11	HDU11				
Uplift Force (Capacity)	lbs	7,151	7,151				

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.80 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE 8						
ROOF		SW-29				
Shearwall Length	ft	9.67				
Total Shearwall Length	ft	9.67				
Shear Trib. Width	ft	13.5				
Line Trib. Length	ft	44.0				
Line Total Length	ft	44.0				
Line Total Shear	lbs	2,849				
Line Unit Shear	plf	295				
75% Wood Shearwall 10d		2	v = 383 plf (1-Side 15/32" Str. I w/ 10d Common Nails @ 4"-6"-12" O.C.)			
Drag Length	ft	6.0				
Drag Force $\times \Omega/1.2$	lbs	809				
Strap Type		MST27				
Roof DL Trib. Width	ft	-				
Ceiling DL Trib. Width	ft	-				
Wall Height	ft	-				
Wall Dead Load	plf	-				
Wall Shear Force	lbs	2,849				
Uplift Force (Demand)	lbs	2,504				
Holdown Type		HDU4				
Uplift Force (Capacity)	lbs	3,424				

TABLES REFERENCED IN BEAM DESIGN SPREADSHEET

2018 NDS Table 4A

Douglas Fir-Larch ¹ WWPA	Design Values for Visually Graded (2"-4")		
	fb	fv	E
Select Str	1,500	180	1,900,000
No.1&Better	1,200	180	1,800,000
No.1	1,000	180	1,700,000
No.2	900	180	1,600,000
No.3	525	180	1,400,000
Stud	700	180	1,400,000

2018 NDS Table 4D

Douglas Fir-Larch ¹ WWPA	Design Values for Visually Graded (5" & Larger)		
	fb	fv	E
Dense Str	1,900	170	1,700,000
Select Str	1,600	170	1,600,000
Dense No. 1	1,550	170	1,700,000
No.1	1,350	170	1,600,000
Dense No. 2	1,000	170	1,400,000
No.2	875	170	1,300,000

Other Material

Prllm2.0E	2,900	290	2,000,000
Mcrllm1.8E	2,600	285	1,800,000
Mcrllm1.9E	2,600	285	1,900,000
Steel Fy=36 ksi	23,760	14400	29,000,000
Steel Fy=46 ksi	30,360	18400	29,000,000
Steel Fy=50 ksi	33,000	20000	29,000,000

1. FOR DOUGLAS FIR-LARCH USE 19% MAX. MOISTURE CONTENT

LOAD SYMBOLS AND NOTATION

D = Dead Load	E = Earthquake Load
Lr = Roof Live Load	W = Wind Load
L = Floor Live Load	

ASCE7-16 2.4 LOAD COMBINATION - ALLOWABLE STRESS DESIGN

- | | | |
|--|----------------------|---|
| 1. D | | |
| 2. D + Lr | Load Duration = 1.25 | } Max. (Dead + Live) |
| 3. D + L | | |
| 4. D + 0.75 (Lr + L) | Load Duration = 1.25 | |
| 5. $(1.0 + 0.14S_{DS})D + 0.7\Omega E$ | | } Max. (Dead + Live + Lateral)
Allowable Stress Increase = 1.2
$\Omega = 2.5$ For Flexible Diaphragm
ASCE7-16 12.4.3.2 |
| 6. $(1.0 + 0.105S_{DS})D + 0.525\Omega E + 0.75(L + Lr)$ | | |
| 7. $(0.6 - 0.14S_{DS})D + 0.7\Omega E$ | | |

TABLE 1604.3 (2022 CBC) DEFLECTION LIMITS

Roof Members	L	D + L
Supporting Plaster Ceiling	$l / 360$	$l / 240$
Supporting Nonplaster Ceiling	$l / 240$	$l / 180$
Not Supporting Ceiling	$l / 180$	$l / 120$
Floor Members	$l / 360$	$l / 240$

SIMPLY SUPPORTED BEAM DESIGN

	RR	RR1	RR2	RR3	CJ	CJ1
SPAN ft	8.00	12.00	19.00	21.00	6.00	10.00
MATERIAL (WOOD, STEEL)	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION in	2x 6	2x 8	2x10	2x12	2x 4	2x 6
GRADE	No.2	No.2	No.2	No.2	No.2	No.2
ALLOWABLE STRESSES						
REPETITIVE MEMBER	YES	YES	YES	YES	YES	YES
REPETITIVE FACTOR, Cr	1.15	1.15	1.15	1.15	1.15	1.15
SIZE FACTOR, Cf	1.30	1.20	1.10	1.00	1.50	1.30
BENDING STRESSES, F'b psi	1,346	1,242	1,139	1,035	1,553	1,346
SHEAR STRESSES, F'v psi	207	207	207	207	207	207
MODULUS OF ELASTICITY ksi	1,600	1,600	1,600	1,600	1,600	1,600
UNIFORM DISTRIBUTED LOAD OVER FULL SPAN						
ROOF TRIB. WIDTH ft	1.33	1.33	1.33	1.33		
CEILING TRIB. WIDTH ft					1.33	1.33
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft						
WALL TYPE (EXT, INT)						
AUTO CALC BEAM WEIGHT						
TOTAL DEAD LOAD (D plf	17.29	17.29	17.29	17.29	7.98	7.98
TOTAL ROOF LIVE LOAD (L _r plf	26.60	26.60	26.60	26.60	-	-
TOTAL FLOOR LIVE LOAD (L plf	-	-	-	-	26.60	26.60
PARTIALLY DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
CEILING TRIB. WIDTH ft						
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft						
WALL TYPE (EXT, INT)						
TOTAL DEAD LOAD (D plf	-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf	-	-	-	-	-	-
TOTAL FLOOR LIVE LOAD (L plf	-	-	-	-	-	-
FROM (LEFT SUPPORT) ft						
LENGTH OF LOAD ft						
TRIANGULAR DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
TOTAL DEAD LOAD (D plf	-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf	-	-	-	-	-	-
CONCENTRATED LOADS						
P1						
DEAD LOAD (P D) lbs						
ROOF LIVE LOAD (P L _r) lbs						
FLOOR LIVE LOAD (P L) lbs						
LATERAL, P (W or 0.7E) lbs						
DIST. (LEFT SUPPORT) ft						
P2						
DEAD LOAD (P D) lbs						
ROOF LIVE LOAD (P L _r) lbs						
FLOOR LIVE LOAD (P L) lbs						
LATERAL, P (W or 0.7E) lbs						
DIST. (LEFT SUPPORT) ft						

SIMPLY SUPPORTED BEAM DESIGN

		RR	RR1	RR2	RR3	CJ	CJ1
SPAN	ft	8.00	12.00	19.00	21.00	6.00	10.00
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	2x 6	2x 8	2x10	2x12	2x 4	2x 6
GRADE		No.2	No.2	No.2	No.2	No.2	No.2
P3							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
P4							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
LOAD CASE 1: D							
MAX. BENDING MOMENT lb-ft		138	311	780	953	36	100
MAX. DEFLECTION in		0.048	0.106	0.320	0.266	0.027	0.054
LEFT REACTION lbs		69	104	164	182	24	40
RIGHT REACTION lbs		69	104	164	182	24	40
LOAD CASE: Lr							
MAX. BENDING MOMENT lb-ft		213	479	1,200	1,466	-	-
MAX. DEFLECTION in		0.074	0.163	0.493	0.409	-	-
LEFT REACTION lbs		106	160	253	279	-	-
RIGHT REACTION lbs		106	160	253	279	-	-
LOAD CASE: L							
MAX. BENDING MOMENT lb-ft		-	-	-	-	120	333
MAX. DEFLECTION in		-	-	-	-	0.090	0.180
LEFT REACTION lbs		-	-	-	-	80	133
RIGHT REACTION lbs		-	-	-	-	80	133
LOAD CASE 2: D + Lr							
		Wood Load Duration Factor = 1.25					
MAX. BENDING MOMENT lb-ft		281	632	1,585	1,936	29	80
MAX. DEFLECTION in		0.122	0.269	0.813	0.674	0.027	0.054
LEFT REACTION lbs		140	211	334	369	19	32
RIGHT REACTION lbs		140	211	334	369	19	32
LOAD CASE 3: D + L							
		Wood Load Duration Factor = 1.0					
MAX. BENDING MOMENT lb-ft		138	311	780	953	156	432
MAX. DEFLECTION in		0.048	0.106	0.320	0.266	0.118	0.234
LEFT REACTION lbs		69	104	164	182	104	173
RIGHT REACTION lbs		69	104	164	182	104	173
LOAD CASE 4: D + 0.75 (Lr + L)							
		Wood Load Duration Factor = 1.25					
MAX. BENDING MOMENT lb-ft		238	536	1,344	1,642	101	279
MAX. DEFLECTION in		0.103	0.228	0.690	0.572	0.095	0.189
LEFT REACTION lbs		149	223	354	391	84	140
RIGHT REACTION lbs		149	223	354	391	84	140
LOAD CASE 5: (1.0 + 0.14S _{DS})D + 0.7ΩE							
		Wood Load Duration Factor = 1.6					
MAX. BENDING MOMENT lb-ft		106	238	596	728	27	76
MAX. DEFLECTION in		0.049	0.108	0.326	0.271	0.028	0.055
LEFT REACTION lbs		70	106	167	185	24	41
RIGHT REACTION lbs		70	106	167	185	24	41

SIMPLY SUPPORTED BEAM DESIGN

		RR	RR1	RR2	RR3	CJ	CJ1
SPAN	ft	8.00	12.00	19.00	21.00	6.00	10.00
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	2x 6	2x 8	2x10	2x12	2x 4	2x 6
GRADE		No.2	No.2	No.2	No.2	No.2	No.2
LOAD CASE 6: $(1.0 + 0.105S_{Ds})D + 0.525\Omega E + 0.75(L + L_r)$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	201	451	1,131	1,382	82	229
MAX. DEFLECTION	in	0.093	0.205	0.619	0.514	0.083	0.165
LEFT REACTION	lbs	134	201	318	351	73	122
RIGHT REACTION	lbs	134	201	318	351	73	122
LOAD CASE 7: $(0.6 - 0.14S_{Ds})D + 0.7\Omega E$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	33	74	184	225	8	24
MAX. DEFLECTION	in	0.015	0.033	0.101	0.084	0.009	0.017
LEFT REACTION	lbs	22	33	52	57	8	13
RIGHT REACTION	lbs	22	33	52	57	8	13
CHECK BENDING STRESSES							
MAX. BENDING MOMENT	lb-ft	280.9	632.1	1,584.5	1,935.7	155.6	432.3
SECTION MODULUS, S	in ³	7.6	13.1	21.4	31.6	3.1	7.6
BENDING STRESS	psi	445.8	577.2	888.9	734.1	609.7	685.9
CHECK BENDING STRESS RATIO		33% OK	46% OK	78% OK	71% OK	39% OK	51% OK
CHECK SHEAR STRESSES							
MAX. SHEAR FORCE	lbs	149.0	223.46	353.81	391.05	103.74	172.90
SECTION AREA, A	in ²	8.25	10.88	13.88	16.88	5.25	8.25
SHEAR STRESS, fv	psi	27.09	30.82	38.25	34.76	29.64	31.44
CHECK SHEAR STRESS RATIO		13% OK	15% OK	18% OK	17% OK	14% OK	15% OK
CHECK DEFLECTION							
MOMENT OF INERTIA, I	in ⁴	20.80	47.63	98.93	177.98	5.36	20.80
MAX. DEFLECTION, DL	in	0.048	0.106	0.320	0.266	0.027	0.054
MAX. DEFLECTION, LL	in	0.074	0.163	0.493	0.409	0.090	0.180
DEFLECTION LIMITS, LL		L / 240	L / 360	L / 360	L / 360	L / 360	L / 360
CHECK DEFLECTION Δ_{LL}		< 0.4 OK	< 0.4 OK	< 0.63 OK	< 0.7 OK	< 0.2 OK	< 0.33 OK
MAX. DEFLECTION, DL+LL	in	0.122	0.269	0.813	0.674	0.118	0.234
DEFLECTION LIMITS, DL+LL		L / 180	L / 240	L / 240	L / 240	L / 240	L / 240
CHECK DEFLECTION $\Delta(DL+LL)$		< 0.53 OK	< 0.6 OK	< 0.95 OK	< 1.05 OK	< 0.3 OK	< 0.5 OK

SIMPLY SUPPORTED BEAM DESIGN

	CJ1	CJ2	CJ3	RDG1	RDG2	RDG3
SPAN ft	15.00	18.00	21.00	19.00	24.00	17.00
MATERIAL (WOOD, STEEL)	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD Prllm	WOOD Prllm
BEAM SECTION in	2x 8	2x10	2x12	6x14	5.25x14	5.25x14
GRADE	No.2	No.2	No.2	No.1	Prllm2.2E	Prllm2.2E
ALLOWABLE STRESSES						
REPETITIVE MEMBER	YES	YES	YES	NO	NO	NO
REPETITIVE FACTOR, Cr	1.15	1.15	1.15	1.00	1.00	1.00
SIZE FACTOR, Cf	1.20	1.10	1.00	0.98	1.00	1.00
BENDING STRESSES, F'b psi	1,242	1,139	1,035	1,327	2,900	2,900
SHEAR STRESSES, F'v psi	207	207	207	170	290	290
MODULUS OF ELASTICITY ksi	1,600	1,600	1,600	1,700	2,200	2,200
UNIFORM DISTRIBUTED LOAD OVER FULL SPAN						
ROOF TRIB. WIDTH ft				15.00	11.00	11.00
CEILING TRIB. WIDTH ft	1.33	1.33	1.33			
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft						
WALL TYPE (EXT, INT)						
AUTO CALC BEAM WEIGHT						
TOTAL DEAD LOAD (D plf	7.98	7.98	7.98	195.04	143.03	143.03
TOTAL ROOF LIVE LOAD (L ₁ plf	-	-	-	300.00	220.00	220.00
TOTAL FLOOR LIVE LOAD (L plf	26.60	26.60	26.60	-	-	-
PARTIALLY DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
CEILING TRIB. WIDTH ft						
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft						
WALL TYPE (EXT, INT)						
TOTAL DEAD LOAD (D plf	-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L ₁ plf	-	-	-	-	-	-
TOTAL FLOOR LIVE LOAD (L plf	-	-	-	-	-	-
FROM (LEFT SUPPORT) ft						
LENGTH OF LOAD ft						
TRIANGULAR DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
TOTAL DEAD LOAD (D plf	-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L ₁ plf	-	-	-	-	-	-
CONCENTRATED LOADS						
P1						
DEAD LOAD (P D) lbs						
ROOF LIVE LOAD (P Lr) lbs						
FLOOR LIVE LOAD (P L) lbs						
LATERAL, P (W or 0.7E) lbs						
DIST. (LEFT SUPPORT) ft						
P2						
DEAD LOAD (P D) lbs						
ROOF LIVE LOAD (P Lr) lbs						
FLOOR LIVE LOAD (P L) lbs						
LATERAL, P (W or 0.7E) lbs						
DIST. (LEFT SUPPORT) ft						

SIMPLY SUPPORTED BEAM DESIGN

		CJ1	CJ2	CJ3	RDG1	RDG2	RDG3
SPAN	ft	15.00	18.00	21.00	19.00	24.00	17.00
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD Prllm	WOOD Prllm
BEAM SECTION	in	2x 8	2x10	2x12	6x14	5.25x14	5.25x14
GRADE		No.2	No.2	No.2	No.1	Prllm2.2E	Prllm2.2E
P3							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
P4							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
LOAD CASE 1: D							
MAX. BENDING MOMENT lb-ft		224	323	440	8,801	10,298	5,167
MAX. DEFLECTION in		0.119	0.119	0.123	0.298	0.404	0.102
LEFT REACTION lbs		60	72	84	1,853	1,716	1,216
RIGHT REACTION lbs		60	72	84	1,853	1,716	1,216
LOAD CASE: Lr							
MAX. BENDING MOMENT lb-ft		-	-	-	13,538	15,840	7,948
MAX. DEFLECTION in		-	-	-	0.459	0.622	0.157
LEFT REACTION lbs		-	-	-	2,850	2,640	1,870
RIGHT REACTION lbs		-	-	-	2,850	2,640	1,870
LOAD CASE: L							
MAX. BENDING MOMENT lb-ft		748	1,077	1,466	-	-	-
MAX. DEFLECTION in		0.398	0.397	0.409	-	-	-
LEFT REACTION lbs		200	239	279	-	-	-
RIGHT REACTION lbs		200	239	279	-	-	-
LOAD CASE 2: D + Lr					Wood Load Duration Factor = 1.25		
MAX. BENDING MOMENT lb-ft		180	259	352	17,871	20,910	10,491
MAX. DEFLECTION in		0.119	0.119	0.123	0.757	1.026	0.258
LEFT REACTION lbs		48	57	67	3,762	3,485	2,469
RIGHT REACTION lbs		48	57	67	3,762	3,485	2,469
LOAD CASE 3: D + L					Wood Load Duration Factor = 1.0		
MAX. BENDING MOMENT lb-ft		973	1,400	1,906	8,801	10,298	5,167
MAX. DEFLECTION in		0.517	0.516	0.531	0.298	0.404	0.102
LEFT REACTION lbs		259	311	363	1,853	1,716	1,216
RIGHT REACTION lbs		259	311	363	1,853	1,716	1,216
LOAD CASE 4: D + 0.75 (Lr + L)					Wood Load Duration Factor = 1.25		
MAX. BENDING MOMENT lb-ft		628	905	1,232	15,163	17,742	8,902
MAX. DEFLECTION in		0.417	0.417	0.429	0.642	0.871	0.219
LEFT REACTION lbs		209	251	293	3,990	3,696	2,618
RIGHT REACTION lbs		209	251	293	3,990	3,696	2,618
LOAD CASE 5: (1.0 + 0.14S _{DS})D + 0.7ΩE					Wood Load Duration Factor = 1.6		
MAX. BENDING MOMENT lb-ft		171	247	336	6,720	7,864	3,945
MAX. DEFLECTION in		0.121	0.121	0.125	0.304	0.412	0.104
LEFT REACTION lbs		61	73	85	1,886	1,747	1,238
RIGHT REACTION lbs		61	73	85	1,886	1,747	1,238

SIMPLY SUPPORTED BEAM DESIGN

		CJ1	CJ2	CJ3	RDG1	RDG2	RDG3
SPAN	ft	15.00	18.00	21.00	19.00	24.00	17.00
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD Prllm	WOOD Prllm
BEAM SECTION	in	2x 8	2x10	2x12	6x14	5.25x14	5.25x14
GRADE		No.2	No.2	No.2	No.1	Prllm2.2E	Prllm2.2E
LOAD CASE 6: $(1.0 + 0.105S_{Ds})D + 0.525\Omega E + 0.75(L + L_r)$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	514	741	1,008	12,761	14,932	7,492
MAX. DEFLECTION	in	0.364	0.364	0.375	0.577	0.782	0.197
LEFT REACTION	lbs	183	219	256	3,582	3,318	2,350
RIGHT REACTION	lbs	183	219	256	3,582	3,318	2,350
LOAD CASE 7: $(0.6 - 0.14S_{Ds})D + 0.7\Omega E$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	53	76	104	2,081	2,434	1,221
MAX. DEFLECTION	in	0.038	0.038	0.039	0.094	0.127	0.032
LEFT REACTION	lbs	19	23	26	584	541	383
RIGHT REACTION	lbs	19	23	26	584	541	383
CHECK BENDING STRESSES							
MAX. BENDING MOMENT	lb-ft	972.6	1,400.5	1,906.2	17,870.9	20,910.4	10,491.5
SECTION MODULUS, S	in ³	13.1	21.4	31.6	167.1	171.5	171.5
BENDING STRESS	psi	888.1	785.7	723.0	1,283.7	1,463.1	734.1
CHECK BENDING STRESS RATIO		72% OK	69% OK	70% OK	97% OK	50% OK	25% OK
CHECK SHEAR STRESSES							
MAX. SHEAR FORCE	lbs	259.4	311.22	363.09	3,990.36	3,696.33	2,618.23
SECTION AREA, A	in ²	10.88	13.88	16.88	74.25	73.50	73.50
SHEAR STRESS, fv	psi	35.77	33.65	32.27	80.61	75.44	53.43
CHECK SHEAR STRESS RATIO		17% OK	16% OK	16% OK	47% OK	26% OK	18% OK
CHECK DEFLECTION							
MOMENT OF INERTIA, I	in ⁴	47.63	98.93	177.98	1,127.67	1,200.50	1,200.50
MAX. DEFLECTION, DL	in	0.119	0.119	0.123	0.298	0.404	0.102
MAX. DEFLECTION, LL	in	0.398	0.397	0.409	0.459	0.622	0.157
DEFLECTION LIMITS, LL		L / 360	L / 360	L / 360	L / 360	L / 360	L / 360
CHECK DEFLECTION ΔLL		< 0.5 OK	< 0.6 OK	< 0.7 OK	< 0.63 OK	< 0.8 OK	< 0.57 OK
MAX. DEFLECTION, DL+LL	in	0.517	0.516	0.531	0.757	1.026	0.258
DEFLECTION LIMITS, DL+LL		L / 240	L / 240	L / 240	L / 240	L / 240	L / 240
CHECK DEFLECTION Δ(DL+LL)		< 0.75 OK	< 0.9 OK	< 1.05 OK	< 0.95 OK	< 1.2 OK	< 0.85 OK

SIMPLY SUPPORTED BEAM DESIGN

		RB1	RDG4	CB1	CB2	RDG5	RDG6
SPAN	ft	12.00	26.50	15.00	5.50	17.50	16.00
MATERIAL (WOOD, STEEL)		WOOD Prllm	WOOD Prllm	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	3.5x11.25	5.25x16	4x12	6x12	6x14	6x12
GRADE		Prllm2.2E	Prllm2.2E	No.1	No.1	No.1	No.1
ALLOWABLE STRESSES							
REPETITIVE MEMBER		NO	NO	NO	NO	NO	NO
REPETITIVE FACTOR, Cr		1.00	1.00	1.00	1.00	1.00	1.00
SIZE FACTOR, Cf		1.00	1.00	1.10	1.00	0.98	1.00
BENDING STRESSES, F'b psi		2,900	2,900	1,100	1,350	1,327	1,350
SHEAR STRESSES, F'v psi		290	290	180	170	170	170
MODULUS OF ELASTICITY ksi		2,200	2,200	1,700	1,700	1,700	1,700
UNIFORM DISTRIBUTED LOAD OVER FULL SPAN							
ROOF TRIB. WIDTH	ft	1.33	11.00	7.50		13.00	6.50
CEILING TRIB. WIDTH	ft				1.33		
FLOOR TRIB. WIDTH	ft						
DECK TRIB. WIDTH	ft						
WALL HEIGHT	ft						
WALL TYPE (EXT, INT)							
AUTO CALC BEAM WEIGHT							
TOTAL DEAD LOAD (D plf		17.33	143.03	97.52	7.98	169.03	84.52
TOTAL ROOF LIVE LOAD (L _r plf		26.66	220.00	150.00	-	260.00	130.00
TOTAL FLOOR LIVE LOAD (L plf		-	-	-	26.60	-	-
PARTIALLY DISTRIBUTED LOAD							
ROOF TRIB. WIDTH	ft						
CEILING TRIB. WIDTH	ft						
FLOOR TRIB. WIDTH	ft						
DECK TRIB. WIDTH	ft						
WALL HEIGHT	ft						
WALL TYPE (EXT, INT)							
TOTAL DEAD LOAD (D plf		-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf		-	-	-	-	-	-
TOTAL FLOOR LIVE LOAD (L plf		-	-	-	-	-	-
FROM (LEFT SUPPORT)	ft						
LENGTH OF LOAD	ft						
TRIANGULAR DISTRIBUTED LOAD							
ROOF TRIB. WIDTH	ft						
TOTAL DEAD LOAD (D plf		-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf		-	-	-	-	-	-
CONCENTRATED LOADS							
P1		RDG2			2-RDG1		
DEAD LOAD (P D) lbs		1,716.3			3,705.7		
ROOF LIVE LOAD (P L _r) lbs		2,640.0			5,700.0		
FLOOR LIVE LOAD (P L) lbs		-			-		
LATERAL, P (W or 0.7E) lbs		-			-		
DIST. (LEFT SUPPORT) ft		2.50			2.00		
P2		RDG3					
DEAD LOAD (P D) lbs		1,215.7					
ROOF LIVE LOAD (P L _r) lbs		1,870.0					
FLOOR LIVE LOAD (P L) lbs		-					
LATERAL, P (W or 0.7E) lbs		-					
DIST. (LEFT SUPPORT) ft		2.50					

SIMPLY SUPPORTED BEAM DESIGN

		RB1	RDG4	CB1	CB2	RDG5	RDG6
SPAN	ft	12.00	26.50	15.00	5.50	17.50	16.00
MATERIAL (WOOD, STEEL)		WOOD Prllm	WOOD Prllm	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	3.5x11.25	5.25x16	4x12	6x12	6x14	6x12
GRADE		Prllm2.2E	Prllm2.2E	No.1	No.1	No.1	No.1
P3							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
P4							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
LOAD CASE 1: D							
MAX. BENDING MOMENT lb-ft		5,932	12,555	2,743	4,697	6,471	2,705
MAX. DEFLECTION in		0.129	0.403	0.157	0.017	0.186	0.105
LEFT REACTION lbs		2,425	1,895	731	2,380	1,479	676
RIGHT REACTION lbs		715	1,895	731	1,369	1,479	676
LOAD CASE: Lr							
MAX. BENDING MOMENT lb-ft		9,124	19,312	4,219	7,182	9,953	4,160
MAX. DEFLECTION in		0.198	0.619	0.242	0.026	0.286	0.162
LEFT REACTION lbs		3,730	2,915	1,125	3,627	2,275	1,040
RIGHT REACTION lbs		1,100	2,915	1,125	2,073	2,275	1,040
LOAD CASE: L							
MAX. BENDING MOMENT lb-ft		-	-	-	101	-	-
MAX. DEFLECTION in		-	-	-	0.000	-	-
LEFT REACTION lbs		-	-	-	73	-	-
RIGHT REACTION lbs		-	-	-	73	-	-
LOAD CASE 2: D + Lr							
		Wood Load Duration Factor = 1.25					
MAX. BENDING MOMENT lb-ft		12,044	25,494	5,569	9,503	13,139	5,492
MAX. DEFLECTION in		0.326	1.022	0.399	0.043	0.472	0.267
LEFT REACTION lbs		4,924	3,848	1,485	4,806	3,003	1,373
RIGHT REACTION lbs		1,452	3,848	1,485	2,754	3,003	1,373
LOAD CASE 3: D + L							
		Wood Load Duration Factor = 1.0					
MAX. BENDING MOMENT lb-ft		5,932	12,555	2,743	4,790	6,471	2,705
MAX. DEFLECTION in		0.129	0.403	0.157	0.018	0.186	0.105
LEFT REACTION lbs		2,425	1,895	731	2,453	1,479	676
RIGHT REACTION lbs		715	1,895	731	1,443	1,479	676
LOAD CASE 4: D + 0.75 (Lr + L)							
		Wood Load Duration Factor = 1.25					
MAX. BENDING MOMENT lb-ft		10,220	21,631	4,725	8,122	11,148	4,660
MAX. DEFLECTION in		0.277	0.867	0.339	0.037	0.401	0.226
LEFT REACTION lbs		5,223	4,081	1,575	5,155	3,185	1,456
RIGHT REACTION lbs		1,539	4,081	1,575	2,979	3,185	1,456
LOAD CASE 5: (1.0 + 0.14S_{ds})D + 0.7ΩE							
		Wood Load Duration Factor = 1.6					
MAX. BENDING MOMENT lb-ft		4,529	9,587	2,094	3,587	4,941	2,065
MAX. DEFLECTION in		0.131	0.410	0.160	0.017	0.189	0.107
LEFT REACTION lbs		2,469	1,929	745	2,423	1,506	688
RIGHT REACTION lbs		728	1,929	745	1,394	1,506	688

SIMPLY SUPPORTED BEAM DESIGN

		RB1	RDG4	CB1	CB2	RDG5	RDG6
SPAN	ft	12.00	26.50	15.00	5.50	17.50	16.00
MATERIAL (WOOD, STEEL)		WOOD Prllm	WOOD Prllm	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	3.5x11.25	5.25x16	4x12	6x12	6x14	6x12
GRADE		Prllm2.2E	Prllm2.2E	No.1	No.1	No.1	No.1
LOAD CASE 6: $(1.0 + 0.105S_{Ds})D + 0.525\Omega E + 0.75(L + L_r)$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	8,601	18,205	3,977	6,834	9,382	3,921
MAX. DEFLECTION	in	0.248	0.778	0.304	0.033	0.360	0.203
LEFT REACTION	lbs	4,689	3,664	1,414	4,626	2,859	1,307
RIGHT REACTION	lbs	1,382	3,664	1,414	2,672	2,859	1,307
LOAD CASE 7: $(0.6 - 0.14S_{Ds})D + 0.7\Omega E$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	1,402	2,968	648	1,110	1,530	639
MAX. DEFLECTION	in	0.041	0.127	0.050	0.005	0.059	0.033
LEFT REACTION	lbs	764	597	231	750	466	213
RIGHT REACTION	lbs	225	597	231	432	466	213
CHECK BENDING STRESSES							
MAX. BENDING MOMENT	lb-ft	12,044.4	25,493.6	5,569.2	9,503.2	13,139.1	5,491.6
SECTION MODULUS, S	in ³	73.8	224.0	73.8	121.2	167.1	121.2
BENDING STRESS	psi	1,957.7	1,365.7	905.2	940.7	943.8	543.6
CHECK BENDING STRESS RATIO		68% OK	47% OK	82% OK	70% OK	71% OK	40% OK
CHECK SHEAR STRESSES							
MAX. SHEAR FORCE	lbs	5,223.0	4,081.36	1,575.14	5,155.44	3,185.28	1,456.13
SECTION AREA, A	in ²	39.38	84.00	39.38	63.25	74.25	63.25
SHEAR STRESS, fv	psi	198.97	72.88	60.01	122.26	64.35	34.53
CHECK SHEAR STRESS RATIO		69% OK	25% OK	33% OK	72% OK	38% OK	20% OK
CHECK DEFLECTION							
MOMENT OF INERTIA, I	in ⁴	415.28	1,792.00	415.28	697.07	1,127.67	697.07
MAX. DEFLECTION, DL	in	0.129	0.403	0.157	0.017	0.186	0.105
MAX. DEFLECTION, LL	in	0.198	0.619	0.242	0.026	0.286	0.162
DEFLECTION LIMITS, LL		L / 360	L / 360	L / 360	L / 360	L / 360	L / 360
CHECK DEFLECTION ALL		< 0.4 OK	< 0.88 OK	< 0.5 OK	< 0.18 OK	< 0.58 OK	< 0.53 OK
MAX. DEFLECTION, DL+LL	in	0.326	1.022	0.399	0.043	0.472	0.267
DEFLECTION LIMITS, DL+LL		L / 240	L / 240	L / 240	L / 240	L / 240	L / 240
CHECK DEFLECTION Δ(DL+LL)		< 0.6 OK	< 1.33 OK	< 0.75 OK	< 0.28 OK	< 0.88 OK	< 0.8 OK

SIMPLY SUPPORTED BEAM DESIGN

		RB2	RB3	RB4	RDG7	RDG8	CB3
SPAN	ft	13.00	16.00	13.00	14.00	9.00	5.00
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	4x12	4x12	6x12	6x14	6x14	6x12
GRADE		No.1	No.1	No.1	No.1	No.1	No.2
ALLOWABLE STRESSES							
REPETITIVE MEMBER		NO	NO	NO	NO	NO	NO
REPETITIVE FACTOR, Cr		1.00	1.00	1.00	1.00	1.00	1.00
SIZE FACTOR, Cf		1.10	1.10	1.00	0.98	0.98	1.00
BENDING STRESSES, F'b psi		1,100	1,100	1,350	1,327	1,327	875
SHEAR STRESSES, F'v psi		180	180	170	170	170	170
MODULUS OF ELASTICITY ksi		1,700	1,700	1,700	1,700	1,700	1,600
UNIFORM DISTRIBUTED LOAD OVER FULL SPAN							
ROOF TRIB. WIDTH	ft	1.33	6.00	1.33	21.00	21.00	
CEILING TRIB. WIDTH	ft						11.00
FLOOR TRIB. WIDTH	ft						
DECK TRIB. WIDTH	ft						
WALL HEIGHT	ft						
WALL TYPE (EXT, INT)							
AUTO CALC BEAM WEIGHT							
TOTAL DEAD LOAD (D plf		17.29	78.02	17.29	273.05	273.05	66.00
TOTAL ROOF LIVE LOAD (L _r plf		26.60	120.00	26.60	420.00	420.00	-
TOTAL FLOOR LIVE LOAD (L plf		-	-	-	-	-	220.00
PARTIALLY DISTRIBUTED LOAD							
ROOF TRIB. WIDTH	ft						
CEILING TRIB. WIDTH	ft						
FLOOR TRIB. WIDTH	ft						
DECK TRIB. WIDTH	ft						
WALL HEIGHT	ft						
WALL TYPE (EXT, INT)							
TOTAL DEAD LOAD (D plf		-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf		-	-	-	-	-	-
TOTAL FLOOR LIVE LOAD (L plf		-	-	-	-	-	-
FROM (LEFT SUPPORT)	ft						
LENGTH OF LOAD	ft						
TRIANGULAR DISTRIBUTED LOAD							
ROOF TRIB. WIDTH	ft						
TOTAL DEAD LOAD (D plf		-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf		-	-	-	-	-	-
CONCENTRATED LOADS							
P1		RDG6		RDG6			RDG7+RDG8
DEAD LOAD (P D) lbs		676.1		845.2			3,140.1
ROOF LIVE LOAD (P L _r) lbs		1,040.0		1,300.0			4,830.0
FLOOR LIVE LOAD (P L) lbs		-		-			-
LATERAL, P (W or 0.7E) lbs		-		-			-
DIST. (LEFT SUPPORT) ft		6.50		6.50			2.50
P2							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P L _r) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							

SIMPLY SUPPORTED BEAM DESIGN

		RB2	RB3	RB4	RDG7	RDG8	CB3
SPAN	ft	13.00	16.00	13.00	14.00	9.00	5.00
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	4x12	4x12	6x12	6x14	6x14	6x12
GRADE		No.1	No.1	No.1	No.1	No.1	No.2
P3							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
P4							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
LOAD CASE 1: D							
MAX. BENDING MOMENT lb-ft		2,563	2,496	3,112	6,690	2,765	4,131
MAX. DEFLECTION in		0.091	0.163	0.066	0.123	0.021	0.014
LEFT REACTION lbs		450	624	535	1,911	1,229	1,735
RIGHT REACTION lbs		450	624	535	1,911	1,229	1,735
LOAD CASE: Lr							
MAX. BENDING MOMENT lb-ft		3,942	3,840	4,787	10,290	4,253	6,038
MAX. DEFLECTION in		0.141	0.251	0.101	0.189	0.032	0.019
LEFT REACTION lbs		693	960	823	2,940	1,890	2,415
RIGHT REACTION lbs		693	960	823	2,940	1,890	2,415
LOAD CASE: L							
MAX. BENDING MOMENT lb-ft		-	-	-	-	-	688
MAX. DEFLECTION in		-	-	-	-	-	0.003
LEFT REACTION lbs		-	-	-	-	-	550
RIGHT REACTION lbs		-	-	-	-	-	550
LOAD CASE 2: D + Lr					Wood Load Duration Factor = 1.25		
MAX. BENDING MOMENT lb-ft		5,204	5,069	6,319	13,584	5,614	8,135
MAX. DEFLECTION in		0.232	0.414	0.167	0.312	0.053	0.033
LEFT REACTION lbs		915	1,267	1,086	3,881	2,495	3,320
RIGHT REACTION lbs		915	1,267	1,086	3,881	2,495	3,320
LOAD CASE 3: D + L					Wood Load Duration Factor = 1.0		
MAX. BENDING MOMENT lb-ft		2,563	2,496	3,112	6,690	2,765	4,819
MAX. DEFLECTION in		0.091	0.163	0.066	0.123	0.021	0.016
LEFT REACTION lbs		450	624	535	1,911	1,229	2,285
RIGHT REACTION lbs		450	624	535	1,911	1,229	2,285
LOAD CASE 4: D + 0.75 (Lr + L)					Wood Load Duration Factor = 1.25		
MAX. BENDING MOMENT lb-ft		4,415	4,301	5,362	11,526	4,763	7,340
MAX. DEFLECTION in		0.197	0.351	0.142	0.265	0.045	0.030
LEFT REACTION lbs		970	1,344	1,152	4,116	2,646	3,959
RIGHT REACTION lbs		970	1,344	1,152	4,116	2,646	3,959
LOAD CASE 5: (1.0 + 0.14S _{ds})D + 0.7ΩE					Wood Load Duration Factor = 1.6		
MAX. BENDING MOMENT lb-ft		1,957	1,906	2,376	5,108	2,111	3,155
MAX. DEFLECTION in		0.093	0.166	0.067	0.125	0.021	0.014
LEFT REACTION lbs		459	635	545	1,946	1,251	1,767
RIGHT REACTION lbs		459	635	545	1,946	1,251	1,767

SIMPLY SUPPORTED BEAM DESIGN

		RB2	RB3	RB4	RDG7	RDG8	CB3
SPAN	ft	13.00	16.00	13.00	14.00	9.00	5.00
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	4x12	4x12	6x12	6x14	6x14	6x12
GRADE		No.1	No.1	No.1	No.1	No.1	No.2
LOAD CASE 6: $(1.0 + 0.105S_{Ds})D + 0.525\Omega E + 0.75(L + L_r)$		Wood Load Duration Factor = 1.6					
MAX. BENDING MOMENT	lb-ft	3,716	3,620	4,512	9,700	4,009	6,164
MAX. DEFLECTION	in	0.177	0.315	0.127	0.238	0.041	0.027
LEFT REACTION	lbs	871	1,207	1,034	3,695	2,375	3,539
RIGHT REACTION	lbs	871	1,207	1,034	3,695	2,375	3,539
LOAD CASE 7: $(0.6 - 0.14S_{Ds})D + 0.7\Omega E$		Wood Load Duration Factor = 1.6					
MAX. BENDING MOMENT	lb-ft	606	590	736	1,581	654	977
MAX. DEFLECTION	in	0.029	0.051	0.021	0.039	0.007	0.004
LEFT REACTION	lbs	142	197	169	602	387	547
RIGHT REACTION	lbs	142	197	169	602	387	547
CHECK BENDING STRESSES							
MAX. BENDING MOMENT	lb-ft	5,203.7	5,069.2	6,319.2	13,583.8	5,613.7	8,135.1
SECTION MODULUS, S	in ³	73.8	73.8	121.2	167.1	167.1	121.2
BENDING STRESS	psi	845.8	823.9	625.5	975.7	403.2	805.3
CHECK BENDING STRESS RATIO		77% OK	75% OK	46% OK	74% OK	30% OK	92% OK
CHECK SHEAR STRESSES							
MAX. SHEAR FORCE	lbs	970.1	1,344.12	1,152.16	4,116.37	2,646.24	3,958.80
SECTION AREA, A	in ²	39.38	39.38	63.25	74.25	74.25	63.25
SHEAR STRESS, fv	psi	36.96	51.20	27.32	83.16	53.46	93.88
CHECK SHEAR STRESS RATIO		21% OK	28% OK	16% OK	49% OK	31% OK	55% OK
CHECK DEFLECTION							
MOMENT OF INERTIA, I	in ⁴	415.28	415.28	697.07	1,127.67	1,127.67	697.07
MAX. DEFLECTION, DL	in	0.091	0.163	0.066	0.123	0.021	0.014
MAX. DEFLECTION, LL	in	0.141	0.251	0.101	0.189	0.032	0.019
DEFLECTION LIMITS, LL		L / 360	L / 360	L / 360	L / 360	L / 360	L / 360
CHECK DEFLECTION Δ_{LL}		< 0.43 OK	< 0.53 OK	< 0.43 OK	< 0.47 OK	< 0.3 OK	< 0.17 OK
MAX. DEFLECTION, DL+LL	in	0.232	0.414	0.167	0.312	0.053	0.033
DEFLECTION LIMITS, DL+LL		L / 240	L / 240	L / 240	L / 240	L / 240	L / 240
CHECK DEFLECTION $\Delta(DL+LL)$		< 0.65 OK	< 0.8 OK	< 0.65 OK	< 0.7 OK	< 0.45 OK	< 0.25 OK

SIMPLY SUPPORTED BEAM DESIGN

	CB4	RB5	HDR1	RB6	RB7	HDR2
SPAN ft	5.00	14.00	16.50	13.00	24.00	3.00
MATERIAL (WOOD, STEEL)	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD Prllm	WOOD DF-L
BEAM SECTION in	4x12	6x12	6x16	6x12	5.25x14	6x 8
GRADE	No.1	No.1	No.1	No.1	Prllm2.2E	No.2
ALLOWABLE STRESSES						
REPETITIVE MEMBER	NO	NO	NO	NO	NO	NO
REPETITIVE FACTOR, Cr	1.00	1.00	1.00	1.00	1.00	1.00
SIZE FACTOR, Cf	1.10	1.00	0.97	1.00	1.00	1.00
BENDING STRESSES, F'b psi	1,100	1,350	1,308	1,350	2,900	875
SHEAR STRESSES, F'v psi	180	170	170	170	290	170
MODULUS OF ELASTICITY ksi	1,700	1,700	1,700	1,700	2,200	1,600
UNIFORM DISTRIBUTED LOAD OVER FULL SPAN						
ROOF TRIB. WIDTH ft	5.00	0.67	6.00	9.00	6.50	11.00
CEILING TRIB. WIDTH ft	4.50					0.67
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft			4.00		4.00	4.00
WALL TYPE (EXT, INT)			INT		EXT	EXT
AUTO CALC BEAM WEIGHT						
TOTAL DEAD LOAD (D plf	92.01	8.71	118.02	117.02	156.52	219.05
TOTAL ROOF LIVE LOAD (L _r plf	100.00	13.40	120.00	180.00	130.00	220.00
TOTAL FLOOR LIVE LOAD (L plf	90.00	-	-	-	-	13.40
PARTIALLY DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
CEILING TRIB. WIDTH ft						
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft						
WALL TYPE (EXT, INT)						
TOTAL DEAD LOAD (D plf	-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf	-	-	-	-	-	-
TOTAL FLOOR LIVE LOAD (L plf	-	-	-	-	-	-
FROM (LEFT SUPPORT) ft						
LENGTH OF LOAD ft						
TRIANGULAR DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
TOTAL DEAD LOAD (D plf	-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf	-	-	-	-	-	-
CONCENTRATED LOADS						
P1	RDG8	RDG5	RDG5			
DEAD LOAD (P D) lbs	1,228.7	1,479.0	1,479.0			
ROOF LIVE LOAD (P L _r) lbs	1,890.0	2,275.0	2,275.0			
FLOOR LIVE LOAD (P L) lbs	-	-	-			
LATERAL, P (W or 0.7E) lbs	-	-	-			
DIST. (LEFT SUPPORT) ft	2.50	7.00	8.25			
P2						
DEAD LOAD (P D) lbs						
ROOF LIVE LOAD (P L _r) lbs						
FLOOR LIVE LOAD (P L) lbs						
LATERAL, P (W or 0.7E) lbs						
DIST. (LEFT SUPPORT) ft						

SIMPLY SUPPORTED BEAM DESIGN

		CB4	RB5	HDR1	RB6	RB7	HDR2
SPAN	ft	5.00	14.00	16.50	13.00	24.00	3.00
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD Prllm	WOOD DF-L
BEAM SECTION	in	4x12	6x12	6x16	6x12	5.25x14	6x 8
GRADE		No.1	No.1	No.1	No.1	Prllm2.2E	No.2
P3							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
P4							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
LOAD CASE 1: D							
MAX. BENDING MOMENT lb-ft		1,823	5,390	10,117	2,472	11,269	246
MAX. DEFLECTION in		0.010	0.130	0.150	0.063	0.442	0.001
LEFT REACTION lbs		844	800	1,713	761	1,878	329
RIGHT REACTION lbs		844	800	1,713	761	1,878	329
LOAD CASE: Lr							
MAX. BENDING MOMENT lb-ft		2,675	8,291	13,468	3,803	9,360	248
MAX. DEFLECTION in		0.014	0.199	0.196	0.098	0.367	0.001
LEFT REACTION lbs		1,195	1,231	2,128	1,170	1,560	330
RIGHT REACTION lbs		1,195	1,231	2,128	1,170	1,560	330
LOAD CASE: L							
MAX. BENDING MOMENT lb-ft		281	-	-	-	-	15
MAX. DEFLECTION in		0.002	-	-	-	-	0.000
LEFT REACTION lbs		225	-	-	-	-	20
RIGHT REACTION lbs		225	-	-	-	-	20
LOAD CASE 2: D + Lr							Wood Load Duration Factor = 1.25
MAX. BENDING MOMENT lb-ft		3,599	10,945	18,868	5,020	16,503	395
MAX. DEFLECTION in		0.024	0.329	0.346	0.161	0.810	0.003
LEFT REACTION lbs		1,632	1,625	3,073	1,545	2,751	527
RIGHT REACTION lbs		1,632	1,625	3,073	1,545	2,751	527
LOAD CASE 3: D + L							Wood Load Duration Factor = 1.0
MAX. BENDING MOMENT lb-ft		2,105	5,390	10,117	2,472	11,269	262
MAX. DEFLECTION in		0.011	0.130	0.150	0.063	0.442	0.001
LEFT REACTION lbs		1,069	800	1,713	761	1,878	349
RIGHT REACTION lbs		1,069	800	1,713	761	1,878	349
LOAD CASE 4: D + 0.75 (Lr + L)							Wood Load Duration Factor = 1.25
MAX. BENDING MOMENT lb-ft		3,233	9,287	16,175	4,259	14,631	355
MAX. DEFLECTION in		0.022	0.279	0.297	0.137	0.718	0.002
LEFT REACTION lbs		1,909	1,724	3,309	1,638	3,048	591
RIGHT REACTION lbs		1,909	1,724	3,309	1,638	3,048	591
LOAD CASE 5: (1.0 + 0.14S _{ds})D + 0.7ΩE							Wood Load Duration Factor = 1.6
MAX. BENDING MOMENT lb-ft		1,392	4,116	7,726	1,888	8,605	188
MAX. DEFLECTION in		0.010	0.132	0.153	0.065	0.450	0.001
LEFT REACTION lbs		860	815	1,744	774	1,912	335
RIGHT REACTION lbs		860	815	1,744	774	1,912	335

SIMPLY SUPPORTED BEAM DESIGN

		CB4	RB5	HDR1	RB6	RB7	HDR2
SPAN	ft	5.00	14.00	16.50	13.00	24.00	3.00
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD Prllm	WOOD DF-L
BEAM SECTION	in	4x12	6x12	6x16	6x12	5.25x14	6x 8
GRADE		No.1	No.1	No.1	No.1	Prllm2.2E	No.2
LOAD CASE 6: $(1.0 + 0.105S_{Ds})D + 0.525\Omega E + 0.75(L + L_r)$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	2,715	7,815	13,688	3,584	12,602	303
MAX. DEFLECTION	in	0.019	0.251	0.268	0.123	0.660	0.002
LEFT REACTION	lbs	1,708	1,548	2,995	1,471	2,800	538
RIGHT REACTION	lbs	1,708	1,548	2,995	1,471	2,800	538
LOAD CASE 7: $(0.6 - 0.14S_{Ds})D + 0.7\Omega E$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	431	1,274	2,392	584	2,664	58
MAX. DEFLECTION	in	0.003	0.041	0.047	0.020	0.139	0.000
LEFT REACTION	lbs	266	252	540	240	592	104
RIGHT REACTION	lbs	266	252	540	240	592	104
CHECK BENDING STRESSES							
MAX. BENDING MOMENT	lb-ft	3,598.8	10,944.7	18,868.3	5,019.7	16,503.3	395.1
SECTION MODULUS, S	in ³	73.8	121.2	220.2	121.2	171.5	51.6
BENDING STRESS	psi	584.9	1,083.4	1,028.1	496.9	1,154.8	92.0
CHECK BENDING STRESS RATIO		53% OK	80% OK	79% OK	37% OK	40% OK	11% OK
CHECK SHEAR STRESSES							
MAX. SHEAR FORCE	lbs	1,909.4	1,723.97	3,308.77	1,638.15	3,048.20	591.15
SECTION AREA, A	in ²	39.38	63.25	85.25	63.25	73.50	41.25
SHEAR STRESS, fv	psi	72.74	40.88	58.22	38.85	62.21	21.50
CHECK SHEAR STRESS RATIO		40% OK	24% OK	34% OK	23% OK	21% OK	13% OK
CHECK DEFLECTION							
MOMENT OF INERTIA, I	in ⁴	415.28	697.07	1,706.78	697.07	1,200.50	193.36
MAX. DEFLECTION, DL	in	0.010	0.130	0.150	0.063	0.442	0.001
MAX. DEFLECTION, LL	in	0.014	0.199	0.196	0.098	0.367	0.001
DEFLECTION LIMITS, LL		L / 360	L / 360	L / 360	L / 360	L / 360	L / 360
CHECK DEFLECTION Δ_{LL}		< 0.17 OK	< 0.47 OK	< 0.55 OK	< 0.43 OK	< 0.8 OK	< 0.1 OK
MAX. DEFLECTION, DL+LL	in	0.024	0.329	0.346	0.161	0.810	0.003
DEFLECTION LIMITS, DL+LL		L / 240	L / 240	L / 240	L / 240	L / 240	L / 240
CHECK DEFLECTION $\Delta(DL+LL)$		< 0.25 OK	< 0.7 OK	< 0.83 OK	< 0.65 OK	< 1.2 OK	< 0.15 OK

SIMPLY SUPPORTED BEAM DESIGN

	HDR3	HDR4	RB	RB	RB	RB
SPAN ft	3.50	6.00	19.00			
MATERIAL (WOOD, STEEL)	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD Prllm	WOOD DF-L	WOOD DF-L
BEAM SECTION in	6x10	6x12	6x12	5.25x14	4x14	4x14
GRADE	No.2	No.2	No.1	Prllm2.2E	No.1	No.1
ALLOWABLE STRESSES						
REPETITIVE MEMBER	NO	NO	NO	NO	NO	NO
REPETITIVE FACTOR, Cr	1.00	1.00	1.00			
SIZE FACTOR, Cf	1.00	1.00	1.00			
BENDING STRESSES, F'b psi	875	875	1,350			
SHEAR STRESSES, F'v psi	170	170	170			
MODULUS OF ELASTICITY ksi	1,600	1,600	1,700			
UNIFORM DISTRIBUTED LOAD OVER FULL SPAN						
ROOF TRIB. WIDTH ft	1.33	6.50				
CEILING TRIB. WIDTH ft			1.00			
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft	3.50	3.50				
WALL TYPE (EXT, INT)	EXT	EXT				
AUTO CALC BEAM WEIGHT						
TOTAL DEAD LOAD (D plf	80.29	147.52	6.00			
TOTAL ROOF LIVE LOAD (L _r plf	26.60	130.00	-			
TOTAL FLOOR LIVE LOAD (L plf	-	-	20.00			
PARTIALLY DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
CEILING TRIB. WIDTH ft						
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft						
WALL TYPE (EXT, INT)						
TOTAL DEAD LOAD (D plf	-	-	-			
TOTAL ROOF LIVE LOAD (L _r plf	-	-	-			
TOTAL FLOOR LIVE LOAD (L plf	-	-	-			
FROM (LEFT SUPPORT) ft						
LENGTH OF LOAD ft						
TRIANGULAR DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
TOTAL DEAD LOAD (D plf	-	-	-			
TOTAL ROOF LIVE LOAD (L _r plf	-	-	-			
CONCENTRATED LOADS						
P1	RDG1	RDG3				
DEAD LOAD (P D) lbs	1,852.9	1,215.7				
ROOF LIVE LOAD (P L _r) lbs	2,850.0	1,870.0				
FLOOR LIVE LOAD (P L) lbs	-	-				
LATERAL, P (W or 0.7E) lbs	-	-				
DIST. (LEFT SUPPORT) ft	1.00	3.00				
P2						
DEAD LOAD (P D) lbs						
ROOF LIVE LOAD (P L _r) lbs						
FLOOR LIVE LOAD (P L) lbs						
LATERAL, P (W or 0.7E) lbs						
DIST. (LEFT SUPPORT) ft						

SIMPLY SUPPORTED BEAM DESIGN

		HDR3	HDR4	RB	RE	RE	RE
SPAN	ft	3.50	6.00	19.00			
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD Prllm	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	6x10	6x12	6x12	5.25x14	4x14	4x14
GRADE		No.2	No.2	No.1	Prllm2.2E	No.1	No.1
P3							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
P4							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
LOAD CASE 1: D							
MAX. BENDING MOMENT lb-ft		1,400	2,487	271			
MAX. DEFLECTION in		0.004	0.012	0.015			
LEFT REACTION lbs		1,464	1,050	57			
RIGHT REACTION lbs		670	1,050	57			
LOAD CASE: Lr							
MAX. BENDING MOMENT lb-ft		2,029	3,390	-			
MAX. DEFLECTION in		0.006	0.016	-			
LEFT REACTION lbs		2,082	1,325	-			
RIGHT REACTION lbs		861	1,325	-			
LOAD CASE: L							
MAX. BENDING MOMENT lb-ft		-	-	903			
MAX. DEFLECTION in		-	-	0.049			
LEFT REACTION lbs		-	-	190			
RIGHT REACTION lbs		-	-	190			
LOAD CASE 2: D + Lr					Wood Load Duration Factor = 1.25		
MAX. BENDING MOMENT lb-ft		2,744	4,702	217			
MAX. DEFLECTION in		0.010	0.029	0.015			
LEFT REACTION lbs		2,837	1,900	46			
RIGHT REACTION lbs		1,225	1,900	46			
LOAD CASE 3: D + L					Wood Load Duration Factor = 1.0		
MAX. BENDING MOMENT lb-ft		1,400	2,487	1,173			
MAX. DEFLECTION in		0.004	0.012	0.064			
LEFT REACTION lbs		1,464	1,050	247			
RIGHT REACTION lbs		670	1,050	247			
LOAD CASE 4: D + 0.75 (Lr + L)					Wood Load Duration Factor = 1.25		
MAX. BENDING MOMENT lb-ft		2,338	4,024	758			
MAX. DEFLECTION in		0.008	0.025	0.052			
LEFT REACTION lbs		3,026	2,044	200			
RIGHT REACTION lbs		1,316	2,044	200			
LOAD CASE 5: (1.0 + 0.14S _{DS})D + 0.7ΩE					Wood Load Duration Factor = 1.6		
MAX. BENDING MOMENT lb-ft		1,069	1,899	207			
MAX. DEFLECTION in		0.004	0.013	0.015			
LEFT REACTION lbs		1,491	1,069	58			
RIGHT REACTION lbs		682	1,069	58			

SIMPLY SUPPORTED BEAM DESIGN

		HDR3	HDR4	RB	RE	RE	RE
SPAN	ft	3.50	6.00	19.00			
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD Prllm	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	6x10	6x12	6x12	5.25x14	4x14	4x14
GRADE		No.2	No.2	No.1	Prllm2.2E	No.1	No.1
LOAD CASE 6: $(1.0 + 0.105S_{Ds})D + 0.525\Omega E + 0.75(L + L_r)$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	1,972	3,402	620			
MAX. DEFLECTION	in	0.007	0.022	0.045			
LEFT REACTION	lbs	2,724	1,849	174			
RIGHT REACTION	lbs	1,189	1,849	174			
LOAD CASE 7: $(0.6 - 0.14S_{Ds})D + 0.7\Omega E$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	331	588	64			
MAX. DEFLECTION	in	0.001	0.004	0.005			
LEFT REACTION	lbs	461	331	18			
RIGHT REACTION	lbs	211	331	18			
CHECK BENDING STRESSES							
MAX. BENDING MOMENT	lb-ft	2,743.6	4,701.9	1,173.3			
SECTION MODULUS, S	in ³	82.7	121.2	121.2			
BENDING STRESS	psi	398.0	465.4	116.1			
CHECK BENDING STRESS RATIO		45% OK	53% OK	9% OK			
CHECK SHEAR STRESSES							
MAX. SHEAR FORCE	lbs	3,025.7	2,044.17	247.00			
SECTION AREA, A	in ²	52.25	63.25	63.25			
SHEAR STRESS, fv	psi	86.86	48.48	5.86			
CHECK SHEAR STRESS RATIO		51% OK	29% OK	3% OK			
CHECK DEFLECTION							
MOMENT OF INERTIA, I	in ⁴	392.96	697.07	697.07			
MAX. DEFLECTION, DL	in	0.004	0.012	0.015			
MAX. DEFLECTION, LL	in	0.006	0.016	0.049			
DEFLECTION LIMITS, LL		L / 360	L / 360	L / 360	L / 360	L / 360	L / 360
CHECK DEFLECTION ALL		< 0.12 OK	< 0.2 OK	< 0.63 OK			
MAX. DEFLECTION, DL+LL	in	0.010	0.029	0.064			
DEFLECTION LIMITS, DL+LL		L / 240	L / 240	L / 240	L / 240	L / 240	L / 240
CHECK DEFLECTION Δ(DL+LL)		< 0.18 OK	< 0.3 OK	< 0.95 OK			

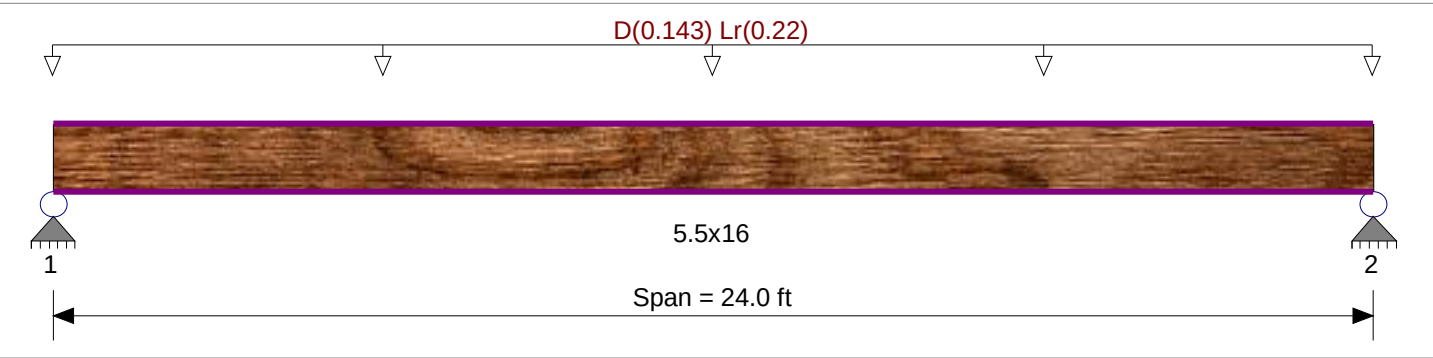
DESCRIPTION: RDG2-Main Building

CODE REFERENCES

Calculations per NDS 2024, IBC 2024
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	<i>E : Modulus of Elasticity</i>	
Load Combination :	ASCE 7-16	Fb -	2,400.0 psi	Ebend- xx	1,800.0ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0ksi
Wood Species :	DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0ksi
Wood Grade :	24F-V8	Fv	265.0 psi	Eminbend - yy	850.0ksi
		Ft	1,100.0 psi	Density	31.210pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.
Beam self weight calculated and added to loading
Uniform Load : D = 0.0130, Lr = 0.020 ksf, Tributary Width = 11.0 ft, (RR)

DESIGN SUMMARY

										Design OK
Maximum Bending Stress Ratio	=	0.493	1	Maximum Shear Stress Ratio	=	0.210	: 1			
Section used for this span		5.5x16		Section used for this span		5.5x16				
fb: Actual	=	1,406.72psi		fv: Actual	=	69.59 psi				
F'b	=	2,856.02psi		F'v	=	331.25 psi				
Load Combination				Load Combination						
			+D+Lr				+D+Lr			
Location of maximum on span	=	12.000ft		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		0.489 in	Ratio = 589 >=360	Span: 1 : Lr Only		n/a				
Max Upward Transient Deflection		0 in	Ratio = 0 <360							
Max Downward Total Deflection		0.849 in	Ratio = 339 >=240	Span: 1 : +D+Lr		n/a				
Max Upward Total Deflection		0 in	Ratio = 0 <240							

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _v	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 24.0 ft	1	0.290	0.124	0.90	1.00	1.00	1.00	0.952	1.00	1.00	1.00	11.67	596.7	2,056.3	1.73	29.5	238.5
+D+Lr														0.0	0.00	0.0	0.0
Length = 24.0 ft	1	0.493	0.210	1.25	1.00	1.00	1.00	0.952	1.00	1.00	1.00	27.51	1,406.7	2,856.0	4.08	69.6	331.3
+D+0.750Lr														0.0	0.00	0.0	0.0
Length = 24.0 ft	1	0.422	0.180	1.25	1.00	1.00	1.00	0.952	1.00	1.00	1.00	23.55	1,204.2	2,856.0	3.50	59.6	331.3
+0.60D														0.0	0.00	0.0	0.0
Length = 24.0 ft	1	0.098	0.042	1.60	1.00	1.00	1.00	0.952	1.00	1.00	1.00	7.00	358.0	3,655.7	1.04	17.7	424.0

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Project Title: 85-25 - E 20th St
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 85-25_Storage AND Main Building.ec6

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

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DESCRIPTION: RDG2-Main Building

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.8490	12.088		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	4.585	4.585
Max Upward from Load Combinations	4.585	4.585
Max Upward from Load Cases	2.640	2.640
D Only	1.945	1.945
+D+Lr	4.585	4.585
+D+0.750Lr	3.925	3.925
+0.60D	1.167	1.167
Lr Only	2.640	2.640

Wood Beam

Project File: 85-25_Storage AND Main Building.ec6

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

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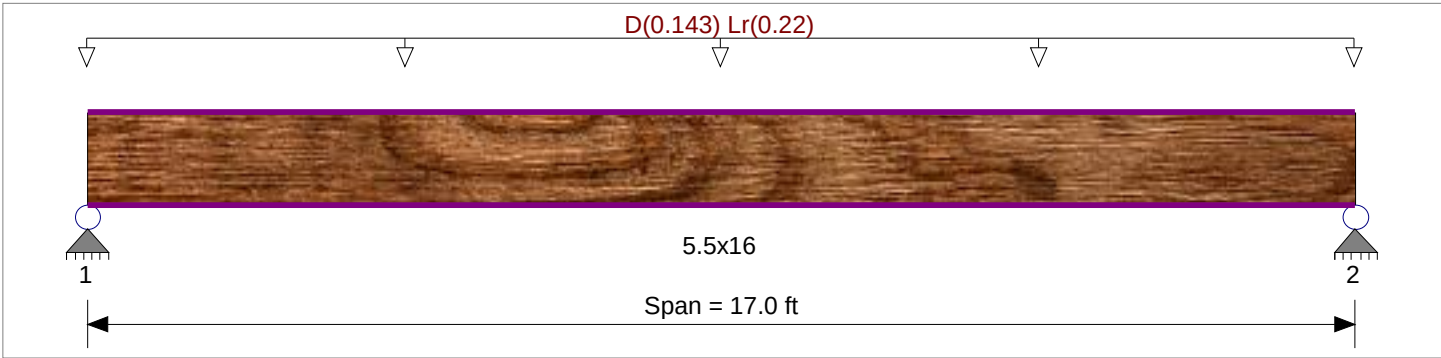
DESCRIPTION: RDG3-Main Building

CODE REFERENCES

Calculations per NDS 2024, IBC 2024
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design	Fb +	2,400.0 psi	E : Modulus of Elasticity	
Load Combination : ASCE 7-16	Fb -	2,400.0 psi	Ebend- xx	1,800.0ksi
	Fc - Prll	1,650.0 psi	Eminbend - xx	950.0ksi
Wood Species : DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0ksi
Wood Grade : 24F-V8	Fv	265.0 psi	Eminbend - yy	850.0ksi
	Ft	1,100.0 psi	Density	31.210pcf
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.0130, Lr = 0.020 ksf, Tributary Width = 11.0 ft, (RR)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.239	1	Maximum Shear Stress Ratio	=	0.141	1
Section used for this span		5.5x16		Section used for this span		5.5x16	
fb: Actual	=	705.80psi		fv: Actual	=	46.87 psi	
F'b	=	2,956.23psi		F'v	=	331.25 psi	
Load Combination				Load Combination			
			+D+Lr				+D+Lr
Location of maximum on span	=	8.500ft		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	0.123 in	Ratio =	1657 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.214 in	Ratio =	954 >=240	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _v	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0			
Length = 17.0 ft	1	0.141	0.083	0.90	1.00	1.00	1.00	0.985	1.00	1.00	1.00	5.85	299.4	2,128.5	1.17	19.9	238.5
+D+Lr														0.0			
Length = 17.0 ft	1	0.239	0.141	1.25	1.00	1.00	1.00	0.985	1.00	1.00	1.00	13.80	705.8	2,956.2	2.75	46.9	331.3
+D+0.750Lr														0.0			
Length = 17.0 ft	1	0.204	0.121	1.25	1.00	1.00	1.00	0.985	1.00	1.00	1.00	11.82	604.2	2,956.2	2.35	40.1	331.3
+0.60D														0.0			
Length = 17.0 ft	1	0.047	0.028	1.60	1.00	1.00	1.00	0.985	1.00	1.00	1.00	3.51	179.6	3,784.0	0.70	11.9	424.0

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Project Title: 85-25 - E 20th St
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 85-25_Storage AND Main Building.ec6

LIC# : KW-06018875, Build:20.25.04.16

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DESCRIPTION: RDG3-Main Building

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.2137	8.562		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	3.248	3.248
Max Upward from Load Combinations	3.248	3.248
Max Upward from Load Cases	1.870	1.870
D Only	1.378	1.378
+D+Lr	3.248	3.248
+D+0.750Lr	2.780	2.780
+0.60D	0.827	0.827
Lr Only	1.870	1.870

Wood Beam

Project File: 85-25_Storage AND Main Building.ecb

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

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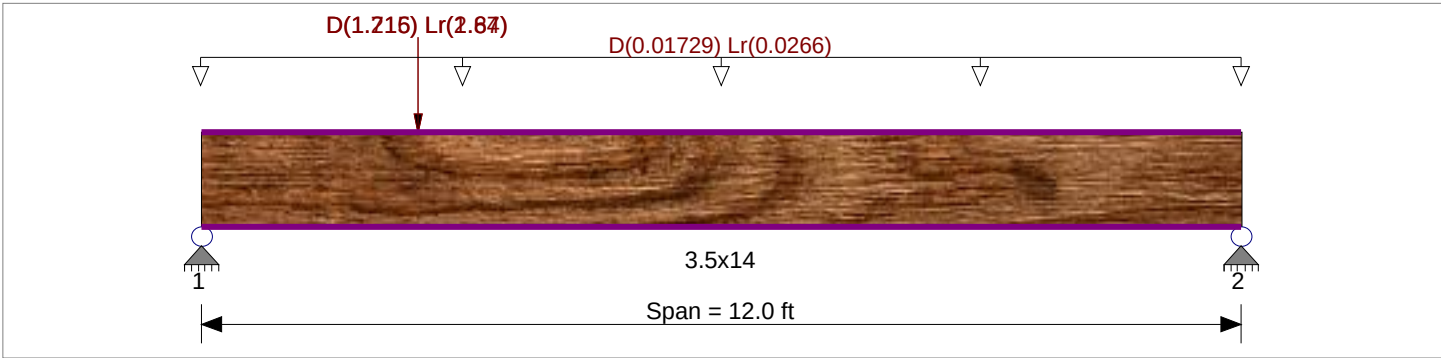
DESCRIPTION: RB1-Main Building

CODE REFERENCES

Calculations per NDS 2024, IBC 2024
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	<i>E : Modulus of Elasticity</i>	
Load Combination	ASCE 7-16	Fb -	2,400.0 psi	Ebend- xx	1,800.0ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0ksi
Wood Species :	DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0ksi
Wood Grade :	24F-V8	Fv	265.0 psi	Eminbend - yy	850.0ksi
		Ft	1,100.0 psi	Density	31.210pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.0130, Lr = 0.020 ksf, Tributary Width = 1.330 ft, (RR)
Point Load : D = 1.716, Lr = 2.640 k @ 2.50 ft, (RDG2)
Point Load : D = 1.215, Lr = 1.870 k @ 2.50 ft, (RDG3)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.537	1	Maximum Shear Stress Ratio	=	0.569	1
Section used for this span		3.5x14		Section used for this span		3.5x14	
fb: Actual	=	1,611.30psi		fv: Actual	=	188.44 psi	
F'b	=	3,000.00psi		F'v	=	331.25 psi	
Load Combination				Load Combination			
Location of maximum on span	=	2.496ft	+D+Lr	Location of maximum on span	=	0.000 ft	+D+Lr
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.126 in	Ratio =	1140 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.212 in	Ratio =	680 >=240	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _v	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0			
Length = 12.0 ft	1	0.298	0.315	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.12	642.7	2,160.0	2.46	75.2	238.5
+D+Lr														0.0			
Length = 12.0 ft	1	0.537	0.569	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	15.35	1,611.3	3,000.0	6.16	188.4	331.3
+D+0.750Lr														0.0			
Length = 12.0 ft	1	0.456	0.483	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	13.04	1,369.1	3,000.0	5.23	160.1	331.3
+0.60D														0.0			
														0.0			

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Wood Beam

Project File: 85-25_Storage AND Main Building.ec6

LIC# : KW-06018875, Build:20.25.04.16

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DESCRIPTION: RB1-Main Building

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _v	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
Length = 12.0 ft	1	0.100	0.106	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.67	385.6	3,840.0	1.47	45.1	424.0

Overall Maximum Deflections

Span	Load Combination	Max. "." Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.2117	5.299		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	6.218	1.877
Max Upward from Load Combinations	6.218	1.877
Max Upward from Load Cases	3.730	1.099
D Only	2.488	0.778
+D+Lr	6.218	1.877
+D+0.750Lr	5.285	1.602
+0.60D	1.493	0.467
Lr Only	3.730	1.099

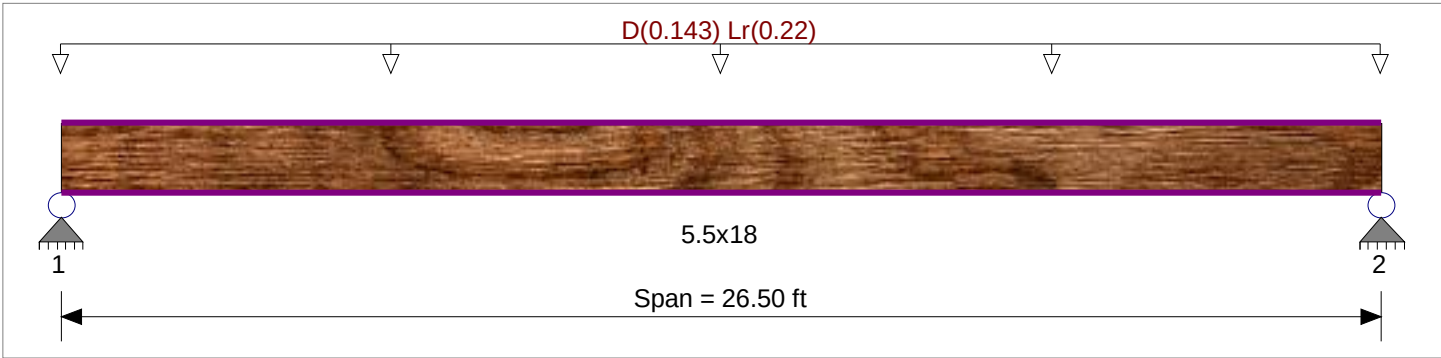
DESCRIPTION: RDG4-Main Building

CODE REFERENCES

Calculations per NDS 2024, IBC 2024
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design	Fb +	2,400.0 psi	E : Modulus of Elasticity	
Load Combination : ASCE 7-16	Fb -	2,400.0 psi	Ebend- xx	1,800.0ksi
	Fc - Prll	1,650.0 psi	Eminbend - xx	950.0ksi
Wood Species : DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0ksi
Wood Grade : 24F-V8	Fv	265.0 psi	Eminbend - yy	850.0ksi
	Ft	1,100.0 psi	Density	31.210pcf
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

Beam self weight calculated and added to loading
Uniform Load : D = 0.0130, Lr = 0.020 ksf, Tributary Width = 11.0 ft, (RR)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.488	1	Maximum Shear Stress Ratio	=	0.207	: 1
Section used for this span		5.5x18		Section used for this span		5.5x18	
fb: Actual	=	1,363.56psi		fv: Actual	=	68.73 psi	
F'b	=	2,794.75psi		F'v	=	331.25 psi	
Load Combination				Load Combination			
			+D+Lr				+D+Lr
Location of maximum on span	=	13.250ft		Location of maximum on span	=	25.049 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.510 in	Ratio =	623 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.892 in	Ratio =	356 >=240	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _v	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0			
Length = 26.50 ft	1	0.290	0.123	0.90	1.00	1.00	1.00	0.932	1.00	1.00	1.00	14.44	583.3	2,012.2	1.94	29.4	238.5
+D+Lr														0.0			
Length = 26.50 ft	1	0.488	0.207	1.25	1.00	1.00	1.00	0.932	1.00	1.00	1.00	33.75	1,363.6	2,794.7	4.54	68.7	331.3
+D+0.750Lr														0.0			
Length = 26.50 ft	1	0.418	0.178	1.25	1.00	1.00	1.00	0.932	1.00	1.00	1.00	28.92	1,168.5	2,794.7	3.89	58.9	331.3
+0.60D														0.0			
Length = 26.50 ft	1	0.098	0.042	1.60	1.00	1.00	1.00	0.932	1.00	1.00	1.00	8.66	350.0	3,577.3	1.16	17.6	424.0

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 www.fastructural.com

Project Title: 85-25 - E 20th St
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 85-25_Storage AND Main Building.ec6

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: RDG4-Main Building

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.8918	13.347		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	5.094	5.094
Max Upward from Load Combinations	5.094	5.094
Max Upward from Load Cases	2.915	2.915
D Only	2.179	2.179
+D+Lr	5.094	5.094
+D+0.750Lr	4.365	4.365
+0.60D	1.307	1.307
Lr Only	2.915	2.915

Wood Beam

Project File: 85-25_Storage AND Main Building.ec6

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

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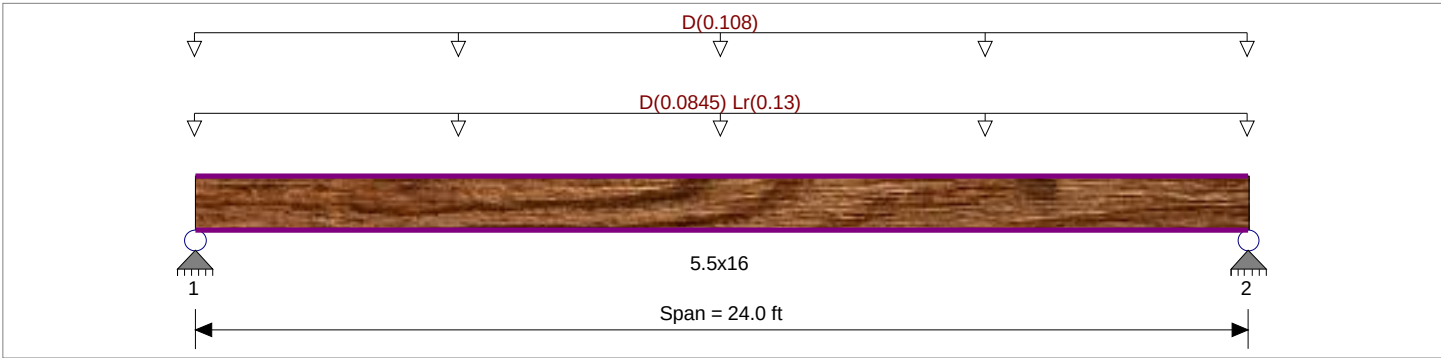
DESCRIPTION: RB7-Main Building

CODE REFERENCES

Calculations per NDS 2024, IBC 2024
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design	Fb +	2,400.0 psi	E : Modulus of Elasticity	
Load Combination : ASCE 7-16	Fb -	2,400.0 psi	Ebend- xx	1,800.0ksi
	Fc - Prll	1,650.0 psi	Eminbend - xx	950.0ksi
Wood Species : DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0ksi
Wood Grade : 24F-V8	Fv	265.0 psi	Eminbend - yy	850.0ksi
	Ft	1,100.0 psi	Density	31.210pcf
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.0130, Lr = 0.020 ksf, Tributary Width = 6.50 ft, (RR)
Uniform Load : D = 0.0180 ksf, Tributary Width = 6.0 ft, (WALL)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.440	1	Maximum Shear Stress Ratio	=	0.188	1
Section used for this span		5.5x16		Section used for this span		5.5x16	
fb: Actual	=	1,257.61psi		fv: Actual	=	62.22 psi	
F'b	=	2,856.02psi		F'v	=	331.25 psi	
Load Combination				Load Combination			
			+D+Lr				+D+Lr
Location of maximum on span	=	12.000ft		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	0.289 in	Ratio =	997 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.759 in	Ratio =	379 >=240	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _v	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0			
Length = 24.0 ft	1	0.379	0.162	0.90	1.00	1.00	1.00	0.952	1.00	1.00	1.00	15.23	779.0	2,056.3	2.26	38.5	238.5
+D+Lr														0.0			
Length = 24.0 ft	1	0.440	0.188	1.25	1.00	1.00	1.00	0.952	1.00	1.00	1.00	24.59	1,257.6	2,856.0	3.65	62.2	331.3
+D+0.750Lr														0.0			
Length = 24.0 ft	1	0.398	0.170	1.25	1.00	1.00	1.00	0.952	1.00	1.00	1.00	22.25	1,137.9	2,856.0	3.30	56.3	331.3
+0.60D														0.0			
Length = 24.0 ft	1	0.128	0.055	1.60	1.00	1.00	1.00	0.952	1.00	1.00	1.00	9.14	467.4	3,655.7	1.36	23.1	424.0

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Project Title: 85-25 - E 20th St
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 85-25_Storage AND Main Building.ec6

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: RB7-Main Building

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.7590	12.088		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	4.099	4.099
Max Upward from Load Combinations	4.099	4.099
Max Upward from Load Cases	2.539	2.539
D Only	2.539	2.539
+D+Lr	4.099	4.099
+D+0.750Lr	3.709	3.709
+0.60D	1.523	1.523
Lr Only	1.560	1.560

COLUMN AXIAL CAPACITY

WOOD PROPERTIES:

Use Douglas Fir-Larch	$C_F =$	1.1	$K_{ce} =$	0.30	Buckling about x-axis?	YES
Species: Stud	$C_m =$	1.0	$c =$	0.80	Check le/d reqmt.	OK!
$E = 1,400,000$ psi	$C_T =$	1.0	$K_f =$	1.00		
$F'_c = 850$ psi	$C_t =$	1.0	(for bolted or nailed members)			
$E' = 1,400,000$ psi	$C_i =$	1.0				
$F_{c*} = 935$ psi	$C_D =$	1.0				

COL	No. of Plies	Size	d_y	d_x	$le_{x,y}$	le/dy	le/dx	(le/d)	Fce	Fce/Fc*	$\frac{1+ Fce/Fc*}{2c}$	C_p	F'c	Column Capacity
C01	1 -	2x 4	1.5	3.50	10	80.0	34.3	34.3	357.29	0.382	0.86	0.346	323.16	1.70 kips
C02	2 -	2x 4	3.0	3.50	10	40.0	34.3	34.3	357.29	0.382	0.86	0.346	323.16	3.39 kips
C03	1 -	2x 6	1.5	5.50	10	80.0	21.8	21.8	882.29	0.944	1.21	0.671	627.00	5.17 kips
C04	2 -	2x 6	3.0	5.50	10	40.0	21.8	21.8	882.29	0.944	1.21	0.671	627.00	10.35 kips
C05	1 -	2x 8	1.5	7.50	10	80.0	16.0	16.0	1,640.63	1.755	1.72	0.844	788.87	8.87 kips
C06	2 -	2x 8	3.0	7.25	10	40.0	16.6	16.6	1,533.07	1.640	1.65	0.830	775.95	16.88 kips

WOOD PROPERTIES:

Use Douglas Fir-Larch	$C_F =$	1.1	$K_{ce} =$	0.30	Buckling about x-axis?	YES
Species: No.1	$C_m =$	1.0	$c =$	0.80	Check le/d reqmt.	OK!
$E = 1,700,000$ psi	$C_T =$	1.0	$K_f =$	1.00		
$F'_c = 1,500$ psi	$C_t =$	1.0	(for bolted or nailed members)			
$E' = 1,700,000$ psi	$C_i =$	1.0				
$F_{c*} = 1,650$ psi	$C_D =$	1.0				

COL	No. of Plies	Size	d_y	d_x	$le_{x,y}$	le/dy	le/dx	(le/d)	Fce	Fce/Fc*	$\frac{1+ Fce/Fc*}{2c}$	C_p	F'c	Column Capacity
C07	1 -	4x 4	3.5	3.50	10	34.3	34.3	34.3	357.29	0.382	0.86	0.346	323.16	3.96 kips
C08	1 -	4x 6	3.5	5.50	10	34.3	21.8	21.8	882.29	0.944	1.21	0.671	627.00	12.07 kips
C09	1 -	4x 8	3.5	7.50	10	34.3	16.0	16.0	1,640.63	1.755	1.72	0.844	788.87	20.71 kips
C10	1 -	6x 6	5.5	5.50	10	21.8	21.8	21.8	882.29	0.944	1.21	0.671	627.00	18.97 kips
C11	1 -	6x 8	5.5	7.25	10	21.8	16.6	16.6	1,533.07	1.640	1.65	0.830	775.95	30.94 kips

Note:

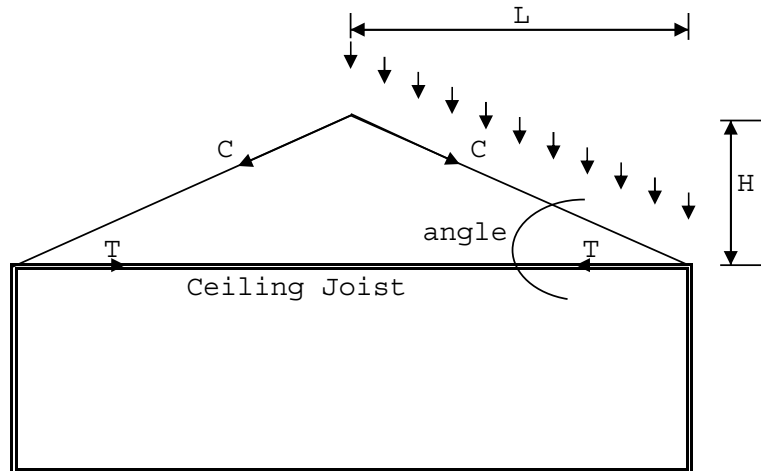
$K_{c_E} = 0.510 - 0.839$ (COV)	where COV =	0.25	- sawn lumber
0.3	- for visually graded lumber	0.15	- MEL
0.384	- for machine evaluated lumber (MEL)	0.11	- MSR (machine stress rated)
0.418	- for products w/ $COV \leq 0.11$	0.10	- glu-lam

FOUNDATION DESIGN

ALLOW BEARING CAPACITY : 1,500.00 psf
 CONCRETE STRENGTH, f'_c : 2,500.00 psi
 YEILD STRENGTH, F_y : 60,000.00 psi

			EXT. WALL FTG	INT. WALL FTG	F1	F1	F2
FOOTING DIMENSION							
WIDTH	in		18.0	18.0	30.0	42.0	60.0
LENGTH	in		12.0	12.0	30.0	42.0	60.0
DEPTH	in		24.0	24.0	24.0	24.0	24.0
COLUMN DIMENSION							
WIDTH	in		4.0	4.0	4.0	4.0	4.0
LENGTH	in		12.0	12.0	4.0	6.0	6.0
FOOTING LOAD							
ROOF TRIB. AREA	ft2		18.0	20.0			
CEILING TRIB. AREA	ft2		18.0	20.0			
FLOOR TRIB. AREA	ft2		-	-			
DECK TRIB. AREA	ft2		-	-			
WALL AREA	ft2		20.0	20.0			
WALL TYPE			EXT	INT			
DEAD LOAD	D	lbs	702.0	580.1	1,500.0	3,500.0	7,500.0
LIVE LOAD	L	lbs	720.0	800.0	1,800.0	5,500.0	9,500.0
SEISMIC LOAD	E	lbs	-	-			
CHECK BEARING CAPACITY							
SERVICE LOAD, (D+L)	lbs		1,422.0	1,380.1	3,300.0	9,000.0	17,000.0
SERVICE LOAD, (D+L+E)/1.33	lbs		1,069.2	1,037.6	2,481.2	6,766.9	12,782.0
SOIL PRESSURE, $f_b = P/A_{FTG}$	psf		948.0	920.0	528.0	734.7	680.0
CHECK $f_b < F_b$			OK	OK	OK	OK	OK
CHECK MINIMUM REINFORCEMENT							
REINFORCEMENT USE			2#4	2#4	2#4	3#4	4#4
AREA OF STEEL, A_s	in2		0.40	0.40	0.40	0.60	0.80
$A_{s1 \text{ min}} = 0.0018 * \text{Area_Conc.}$	in2		0.68	0.68	1.13	1.59	2.27
$A_{s2 \text{ min}} = 1.33 * A_{s \text{ Required}}$	in2		0.00	0.00	0.02	0.07	0.21
$A_s \text{ min} = \text{Min}(A_{s1}, A_{s2})$	in2		0.00	0.00	0.02	0.07	0.21
CHECK $A_s > A_s \text{ MIN}$			OK	OK	OK	OK	OK
CHECK BENDING STRESSES							
FACTORED, $1.4D+1.7L$	lbs		2,207	2,172	5,160	14,250	26,650
FACTORED, $1.4(D+L+E)/1.33$	lbs		1,991	1,932	4,620	12,600	23,800
MAXIMUM MOMENT, M_u	lb-in		3,004	2,956	14,534	61,241	174,113
b	in		12.00	12.00	30.00	42.00	60.00
$a = A_s * F_y / 0.85 * f'_c * b$	in		0.94	0.94	0.38	0.40	0.38
$\phi M_n = \phi * A_s * F_y * (d - a/2)$	lb-in		443,435	443,435	449,534	673,866	899,068
CHECK $M_u < \phi M_n$			OK	OK	OK	OK	OK
CHECK SHEAR STRESSES							
MAXIMUM SHEAR, V_u	lbs		-	-	1,548	7,125	17,323
b	in		12.00	12.00	30.00	42.00	60.00
$\phi V_c = 2 * \phi * b * w * d * \text{sqrt}(f'_c)$	lbs		21,420	21,420	53,550	74,970	107,100
CHECK $V_u < \phi V_c$			OK	OK	OK	OK	OK

Nailing & Bolting Schedule for eaves tie connections



$W = L(\text{ft}) \times \text{rafter spacing (ft)} \times \text{roof design load (L.L. + D.L. in psf)}$

$$T = \frac{W \times L}{2H}$$

Given:

Roof Rafter

Roof Rafter Trib. Width	L	=	9.5	ft
Roof Dead Load	DL	=	13.003	psf
Roof Live Load	LL	=	20	psf
Roof Rafter Spacing	s	=	1.33	ft
Roof Pitch			5:12	

Calculated:

Tan a=	0.42
Angle a	22.62
H=	$L \times \tan a$
	3.96
W=	417
T=	500

Design criteria:

Nail Capcit: #/nail	Cd	#/nail
8d	98	1.25
16d	103	1.25
1/2" MB		400

Number of nails required: $T/\text{nail capacity}$

8d=	4 nails @ each connection
16d=	4 nails @ each connection
1/2" bolts=	1 bolts @ each connection

**E 20th St,
Upland, CA 91784**

STRUCTURAL CALCULATIONS
Maintenance Building

Date : 09/12/25

Rev. : 0

CALCULATION INDEX

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GRAVITY LOADS

ROOF LOADS

DEAD LOAD		
Roofing Shingles Asphalt (1/4 in)	2.00	psf
Sheathing 15/32" Plywood	2.81	psf
Framing 2x12 @ 16" O.C. DF-Larch	3.15	psf
Drywall Finish	3.00	psf
Insulation Fiber Glass Loose	0.56	psf
Miscellaneous	1.48	psf
ROOF DEAD LOAD TOTAL	13.00	psf
ROOF LIVE LOAD TOTAL	20.00	psf

FLOOR LOADS

DEAD LOAD		
Flooring Hardwood (1 in)	3.00	psf
Sheathing 3/4" T&G	2.25	psf
Framing 2x12 @ 16" O.C. DF-Larch	3.15	psf
Drywall Finish	3.00	psf
Miscellaneous	1.60	psf
FLOOR DEAD LOAD TOTAL	13.00	psf
FLOOR LIVE LOAD TOTAL	40.00	psf

OPEN DECK

DEAD LOAD		
Flooring Hardwood (1 in)	3.00	psf
Sheathing 3/4" T&G	2.25	psf
Framing 2x12 @ 16" O.C. DF-Larch	3.15	psf
Drywall Finish	3.00	psf
Miscellaneous	1.60	psf
DECK DEAD LOAD TOTAL	13.00	psf
DECK LIVE LOAD TOTAL	100.00	psf

CEILING LOADS

DEAD LOAD		
Framing 2x 8 @ 16" O.C. DF-Larch	2.10	psf
Drywall Finish	3.00	psf
Miscellaneous	1.00	psf
CEILING DEAD LOAD TOTAL	6.00	psf
CEILING LIVE LOAD TOTAL	20.00	psf

EXTERIOR WALL

Wall Plaster (1 in) on Metal Lath	10.00	psf
Sheathing 15/32" Plywood	2.81	psf
Wall Wood Studs 2x6 @ 16" O.C. DF-Larch	1.58	psf
Insulation Fiber Glass Loose	0.50	psf
Drywall Finish	3.00	psf
EXTERIOR WALL LOAD	18.00	psf

INTERIOR WALL

Drywall Finish	3.00	psf
Wall Wood Studs 2x4 @ 16" O.C. DF-Larch	1.05	psf
Sheathing 15/32" Plywood	2.81	psf
Drywall Finish	3.00	psf
INTERIOR WALL LOAD	10.00	psf

BUILDING WEIGHT

DESIGN CRITERIA

Code Used : 2022 CBC
Building Code : 2021 IBC
LACity Requirement : Yes

Concrete F'c : 2,500 psi
Steel Fy : 60,000 psi

SOILS REPORT

NO SOILS REPORT

Bearing Capacity : 1,500 psf

SEISMIC DESIGN PARAMETERS

Short Period, Fa : 1.2 0.2 sec Fig 1613.5(1)
Long Period, Fv : 1.5 1.0 sec Fig 1613.5(2)
Response Spectral : 198.00% 0.2 sec Fig 1613.5(3)
Response Spectral : 67.00% 1.0 sec Fig 1613.5(4)

Latitude : 34.135827
Longitude : -117.646592
Site Class : D

BUILDING SUMMARY

Level	Height ft	Elevation	Unit Weight psf	
			Dead	Live
ROOF	8.50	8.50	19.00	20.00

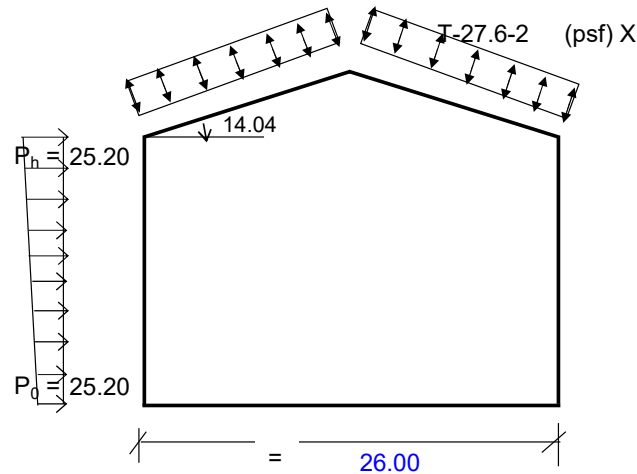
Wall Type	Weight psf
Exterior	18.00
Interior	10.00

BUILDING WEIGHT

ROOF

Area Type	L (ft)	W (ft)	H (ft)	Area (ft^2)	Dead Load (psf)	Weight (kips)
Roof	158.00	26.00		4,703.9	13.00	61.16
Ext. Wall	316.00	52.00	4.25		18.00	28.15
Int. Wall	10.00	78.00	4.25		10.00	3.74
TOTAL ROOF AREA / LOAD				4,703.9		93.05

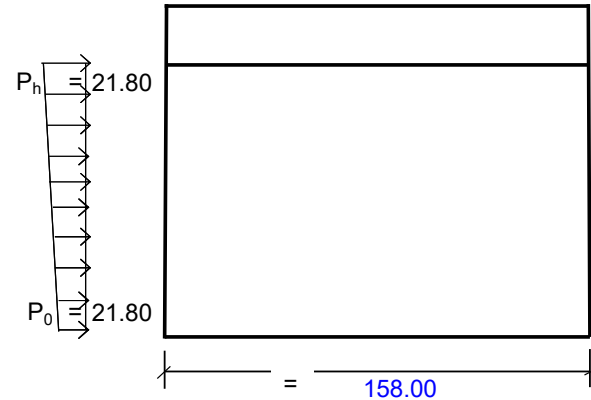
27.5 Directional Procedure, Part 2: Enclosed Simple Diaphragm Buildings With $H \leq 160$ Ft



Example will work for V = 110, 115 and 120 MPH only.

Wind Normal to Ridge

Roof Exp adjust factor T-27.6-2 = 1



Wind Parallel to Ridge

Roof Height $h = 15$ feet
 Exposure **C**
 Roof Pitch = **3.00 :12**
 Topography factor $K_{zt} = 1.00$
 Building & Structure Risk Category = **II, standard**
 Wind Speed $V = 110$ mph
 Wind Normal to Ridge, $L/B = 0.165$ use 0.5
 Since $.2 \leq L/B \leq 5.0 = \text{No}$
 and $h \leq 60 = \text{Yes}$
 Building meet Class I condition ? No
 Since $.5 \leq L/B \leq 2.0 = \text{No}$
 and $60 \leq h \leq 160 = \text{No}$
 Fundamental period = **0.500 sec**
 More than $h/75$ sec. NG
 Building meet Class II condition ? No
Simplified method cannot be use
 From T-27.6-1, $V = 110$ MPH Exp C, $L/B = 0.5$ and 15 FT. HT
 $P_h = 25.20 K_{zt}$
 $P_o = 25.20 K_{zt}$
 T-27.6-2 $V = 110$ Exp C, $h = 15$ Roof slope = 3:12
 Zone 1 = **-23.5**
 Adjust factor T-27.6-2, = 1.00
 Design roof pressure for zone 1 = **-23.50 PSF**

Wind Base Shear = **35.83 kips**

26.8.2, Figure 26.8-1

IBC T-1604.5

Fig. 26.5-1A, MRI = 700 yrs

Wind Parallel to Ridge, $L/B = 6.08$ use 2.0
 Since $.2 \leq L/B \leq 5.0 = \text{No}$
 and $h \leq 60 = \text{Yes}$
 Building meet Class I condition ? No
 Since $.5 \leq L/B \leq 2.0 = \text{No}$
 and $60 \leq h \leq 160 = \text{No}$
 Fundamental period = **0.500 sec**
 More than $h/75$ sec. NG
 Building meet Class II condition ? No
Simplified method cannot be use
 From T-27.6-1, $V = 110$ MPH Exp C, $L/B = 2.0$ and 15 FT. HT
 $P_h = 21.80 K_{zt}$
 $P_o = 21.80 K_{zt}$

Wind Base Shear = **5.10 kips**

ASCE Hazards Report

Standard: ASCE/SEI 7-22

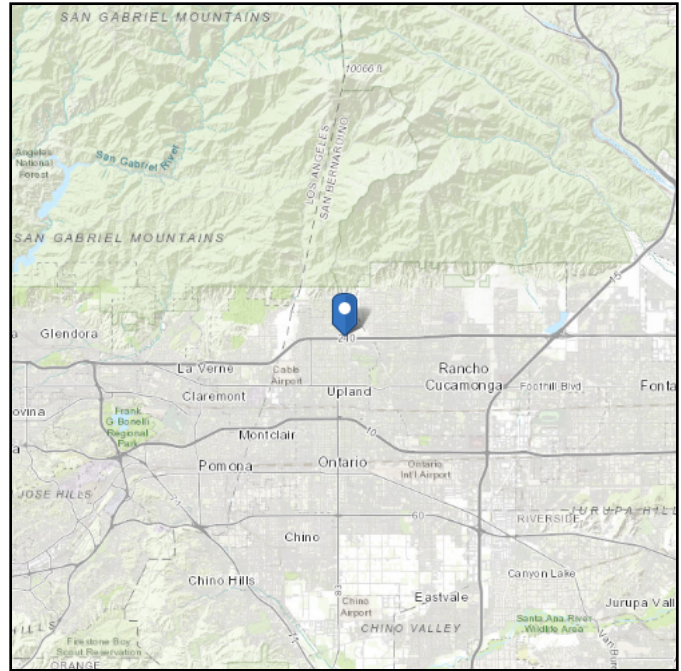
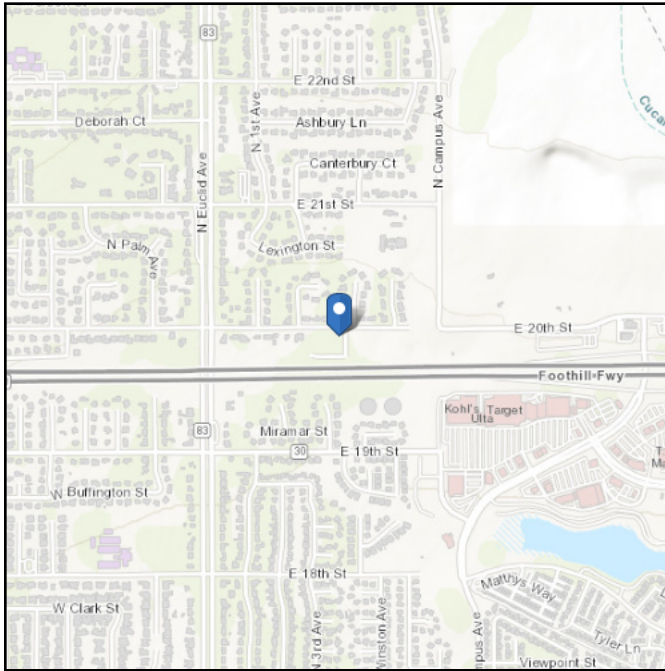
Risk Category: II

Soil Class: Default

Latitude: 34.135827

Longitude: -117.646592

Elevation: 1648.2100926574194 ft
(NAVD 88)



Wind

Results:

Wind Speed	95 Vmph
10-year MRI	66 Vmph
25-year MRI	72 Vmph
50-year MRI	77 Vmph
100-year MRI	81 Vmph
300-year MRI	89 Vmph
700-year MRI	95 Vmph
1,700-year MRI	102 Vmph
3,000-year MRI	106 Vmph
10,000-year MRI	115 Vmph
100,000-year MRI	133 Vmph
1,000,000-year MRI	151 Vmph

Data Source: ASCE/SEI 7-22, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2

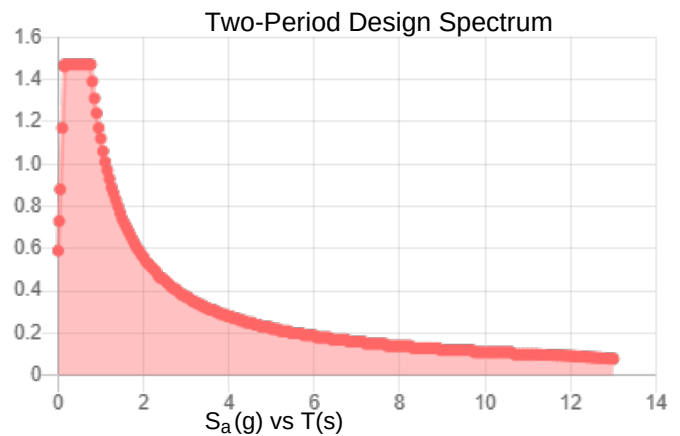
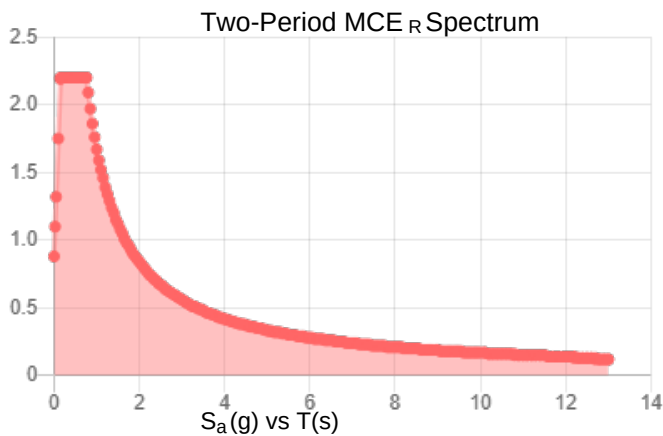
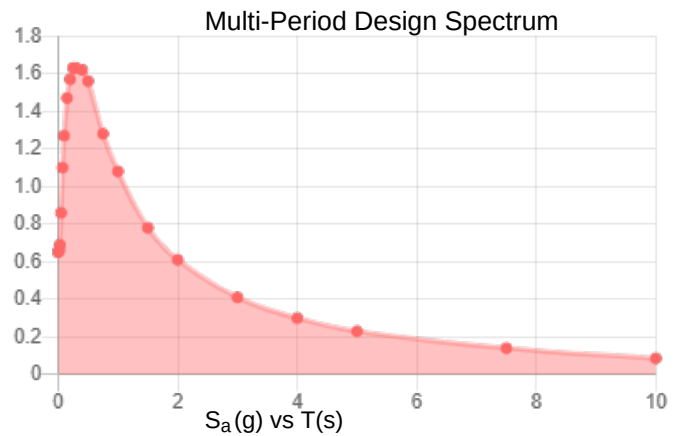
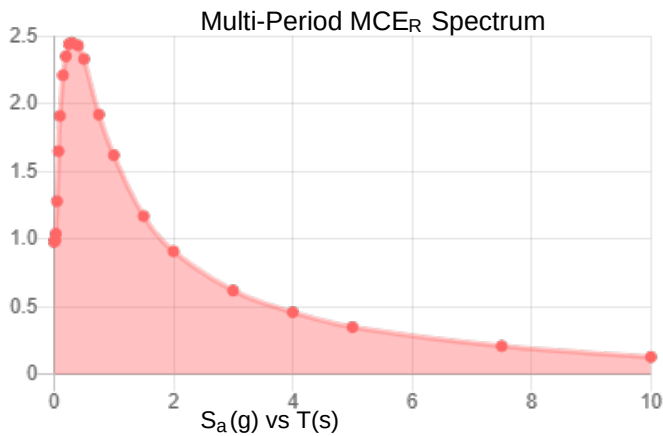
Date Accessed: Fri Aug 15 2025

Site Soil Class: Default

Results:

PGA _M :	0.85	T _L :	12
S _{MS} :	2.2	S _S :	1.98
S _{M1} :	1.67	S ₁ :	0.67
S _{DS} :	1.47	V _{S30} :	260
S _{D1} :	1.12		

Seismic Design Category: D



MCE_R Vertical Response Spectrum
Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum
Vertical ground motion data has not yet been made available by USGS.

IBC2021 (1613), ASCE 7-16 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

Response Spectral Acc. (0.2 sec) $S_s = 1.980\text{ g}$

Figure 22-1,22-3,22-5,22-6

Response Spectral Acc. (1.0 sec) $S_1 = 0.670\text{ g}$

Figure 22-2,22-4,22-5,22-6

Soil Site Class

Table 20-3-1, Default = D

Site Coefficient $F_a = 1.200$

Table 11.4-1

Site Coefficient $F_v = 1.500$

Table 11.4-2

Max Considered Earthquake Acc. $S_{MS} = F_a \cdot S_s = 2.376\text{ g}$ (11.4-1)

Max Considered Earthquake Acc. $S_{M1} = F_v \cdot S_1 = 1.005\text{ g}$ (11.4-2)

@ 5% Damped Design $S_{DS} = 2/3(S_{MS}) = 1.584\text{ g}$ (11.4-3)

$S_{D1} = 2/3(S_{M1}) = 0.670\text{ g}$ (11.4-4)

Building Risk Categories

Table 1604.5

Design Category Consideration:

with dist. between seismic resisting system >40ft

Seismic Design Category for 0.1sec D

Table 11.6-1

Seismic Design Category for 1.0sec D

Table 11.6-2

$S_1 < .75\text{ g}$ NA

Section 11.6

Since $T_a < .8T_s$ (see below), SDC = D

Control (exception of Section 11.6 does not apply)

IBC, Seismic Design Category D

IRC, Seismic Design Category = E

T-R301.2.2.1.1

12.8 Equivalent lateral force procedure

A. BEARING WALL SYSTEMS

T-12.2

Seismic Force Resisting Systems

15. Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance or steel

$C_t = 0.02$

$\alpha = 0.75$

T-12.8-2

Building ht. $H_n = 35$

ft

Unfinished Building Height (ft) = 65

$C_u = 1.400$

for S_{D1} of 0.670g

Table 12.8-1

Approx Fundamental period, $T_a = C_t(h_n)^x = 0.288$

12.8-7

$T_L = 6.000\text{ Sec}$

Calculated T shall not exceed $\leq C_u \cdot T_a = 0.403$

Use T = sec.

$0.8T_s = 0.8(S_{D1}/S_{DS}) = 0.338 < T_a$

Is structure Regular & ≤ 5 stories?

12.8.1.3

Response Spectral Acc. (0.2 sec) $S_s = 1.980\text{ g}$

$F_a = 1.20$

@ 5% Damped Design $S_{DS} = 2/3(F_a \cdot S_s) = 1.584\text{ g}$ (11.4-3)

Response Modification Coef. $R = 6.5$

Table-12.2-1

Over Strength Factor $\Omega_o = 2.5$

foot note g

Importance factor $I = 1$

T 1.5-2

Seismic Base Shear $V = C_s W$

$$C_s = \frac{S_{DS}}{R/I} = 0.244 \quad (12.8-2)$$

$$\text{or need not to exceed, } C_s = \frac{S_{D1}}{(R/I) \cdot T} = 0.256 \quad \text{For } T \leq T_L \quad (12.8-3)$$

$$\text{or } C_s = \frac{S_{D1} T_L}{T^2 (R/I)} \quad \text{N/A} \quad \text{For } T > T_L \quad (12.8-4)$$

C_s shall not be less than $= 0.044 S_{DS} \cdot I \geq 0.1 = 0.070$ (12.8-5)

Min $C_s = 0.5 S_1 / R = 0.052$ For $S_1 \geq 0.6\text{ g}$ (12.8-6)

Use $C_s = 0.244$

Design base shear $V = 0.244 W$ Control

12.14 Simplified Seismic base shear

15. Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance or steel

T-12.14

@ 5% Damped Design $S_{DS} = 1.584$

SDC = D

Limitations: P

$F = 1.2$

For three story building

$R = 6.5$

$$V = \frac{F S_{DS} (W)}{R} = 0.292 W$$

(12.14-11)

LATERAL FORCE DISTRIBUTION

SEISMIC BASE SHEAR

Importance Factor, I ASCE7-16 Table 11.5-1 I = 1.00
EQ Acc. @ 5% Damped Design ASCE7-16 EQ 11.4-3 SDS = 1.58
Vult = 2021 IBC SHEET = 0.244 W
Vwork = Vult / 1.4 = 0.174 W Working Stress Level
Vwork = 0.174 * 93.1 kips = 16.2 kips Seismic Base Shear
p = 1.30 ==> Vwork = 21.1 kips Seismic Base Shear

WIND BASE SHEAR

Vult = MWFR < 160 ft = 35.8 kips
Vwork = Vult * 0.6 = 21.5 kips

WIND BASE SHEAR GOVERNS

DESIGN BASE SHEAR	=	21.5 kips
-------------------	---	-----------

BASE SHEAR VERTICAL DISTRIBUTION

Floor Type	Height hx ft	Floor Area ft ²	Weight wx kips	wx*hx kips-ft	Fx % wx*hx Σ (wx*hx)	Fx (V-Ft) Fx% kips	Story V Ft+ΣFi kips	Fx Fl Area psf
ROOF	8.50	4,704	93.05	791	100.0%	21.50	21.50	4.57
		Total	93.05	791	100.0%	21.50		

DIAPHRAGM FORCE DISTRIBUTION

Floor Type	Weight		Story V Ft+ΣFi kips	Fpx V*wp/Σw kips	Min Fpx ¹ kips	Max Fpx ² kips	Fpx Design kips
	wp kips	Σwi kips					
ROOF	78.98	78.98	21.50	21.50	25.02	50.04	25.02

1 Fpx min = 0.2*SDs*I*wp (for Flexible Diaphragm)

ASCE7-16 12.10.1.1

2 Fpx max = 0.4*SDs*I*wp (for Flexible Diaphragm)

ASCE7-16 12.10.1.1

SHEARWALL DESIGN FORCES

Level	Fl. Area	Wall Ht.	Stry Shear kips	Base ² Shear %	Shear psf
	ft ²	ft			
ROOF	4,704	8.50	21.50	100.0%	4.57

1 Redundancy Factor Used Per ASCE7-16 12.3.4.2

TABLES REFERENCED IN SHEARWALL DESIGN SPREADSHEET

Table 2306.4.1 Allowable Shear (plf) for Wood Shearwall Panels For Wind or Seismic Loading

Mark	# sides	Sheathing	Edge Nailing	A35/LTP 4 Simpson	Sill Plate @ Framed Floor	Anchor @ Foundation	V _{allow} (100%) (plf)	V _{allow} (75%) (plf)
1	1	15/32"	10d @6"	@16"	3/8" dia Lag Screw @32"OC	5/8" dia A.B. @48"OC	340	255
2	1	15/32"	10d @4"	@8"	3/8" dia Lag Screw @16"OC	5/8" dia A.B. @32"OC	510	380
3	1	15/32"	10d @3"	@12"	3/8" dia Lag Screw @12"OC	5/8" dia A.B. @24"OC	665	500
4	1	15/32"	10d @2"	@8"	3/8" dia Lag Screw @9"OC	5/8" dia A.B. @16"OC	870	650
5	2	15/32"	10d @4"	@8"	3/8" dia Lag Screw @9"OC	5/8" dia A.B. @16"OC	1,020	765
6	2	15/32"	10d @3"	@4"	3/8" dia Lag Screw @9"OC	5/8" dia A.B. @12"OC	1,370	1,030
7	2	15/32"	10d @2"	@4"	3/8" dia Lag Screw @4"OC	5/8" dia A.B. @16"OC	1,740	1,305

NOTES:

1. ALL SHEATHING TO BE DOUGLAS FIR-LARCH STRUCT. I.
2. ALL FIELD NAILING TO INTERMEDIATE FRAMING TO BE AT 12" O.C.
3. FRAMING AT ADJOINING PANEL EDGES SHALL BE 3" (INCH) NOMINAL OR THICKER (i.e. MIN 3x VERTICAL STUDS AND BLOCKING) NAILS SHALL BE STAGGERED IN TWO LINES ALONG PANEL EDGES WHERE NAILS ARE SPACED 2" INCHES ON CENTER OR WHEN 10d COMMON NAILS SPACED 3" O.C. PENETRATE FRAMING MORE THAN 1 5/8".
4. WHERE PLYWOOD APPLIES ON BOTH FACES OF A WALL AND NAIL SPACING IS LESS THAN 6" INCHES ON CENTER ON EITHER SIDE. PANEL JOINTS SHALL HAVE MIN 3x (VERTICAL STUDS & BLOCKING) AND NAILS SHALL BE STAGGERED. OFFSET PANEL JOINTS TO FALL ON DIFFERENT FRAMING MEMBER WHERE PLYWOOD IS PLACED AT BOTH FACES.
5. NAILS SHALL BE PLACED NOT LESS THAN 1/2" INCH FROM PANEL EDGES.
6. FOR HOLD-DOWNS AT THE END OF THE SHEAR WALL, SEE PLANS AND SCHEDULE. PROVIDE 3"x3"x1/4" WASHER PLATE AT ALL SILL PLATE ANCHOR BOLTS AND AT ALL HOLD-DOWN BOLTS CONNECTED TO VERTICAL POSTS.
7. USE WELDED STUD BOLTS OF SAME SIZE & SPACING AS OF SILL PLATE ANCHOR BOLTS, WHERE STEEL BEAM OCCURS UNDER SHEAR WALL.
8. PROVIDE DOUBLE PARALLAM BLOCKING TO ACCOMODATE 2 ROWS OF SIMPSON SDS SCREWS.
9. FOUNDATION SILL PLATE SHALL BE 3x. U.N.O.
10. SIMPSON SDS 3/8"x7" WOOD SCREWS. TYP. USE |2"x4|4" SDS INTO NAILER AT STEEL BEAMS.
11. WHERE 2-2x SILL PLATES ARE USED, SCREW LOWER PLATE TO FRAMING BELOW. DRILL HOLE IN UPPER SILL PLATE FOR SCREW HEAD SO THAT UPPER PLATE SITS FLAT ON LOW SILL PLATE.

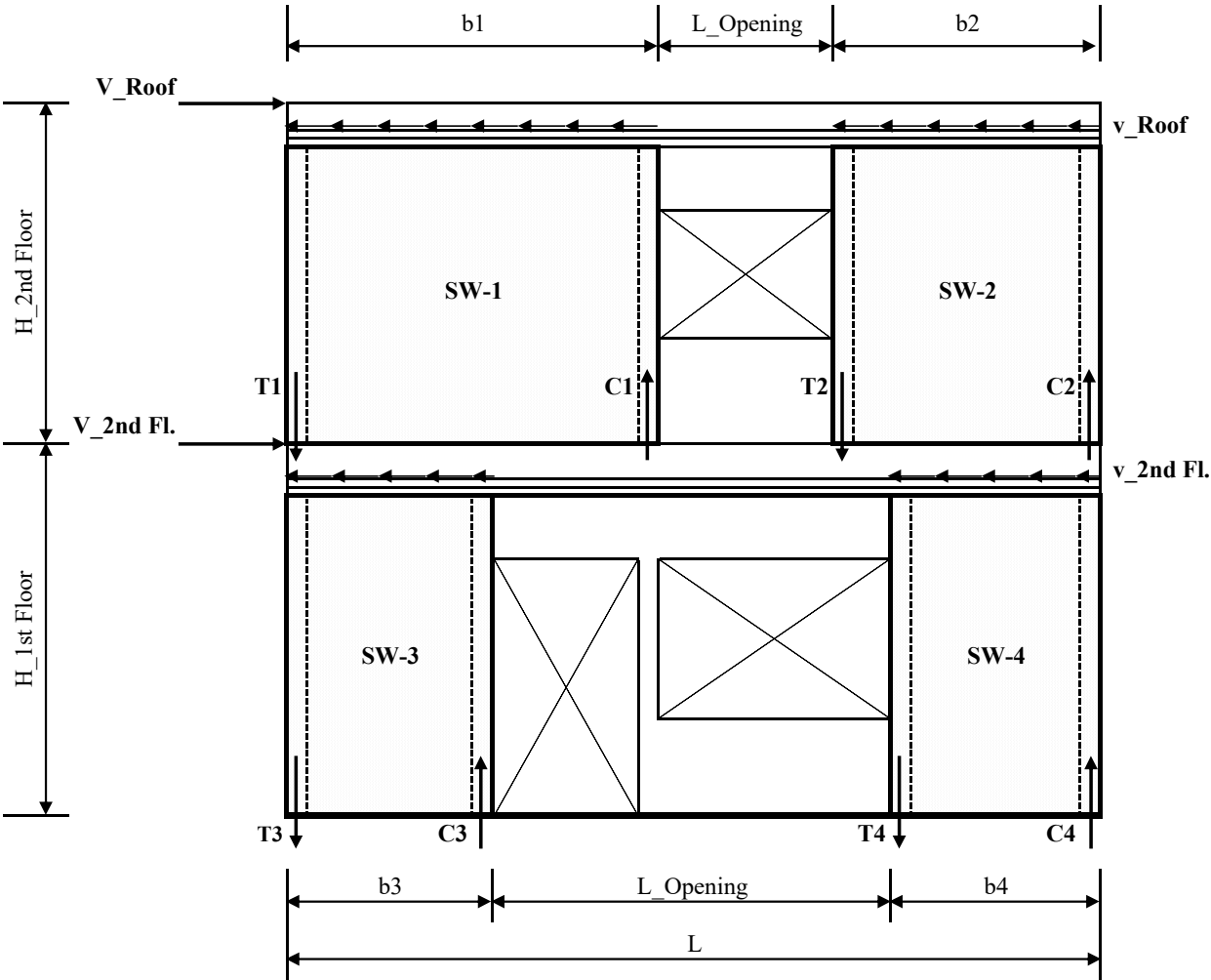
TABLES REFERENCED IN SHEARWALL DESIGN SPREADSHEET (HOLDOWN)

Holdown Capacity

HD Type	Max Uplift Force (lbs)			Post Size
	75%	Full	Used	
HDU2	2,306	3,075	2,306	4x4
HDU4	3,424	4,565	3,424	4x4
HDU5	4,253	5,670	4,253	4x4
HDU8	5,903	7,870	5,903	4x6
HDU11	7,151	9,535	7,151	4x6
HDU14	10,793	14,390	10,793	4x8

NOTES:

1. USE THREADED ROD @ EXISTING FOOTING ONLY. SEE DETAIL 16/S4 LARR# 25279.
2. USE SSTB @ NEW FOOTING ONLY PER LARR# 25248.
3. PROVIDE MIN. POST SIZE AS INDICATED, INCREASE SIZE TO MATCH WALL THICKNESS.
4. ALL HOLDOWN TO BE DOUGLAS FIR-LARCH No. 1.



AT ROOF LEVEL

Total Shearwall Length	ft	=	Sum (b1 + b2 + ..)
Total Shear (V_Roof)	lbs	=	Shear Trib Width * Line Trib Length (L) * Roof Unit Shear (psf)
Unit Shear (v_Roof)	plf	=	V_Roof / Sum(b1 + b2 + ..)
SW-1 Dead Load	lbs	=	Roof_Dead_Laod + Ceiling_Dead_Load + 2nd Fl Wall_Selfweight
SW-1 Uplift Force T1	lbs	=	(Upward Force) - (Downward Force)
		=	(v_Roof * H_2nd Fl) - (0.6 - 0.14*Sds) * SW-1 Dead Load /2)
Drag Force at Open	lbs	=	V Roof / Line Trib Length (L) * L Opening

AT SECOND FLOOR

Total Shearwall Length	ft	=	Sum (b3 + b4 + ..)
Total Shear (V_2nd Fl)	lbs	=	Shear Trib Width * Line Trib Length (L) * 2nd Fl Unit Shear (psf) + Roof Total Shear (V_Roof)
Unit Shear (v_2nd Fl)	plf	=	V_2nd Fl / Sum(b3 + b4 + ..)
SW-3 Dead Load	lbs	=	Roof_Dead_Laod + Ceiling_Dead_Load + 2nd Fl_Wall_Selfweight + Floor_Dead_Load + 1st Fl_Wall_Selfweight
SW-3 Uplift Force T3	lbs	=	(Upward Force) - (Downward Force)
Upward Force	lbs	=	v_Roof * (H_2nd Fl + H_1st Fl) + v_2nd Fl * H_1st Fl
Downward Force	lbs	=	(0.6 - 0.14*Sds) * SW-3 Dead Load /2
Drag Force at Open	lbs	=	V 2nd Fl / Line Trib Length (L) * L Opening

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.57 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE A PERFORATED									
ROOF			SW-01						
Total Shearwall Segments	ΣL_i	ft	14.49						
Total Shearwall Length	L_{tot}	ft	21.00						
Percent Full-Height Sheathing		%	69%						
Total Area of Openings	A_o	ft ²	29.30						
Sheathing Area Ratio	$r = \frac{1}{1 + \frac{A_o}{h \Sigma L_i}}$		0.81						
Shear Capacity Adjustment Factor	$C_o = \left(\frac{r}{3 - 2r} \right) \frac{L_{tot}}{\Sigma L_i}$		0.85						
Shear Trib. Width		ft	14.5						
Line Trib. Length		ft	25.0						
Shortest Panel > H/3.5 and < H/2		ft	3.0						
Line Total Shear	V	lbs	1,657						
Line Unit Shear	$V / (\Sigma L_i \times C_o)$	plf	135	x 4.25 ft / 3 ft =		192	pdf		
100% Wood Shearwall 10d	1		v = 340 plf (1-Side 15/32" Str. 1 w/ 10d Common Nails @ 6"-6"-12" O.C.)						
Roof DL Trib. Width		ft	-						
Ceiling DL Trib. Width		ft	-						
Wall Height		ft	8.5						
Wall Dead Load		plf	153.0						
Uplift Force (Demand)	$T = \frac{Vh}{C_o \Sigma L_i}$	lbs	730						
Holdown Type			HDU4						
Uplift Force (Capacity)		lbs	3,424						

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.57 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE B PERFORATED									
ROOF			SW-02						
Total Shearwall Segments	ΣL_i	ft	14.49						
Total Shearwall Length	L_{tot}	ft	21.00						
Percent Full-Height Sheathing		%	69%						
Total Area of Openings	A_o	ft ²	29.30						
Sheathing Area Ratio	$r = \frac{1}{1 + \frac{A_o}{h \Sigma L_i}}$		0.81						
Shear Capacity Adjustment Factor	$C_o = \left(\frac{r}{3 - 2r} \right) \frac{L_{tot}}{\Sigma L_i}$		0.85						
Shear Trib. Width		ft	14.5						
Line Trib. Length		ft	25.0						
Shortest Panel > H/3.5 and < H/2		ft	3.0						
Line Total Shear	V	lbs	1,657						
Line Unit Shear	$V / (\Sigma L_i \times C_o)$	plf	135	x 4.25 ft / 3 ft =		192	pdf		
100% Wood Shearwall 10d	1		v = 340 plf (1-Side 15/32" Str. 1 w/ 10d Common Nails @ 6"-6"-12" O.C.)						
Roof DL Trib. Width		ft	-						
Ceiling DL Trib. Width		ft	-						
Wall Height		ft	8.5						
Wall Dead Load		plf	153.0						
Uplift Force (Demand)	$T = \frac{Vh}{C_o \Sigma L_i}$	lbs	730						
Holdown Type			HDU4						
Uplift Force (Capacity)		lbs	3,424						

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.57 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE 2 PERFORATED							
ROOF			SW-03				
Total Shearwall Segments	ΣL_i	ft	21.04				
Total Shearwall Length	L_{tot}	ft	25.83				
Percent Full-Height Sheathing		%	81%				
Total Area of Openings	A_o	ft ²	19.53				
Sheathing Area Ratio	$r = \frac{1}{1 + \frac{A_o}{h \Sigma L_i}}$		0.90				
Shear Capacity Adjustment Factor	$C_o = \left(\frac{r}{3 - 2r} \right) \frac{L_{tot}}{\Sigma L_i}$		0.92				
Shear Trib. Width		ft	12.5				
Line Trib. Length		ft	29.0				
Shortest Panel > H/3.5 and < H/2		ft					
Line Total Shear	V	lbs	1,657				
Line Unit Shear	$V / (\Sigma L_i \times C_o)$	plf	85				
100% Wood Shearwall 10d	1		v = 340 plf (1-Side 15/32" Str. 1 w/ 10d Common Nails @ 6"-6"-12" O.C.)				
Roof DL Trib. Width		ft	-				
Ceiling DL Trib. Width		ft	-				
Wall Height		ft	8.5				
Wall Dead Load		plf	153.0				
Uplift Force (Demand)	$T = \frac{Vh}{C_o \Sigma L_i}$	lbs	115				
Holdown Type			HDU4				
Uplift Force (Capacity)		lbs	3,424				

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.57 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE 3 PERFORATED							
ROOF			SW-04				
Total Shearwall Segments	ΣL_i	ft	21.49				
Total Shearwall Length	L_{tot}	ft	25.83				
Percent Full-Height Sheathing		%	83%				
Total Area of Openings	A_o	ft ²	19.53				
Sheathing Area Ratio	$r = \frac{1}{1 + \frac{A_o}{h \Sigma L_i}}$		0.90				
Shear Capacity Adjustment Factor	$C_o = \left(\frac{r}{3 - 2r} \right) \frac{L_{tot}}{\Sigma L_i}$		0.91				
Shear Trib. Width		ft	12.5				
Line Trib. Length		ft	29.0				
Shortest Panel > H/3.5 and < H/2		ft					
Line Total Shear	V	lbs	1,657				
Line Unit Shear	$V / (\Sigma L_i \times C_o)$	plf	85				
100% Wood Shearwall 10d	1		v = 340 plf (1-Side 15/32" Str. 1 w/ 10d Common Nails @ 6"-6"-12" O.C.)				
Roof DL Trib. Width		ft	-				
Ceiling DL Trib. Width		ft	-				
Wall Height		ft	8.5				
Wall Dead Load		plf	153.0				
Uplift Force (Demand)	$T = \frac{Vh}{C_o \Sigma L_i}$	lbs	98				
Holdown Type			HDU4				
Uplift Force (Capacity)		lbs	3,424				

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.57 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE A							
ROOF		SW-05	SW-06	SW-07	SW8-9	SW-10	SW-11
Shearwall Length	ft	34.25	12.75	22.00	11.50	3.08	3.08
Total Shearwall Length	ft	86.66					
Shear Trib. Width	ft	16.0					
Line Trib. Length	ft	158.0					
Line Total Length	ft	158.0					
Line Total Shear	lbs	13,212	TOP ROOF ADDED				
Line Unit Shear	plf	152	x 4 ft / 3.08 ft =		210.4	plf	
75% Wood Shearwall 10d		1	v = 255 plf (1-Side 15/32" Str. I w/ 10d Common Nails @ 6"-6"-12" O.C.)				
Drag Length	ft	6.0					
Drag Force $\times \Omega/1.2$	lbs	1,045					
Strap Type		MST27					
Roof DL Trib. Width	ft	-	-	-	-	-	-
Ceiling DL Trib. Width	ft	-	-	-	-	-	-
Wall Height	ft	8.5	8.5	8.5	8.5	8.5	8.5
Wall Dead Load	plf	153.0	153.0	153.0	153.0	153.0	153.0
Wall Shear Force	lbs	5,222	1,944	3,354	1,753	470	470
Uplift Force (Demand)	lbs	797	1,419	1,152	1,455	1,699	1,699
Holdown Type		HDU4	HDU4	HDU4	HDU4	HDU4	HDU4
Uplift Force (Capacity)	lbs	3,424	3,424	3,424	3,424	3,424	3,424

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.57 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE B							
ROOF		SW-12	SW-13	SW-14	SW-15	SW-16	
Shearwall Length	ft	34.25	12.75	22.00	22.00	16.58	
Total Shearwall Length	ft	107.58					
Shear Trib. Width	ft	16.0					
Line Trib. Length	ft	158.0					
Line Total Length	ft	158.0					
Line Total Shear	lbs	13,212	TOP ROOF ADDED				
Line Unit Shear	plf	123					
75% Wood Shearwall 10d		1	v = 255 plf (1-Side 15/32" Str. I w/ 10d Common Nails @ 6"-6"-12" O.C.)				
Drag Length	ft	6.0					
Drag Force $\times \Omega/1.2$	lbs	1,045					
Strap Type		MST27					
Roof DL Trib. Width	ft	-	-	-	-	-	-
Ceiling DL Trib. Width	ft	-	-	-	-	-	-
Wall Height	ft	8.5	8.5	8.5	8.5	8.5	
Wall Dead Load	plf	153.0	153.0	153.0	153.0	153.0	
Wall Shear Force	lbs	4,206	1,566	2,702	2,702	2,036	
Uplift Force (Demand)	lbs	53	675	407	407	564	
Holdown Type		HDU4	HDU4	HDU4	HDU4	HDU4	
Uplift Force (Capacity)	lbs	3,424	3,424	3,424	3,424	3,424	

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.57 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE 1							
ROOF		SW-17	SW-18				
Shearwall Length	ft	7.67	7.67				
Total Shearwall Length	ft	15.34					
Shear Trib. Width	ft	17.0					
Line Trib. Length	ft	26.0					
Line Total Length	ft	26.0					
Line Total Shear	lbs	2,020					
Line Unit Shear	plf	132					
75% Wood Shearwall 10d		1	v = 255 plf (1-Side 15/32" Str. I w/ 10d Common Nails @ 6"-6"-12" O.C.)				
Drag Length	ft	6.0					
Drag Force $\times \Omega/1.2$	lbs	971					
Strap Type		MST27					
Roof DL Trib. Width	ft	-	-	-	-	-	-
Ceiling DL Trib. Width	ft	-	-	-	-	-	-
Wall Height	ft	8.5	8.5				
Wall Dead Load	plf	153.0	153.0				
Wall Shear Force	lbs	1,010	1,010				
Uplift Force (Demand)	lbs	898	898				
Holdown Type		HDU4	HDU4				
Uplift Force (Capacity)	lbs	3,424	3,424				

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.57 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE 1.5-MF						
ROOF		MF1				
Shearwall Length	ft	7.50				
Total Shearwall Length	ft	7.50				
Shear Trib. Width	ft	26.0				
Line Trib. Length	ft	26.0				
Line Total Length	ft	26.0				
Line Total Shear	lbs	3,090				
Line Unit Shear	plf					
75% Wood Shearwall 10d						
Drag Length	ft	6.0				
Drag Force $\times \Omega/1.2$	lbs					
Strap Type						
Roof DL Trib. Width	ft	-	-	-	-	-
Ceiling DL Trib. Width	ft	-	-	-	-	-
Wall Height	ft	8.5				
Wall Dead Load	plf					
Wall Shear Force	lbs					
Uplift Force (Demand)	lbs					
Holdown Type						
Uplift Force (Capacity)	lbs					

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.57 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE 1.8-MF

ROOF		MF1					
Shearwall Length	ft	7.50					
Total Shearwall Length	ft	7.50					
Shear Trib. Width	ft	26.0					
Line Trib. Length	ft	26.0					
Line Total Length	ft	26.0					
Line Total Shear + TOP RO	lbs	4,747					
Line Unit Shear	plf						
75% Wood Shearwall 10d							
Drag Length	ft	6.0					
Drag Force $\times \Omega/1.2$	lbs						
Strap Type							
Roof DL Trib. Width	ft	-	-	-	-	-	-
Ceiling DL Trib. Width	ft	-	-	-	-	-	-
Wall Height	ft	8.5					
Wall Dead Load	plf						
Wall Shear Force	lbs						
Uplift Force (Demand)	lbs						
Holdown Type							
Uplift Force (Capacity)	lbs						

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.57 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE 3

ROOF	SW-19					
Shearwall Length ft	8.17					
Total Shearwall Length ft	8.17					
Shear Trib. Width ft	17.0					
Line Trib. Length ft	26.0					
Line Total Length ft	26.0					
Line Total Shear +TOP ROO lbs	3,677					
Line Unit Shear plf	450					
75% Wood Shearwall 10d	4	v = 653 plf (1-Side 15/32" Str. I w/ 10d Common Nails @ 2"-6"-12" O.C.)				
Drag Length ft	6.0					
Drag Force $\times \Omega/1.2$ lbs	1,768					
Strap Type	MST27					
Roof DL Trib. Width ft	-	-	-	-	-	-
Ceiling DL Trib. Width ft	-	-	-	-	-	-
Wall Height ft	8.5					
Wall Dead Load plf	153.0					
Wall Shear Force lbs	3,677					
Uplift Force (Demand) lbs	3,589					
Holdown Type	HDU14					
Uplift Force (Capacity) lbs	10,793					

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.57 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE 4							
ROOF		SW-20	SW-21				
Shearwall Length	ft	7.75	4.00				
Total Shearwall Length	ft	11.75					
Shear Trib. Width	ft	25.0					
Line Trib. Length	ft	31.0					
Line Total Length	ft	31.0					
Line Total Shear +TOP ROO lbs		3,542					
Line Unit Shear	plf	301	x 4 ft / 4 ft =	320.3	plf		
75% Wood Shearwall 10d		3	v = 499 plf (1-Side 15/32" Str. I w/ 10d Common Nails @ 3"-6"-12" O.C.)				
Drag Length	ft	6.0					
Drag Force $\times \Omega / 1.2$	lbs	1,428					
Strap Type		MST27					
Roof DL Trib. Width	ft	-	-	-	-	-	-
Ceiling DL Trib. Width	ft	-	-	-	-	-	-
Wall Height	ft	8.5	8.5				
Wall Dead Load	plf	153.0	153.0				
Wall Shear Force	lbs	2,336	1,206				
Uplift Force (Demand)	lbs	2,499	2,607				
Holdown Type		HDU5	HDU5				
Uplift Force (Capacity)	lbs	4,253	4,253				

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.57 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE 5							
ROOF		SW-22	SW-23				
Shearwall Length	ft	9.58	14.00				
Total Shearwall Length	ft	23.58					
Shear Trib. Width	ft	24.0					
Line Trib. Length	ft	31.0					
Line Total Length	ft	31.0					
Line Total Shear +TOP ROO lbs		3,401					
Line Unit Shear	plf	144					
75% Wood Shearwall 10d		1	v = 255 plf (1-Side 15/32" Str. I w/ 10d Common Nails @ 6"-6"-12" O.C.)				
Drag Length	ft	6.0					
Drag Force $\times \Omega/1.2$	lbs	1,371					
Strap Type		MST27					
Roof DL Trib. Width	ft	-	-	-	-	-	-
Ceiling DL Trib. Width	ft	-	-	-	-	-	-
Wall Height	ft	8.5	8.5				
Wall Dead Load	plf	153.0	153.0				
Wall Shear Force	lbs	1,382	2,019				
Uplift Force (Demand)	lbs	949	821				
Holdown Type		HDU4	HDU4				
Uplift Force (Capacity)	lbs	3,424	3,424				

SHEARWALL DESIGN

Fl. Type Story Shear
ROOF 4.57 psf

Wall Type Wall Height
Ext. 8.50 ft

LINE 6						
ROOF	SW-24					
Shearwall Length ft	14.92					
Total Shearwall Length ft	14.92					
Shear Trib. Width ft	13.5					
Line Trib. Length ft	31.0					
Line Total Length ft	31.0					
Line Total Shear +TOP ROO lbs	1,913					
Line Unit Shear plf	128					
75% Wood Shearwall 10d	1	v = 255 plf (1-Side 15/32" Str. I w/ 10d Common Nails @ 6"-6"-12" O.C.)				
Drag Length ft	6.0					
Drag Force $\times \Omega/1.2$ lbs	771					
Strap Type	MST27					
Roof DL Trib. Width ft	-	-	-	-	-	-
Ceiling DL Trib. Width ft	-	-	-	-	-	-
Wall Height ft	8.5					
Wall Dead Load plf	153.0					
Wall Shear Force lbs	1,913					
Uplift Force (Demand) lbs	658					
Holdown Type	HDU4					
Uplift Force (Capacity) lbs	3,424					

TABLES REFERENCED IN BEAM DESIGN SPREADSHEET

2018 NDS Table 4A

Douglas Fir-Larch ¹ WWPA	Design Values for Visually Graded (2"-4")		
	fb	fv	E
Select Str	1,500	180	1,900,000
No.1 & Better	1,200	180	1,800,000
No.1	1,000	180	1,700,000
No.2	900	180	1,600,000
No.3	525	180	1,400,000
Stud	700	180	1,400,000

2018 NDS Table 4D

Douglas Fir-Larch ¹ WWPA	Design Values for Visually Graded (5" & Larger)		
	fb	fv	E
Dense Str	1,900	170	1,700,000
Select Str	1,600	170	1,600,000
Dense No. 1	1,550	170	1,700,000
No.1	1,350	170	1,600,000
Dense No. 2	1,000	170	1,400,000
No.2	875	170	1,300,000

Other Material

Prllm2.0E	2,900	290	2,000,000
Mcrlm1.8E	2,600	285	1,800,000
Mcrlm1.9E	2,600	285	1,900,000
Steel Fy=36 ksi	23,760	14400	29,000,000
Steel Fy=46 ksi	30,360	18400	29,000,000
Steel Fy=50 ksi	33,000	20000	29,000,000

1. FOR DOUGLAS FIR-LARCH USE 19% MAX. MOISTURE CONTENT

LOAD SYMBOLS AND NOTATION

D = Dead Load	E = Earthquake Load
Lr = Roof Live Load	W = Wind Load
L = Floor Live Load	

ASCE7-16 2.4 LOAD COMBINATION - ALLOWABLE STRESS DESIGN

- | | | |
|--|----------------------|--|
| 1. D | | |
| 2. D + Lr | Load Duration = 1.25 | } Max. (Dead + Live) |
| 3. D + L | | |
| 4. D + 0.75 (Lr + L) | Load Duration = 1.25 | |
| 5. $(1.0 + 0.14S_{DS})D + 0.7\Omega E$ | | } Max. (Dead + Live + Lateral)
Allowable Stress Increase = 1.2
$\Omega = 2.5$ For Flexible Diaphragm |
| 6. $(1.0 + 0.105S_{DS})D + 0.525\Omega E + 0.75(L + Lr)$ | | |
| 7. $(0.6 - 0.14S_{DS})D + 0.7\Omega E$ | | |
- ASCE7-16 12.4.3.2

TABLE 1604.3 (2022 CBC) DEFLECTION LIMITS

Roof Members	L	D + L
Supporting Plaster Ceiling	$l / 360$	$l / 240$
Supporting Nonplaster Ceiling	$l / 240$	$l / 180$
Not Supporting Ceiling	$l / 180$	$l / 120$
Floor Members	$l / 360$	$l / 240$

SIMPLY SUPPORTED BEAM DESIGN

	RR	RR1	RR2	RR3	CJ	CJ1
SPAN ft	8.00	12.00	19.00	21.00	6.00	10.00
MATERIAL (WOOD, STEEL)	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION in	2x 6	2x 8	2x10	2x12	2x 4	2x 6
GRADE	No.2	No.2	No.2	No.2	No.2	No.2
ALLOWABLE STRESSES						
REPETITIVE MEMBER	YES	YES	YES	YES	YES	YES
REPETITIVE FACTOR, Cr	1.15	1.15	1.15	1.15	1.15	1.15
SIZE FACTOR, Cf	1.30	1.20	1.10	1.00	1.50	1.30
BENDING STRESSES, F'b psi	1,346	1,242	1,139	1,035	1,553	1,346
SHEAR STRESSES, F'v psi	207	207	207	207	207	207
MODULUS OF ELASTICITY ksi	1,600	1,600	1,600	1,600	1,600	1,600
UNIFORM DISTRIBUTED LOAD OVER FULL SPAN						
ROOF TRIB. WIDTH ft	1.33	1.33	1.33	1.33		
CEILING TRIB. WIDTH ft					1.33	1.33
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft						
WALL TYPE (EXT, INT)						
AUTO CALC BEAM WEIGHT						
TOTAL DEAD LOAD (D plf	17.29	17.29	17.29	17.29	7.98	7.98
TOTAL ROOF LIVE LOAD (L _r plf	26.60	26.60	26.60	26.60	-	-
TOTAL FLOOR LIVE LOAD (L plf	-	-	-	-	26.60	26.60
PARTIALLY DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
CEILING TRIB. WIDTH ft						
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft						
WALL TYPE (EXT, INT)						
TOTAL DEAD LOAD (D plf	-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf	-	-	-	-	-	-
TOTAL FLOOR LIVE LOAD (L plf	-	-	-	-	-	-
FROM (LEFT SUPPORT) ft						
LENGTH OF LOAD ft						
TRIANGULAR DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
TOTAL DEAD LOAD (D plf	-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf	-	-	-	-	-	-
CONCENTRATED LOADS						
P1						
DEAD LOAD (P D) lbs						
ROOF LIVE LOAD (P L _r) lbs						
FLOOR LIVE LOAD (P L) lbs						
LATERAL, P (W or 0.7E) lbs						
DIST. (LEFT SUPPORT) ft						
P2						
DEAD LOAD (P D) lbs						
ROOF LIVE LOAD (P L _r) lbs						
FLOOR LIVE LOAD (P L) lbs						
LATERAL, P (W or 0.7E) lbs						
DIST. (LEFT SUPPORT) ft						

SIMPLY SUPPORTED BEAM DESIGN

		RR	RR1	RR2	RR3	CJ	CJ1
SPAN	ft	8.00	12.00	19.00	21.00	6.00	10.00
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	2x 6	2x 8	2x10	2x12	2x 4	2x 6
GRADE		No.2	No.2	No.2	No.2	No.2	No.2
P3							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
P4							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
LOAD CASE 1: D							
MAX. BENDING MOMENT lb-ft		138	311	780	953	36	100
MAX. DEFLECTION in		0.048	0.106	0.320	0.266	0.027	0.054
LEFT REACTION lbs		69	104	164	182	24	40
RIGHT REACTION lbs		69	104	164	182	24	40
LOAD CASE: Lr							
MAX. BENDING MOMENT lb-ft		213	479	1,200	1,466	-	-
MAX. DEFLECTION in		0.074	0.163	0.493	0.409	-	-
LEFT REACTION lbs		106	160	253	279	-	-
RIGHT REACTION lbs		106	160	253	279	-	-
LOAD CASE: L							
MAX. BENDING MOMENT lb-ft		-	-	-	-	120	333
MAX. DEFLECTION in		-	-	-	-	0.090	0.180
LEFT REACTION lbs		-	-	-	-	80	133
RIGHT REACTION lbs		-	-	-	-	80	133
LOAD CASE 2: D + Lr							
		Wood Load Duration Factor = 1.25					
MAX. BENDING MOMENT lb-ft		281	632	1,585	1,936	29	80
MAX. DEFLECTION in		0.122	0.269	0.813	0.674	0.027	0.054
LEFT REACTION lbs		140	211	334	369	19	32
RIGHT REACTION lbs		140	211	334	369	19	32
LOAD CASE 3: D + L							
		Wood Load Duration Factor = 1.0					
MAX. BENDING MOMENT lb-ft		138	311	780	953	156	432
MAX. DEFLECTION in		0.048	0.106	0.320	0.266	0.118	0.234
LEFT REACTION lbs		69	104	164	182	104	173
RIGHT REACTION lbs		69	104	164	182	104	173
LOAD CASE 4: D + 0.75 (Lr + L)							
		Wood Load Duration Factor = 1.25					
MAX. BENDING MOMENT lb-ft		238	536	1,344	1,642	101	279
MAX. DEFLECTION in		0.103	0.228	0.690	0.572	0.095	0.189
LEFT REACTION lbs		149	223	354	391	84	140
RIGHT REACTION lbs		149	223	354	391	84	140
LOAD CASE 5: (1.0 + 0.14S _{ds})D + 0.7ΩE							
		Wood Load Duration Factor = 1.6					
MAX. BENDING MOMENT lb-ft		106	238	596	728	27	76
MAX. DEFLECTION in		0.049	0.108	0.326	0.271	0.028	0.055
LEFT REACTION lbs		70	106	167	185	24	41
RIGHT REACTION lbs		70	106	167	185	24	41

SIMPLY SUPPORTED BEAM DESIGN

		RR	RR1	RR2	RR3	CJ	CJ1
SPAN	ft	8.00	12.00	19.00	21.00	6.00	10.00
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	2x 6	2x 8	2x10	2x12	2x 4	2x 6
GRADE		No.2	No.2	No.2	No.2	No.2	No.2
LOAD CASE 6: $(1.0 + 0.105S_{Ds})D + 0.525\Omega E + 0.75(L + L_r)$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	201	451	1,131	1,382	82	229
MAX. DEFLECTION	in	0.093	0.205	0.619	0.514	0.083	0.165
LEFT REACTION	lbs	134	201	318	351	73	122
RIGHT REACTION	lbs	134	201	318	351	73	122
LOAD CASE 7: $(0.6 - 0.14S_{Ds})D + 0.7\Omega E$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	33	74	184	225	8	24
MAX. DEFLECTION	in	0.015	0.033	0.101	0.084	0.009	0.017
LEFT REACTION	lbs	22	33	52	57	8	13
RIGHT REACTION	lbs	22	33	52	57	8	13
CHECK BENDING STRESSES							
MAX. BENDING MOMENT	lb-ft	280.9	632.1	1,584.5	1,935.7	155.6	432.3
SECTION MODULUS, S	in ³	7.6	13.1	21.4	31.6	3.1	7.6
BENDING STRESS	psi	445.8	577.2	888.9	734.1	609.7	685.9
CHECK BENDING STRESS RATIO		33% OK	46% OK	78% OK	71% OK	39% OK	51% OK
CHECK SHEAR STRESSES							
MAX. SHEAR FORCE	lbs	149.0	223.46	353.81	391.05	103.74	172.90
SECTION AREA, A	in ²	8.25	10.88	13.88	16.88	5.25	8.25
SHEAR STRESS, fv	psi	27.09	30.82	38.25	34.76	29.64	31.44
CHECK SHEAR STRESS RATIO		13% OK	15% OK	18% OK	17% OK	14% OK	15% OK
CHECK DEFLECTION							
MOMENT OF INERTIA, I	in ⁴	20.80	47.63	98.93	177.98	5.36	20.80
MAX. DEFLECTION, DL	in	0.048	0.106	0.320	0.266	0.027	0.054
MAX. DEFLECTION, LL	in	0.074	0.163	0.493	0.409	0.090	0.180
DEFLECTION LIMITS, LL		L / 240	L / 360	L / 360	L / 360	L / 360	L / 360
CHECK DEFLECTION ΔLL		< 0.4 OK	< 0.4 OK	< 0.63 OK	< 0.7 OK	< 0.2 OK	< 0.33 OK
MAX. DEFLECTION, DL+LL	in	0.122	0.269	0.813	0.674	0.118	0.234
DEFLECTION LIMITS, DL+LL		L / 180	L / 240	L / 240	L / 240	L / 240	L / 240
CHECK DEFLECTION Δ(DL+LL)		< 0.53 OK	< 0.6 OK	< 0.95 OK	< 1.05 OK	< 0.3 OK	< 0.5 OK

SIMPLY SUPPORTED BEAM DESIGN

	RR	RR1	RR2	RDG1	HDR1	HDR2
SPAN ft	15.00	18.00	21.00	25.50	2.50	11.00
MATERIAL (WOOD, STEEL)	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION in	2x 8	2x10	2x12	6x16	6x10	6x10
GRADE	No.2	No.2	No.2	No.1	No.2	No.2
ALLOWABLE STRESSES						
REPETITIVE MEMBER	YES	YES	YES	NO	NO	NO
REPETITIVE FACTOR, Cr	1.15	1.15	1.15	1.00	1.00	1.00
SIZE FACTOR, Cf	1.20	1.10	1.00	0.97	1.00	1.00
BENDING STRESSES, F'b psi	1,242	1,139	1,035	1,308	875	875
SHEAR STRESSES, F'v psi	207	207	207	170	170	170
MODULUS OF ELASTICITY ksi	1,600	1,600	1,600	1,700	1,600	1,600
UNIFORM DISTRIBUTED LOAD OVER FULL SPAN						
ROOF TRIB. WIDTH ft	1.33	1.33	1.33	10.00	1.33	0.67
CEILING TRIB. WIDTH ft						
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft						6.00
WALL TYPE (EXT, INT)						EXT
AUTO CALC BEAM WEIGHT						
TOTAL DEAD LOAD (D plf	17.29	17.29	17.29	130.03	17.29	116.71
TOTAL ROOF LIVE LOAD (L ₁ plf	26.60	26.60	26.60	200.00	26.60	13.40
TOTAL FLOOR LIVE LOAD (L plf	-	-	-	-	-	-
PARTIALLY DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
CEILING TRIB. WIDTH ft						
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft						
WALL TYPE (EXT, INT)						
TOTAL DEAD LOAD (D plf	-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L ₁ plf	-	-	-	-	-	-
TOTAL FLOOR LIVE LOAD (L plf	-	-	-	-	-	-
FROM (LEFT SUPPORT) ft						
LENGTH OF LOAD ft						
TRIANGULAR DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
TOTAL DEAD LOAD (D plf	-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L ₁ plf	-	-	-	-	-	-
CONCENTRATED LOADS						
P1					RDG1	
DEAD LOAD (P D) lbs					1,657.8	
ROOF LIVE LOAD (P Lr) lbs					2,550.0	
FLOOR LIVE LOAD (P L) lbs					-	
LATERAL, P (W or 0.7E) lbs					-	
DIST. (LEFT SUPPORT) ft					1.25	
P2						
DEAD LOAD (P D) lbs						
ROOF LIVE LOAD (P Lr) lbs						
FLOOR LIVE LOAD (P L) lbs						
LATERAL, P (W or 0.7E) lbs						
DIST. (LEFT SUPPORT) ft						

SIMPLY SUPPORTED BEAM DESIGN

		RR	RR1	RR2	RDG1	HDR1	HDR2
SPAN	ft	15.00	18.00	21.00	25.50	2.50	11.00
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	2x 8	2x10	2x12	6x16	6x10	6x10
GRADE		No.2	No.2	No.2	No.1	No.2	No.2
P3							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
P4							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
LOAD CASE 1: D							
MAX. BENDING MOMENT lb-ft		486	700	953	10,569	1,050	1,765
MAX. DEFLECTION in		0.258	0.258	0.266	0.426	0.002	0.061
LEFT REACTION lbs		130	156	182	1,658	851	642
RIGHT REACTION lbs		130	156	182	1,658	851	642
LOAD CASE: Lr							
MAX. BENDING MOMENT lb-ft		748	1,077	1,466	16,256	1,615	203
MAX. DEFLECTION in		0.398	0.397	0.409	0.656	0.002	0.007
LEFT REACTION lbs		200	239	279	2,550	1,308	74
RIGHT REACTION lbs		200	239	279	2,550	1,308	74
LOAD CASE: L							
MAX. BENDING MOMENT lb-ft		-	-	-	-	-	-
MAX. DEFLECTION in		-	-	-	-	-	-
LEFT REACTION lbs		-	-	-	-	-	-
RIGHT REACTION lbs		-	-	-	-	-	-
LOAD CASE 2: D + Lr					Wood Load Duration Factor = 1.25		
MAX. BENDING MOMENT lb-ft		988	1,422	1,936	21,460	2,131	1,574
MAX. DEFLECTION in		0.656	0.655	0.674	1.082	0.004	0.068
LEFT REACTION lbs		263	316	369	3,366	1,727	572
RIGHT REACTION lbs		263	316	369	3,366	1,727	572
LOAD CASE 3: D + L					Wood Load Duration Factor = 1.0		
MAX. BENDING MOMENT lb-ft		486	700	953	10,569	1,050	1,765
MAX. DEFLECTION in		0.258	0.258	0.266	0.426	0.002	0.061
LEFT REACTION lbs		130	156	182	1,658	851	642
RIGHT REACTION lbs		130	156	182	1,658	851	642
LOAD CASE 4: D + 0.75 (Lr + L)					Wood Load Duration Factor = 1.25		
MAX. BENDING MOMENT lb-ft		838	1,207	1,642	18,209	1,808	1,534
MAX. DEFLECTION in		0.557	0.556	0.572	0.918	0.003	0.066
LEFT REACTION lbs		279	335	391	3,570	1,832	697
RIGHT REACTION lbs		279	335	391	3,570	1,832	697
LOAD CASE 5: (1.0 + 0.14S _{DS})D + 0.7ΩE					Wood Load Duration Factor = 1.6		
MAX. BENDING MOMENT lb-ft		371	535	728	8,070	802	1,348
MAX. DEFLECTION in		0.263	0.263	0.271	0.434	0.002	0.062
LEFT REACTION lbs		132	158	185	1,688	866	654
RIGHT REACTION lbs		132	158	185	1,688	866	654

SIMPLY SUPPORTED BEAM DESIGN

		RR	RR1	RR2	RDG1	HDR1	HDR2
SPAN	ft	15.00	18.00	21.00	25.50	2.50	11.00
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	2x 8	2x10	2x12	6x16	6x10	6x10
GRADE		No.2	No.2	No.2	No.1	No.2	No.2
LOAD CASE 6: $(1.0 + 0.105S_{Ds})D + 0.525\Omega E + 0.75(L + L_r)$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	705	1,016	1,382	15,324	1,522	1,382
MAX. DEFLECTION	in	0.500	0.499	0.514	0.824	0.003	0.064
LEFT REACTION	lbs	251	301	351	3,205	1,644	670
RIGHT REACTION	lbs	251	301	351	3,205	1,644	670
LOAD CASE 7: $(0.6 - 0.14S_{Ds})D + 0.7\Omega E$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	115	166	225	2,498	248	417
MAX. DEFLECTION	in	0.081	0.081	0.084	0.134	0.000	0.019
LEFT REACTION	lbs	41	49	57	523	268	202
RIGHT REACTION	lbs	41	49	57	523	268	202
CHECK BENDING STRESSES							
MAX. BENDING MOMENT	lb-ft	987.6	1,422.1	1,935.7	21,459.9	2,131.3	1,765.3
SECTION MODULUS, S	in ³	13.1	21.4	31.6	220.2	82.7	82.7
BENDING STRESS	psi	901.9	797.8	734.1	1,169.3	309.2	256.1
CHECK BENDING STRESS RATIO		73% OK	70% OK	71% OK	89% OK	35% OK	29% OK
CHECK SHEAR STRESSES							
MAX. SHEAR FORCE	lbs	279.3	335.19	391.05	3,570.32	1,831.71	697.19
SECTION AREA, A	in ²	10.88	13.88	16.88	85.25	52.25	52.25
SHEAR STRESS, fv	psi	38.53	36.24	34.76	62.82	52.59	20.02
CHECK SHEAR STRESS RATIO		19% OK	18% OK	17% OK	37% OK	31% OK	12% OK
CHECK DEFLECTION							
MOMENT OF INERTIA, I	in ⁴	47.63	98.93	177.98	1,706.78	392.96	392.96
MAX. DEFLECTION, DL	in	0.258	0.258	0.266	0.426	0.002	0.061
MAX. DEFLECTION, LL	in	0.398	0.397	0.409	0.656	0.002	0.007
DEFLECTION LIMITS, LL		L / 360	L / 360	L / 360	L / 360	L / 360	L / 360
CHECK DEFLECTION ALL		< 0.5 OK	< 0.6 OK	< 0.7 OK	< 0.85 OK	< 0.08 OK	< 0.37 OK
MAX. DEFLECTION, DL+LL	in	0.656	0.655	0.674	1.082	0.004	0.068
DEFLECTION LIMITS, DL+LL		L / 240	L / 240	L / 240	L / 240	L / 240	L / 240
CHECK DEFLECTION Δ(DL+LL)		< 0.75 OK	< 0.9 OK	< 1.05 OK	< 1.28 OK	< 0.13 OK	< 0.55 OK

SIMPLY SUPPORTED BEAM DESIGN

	FB1	FB2	HDR3	HDR4	CB1	CB2
SPAN ft	20.00	20.00	17.00	10.50	18.50	13.00
MATERIAL (WOOD, STEEL)	WOOD Prllm	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION in	7x11.875	6x14	6x14	6x14	6x16	6x12
GRADE	Prllm2.2E	No.1	No.1	No.1	No.1	No.1
ALLOWABLE STRESSES						
REPETITIVE MEMBER	NO	NO	NO	NO	NO	NO
REPETITIVE FACTOR, Cr	1.00	1.00	1.00	1.00	1.00	1.00
SIZE FACTOR, Cf	1.00	0.98	0.98	0.98	0.97	1.00
BENDING STRESSES, F'b psi	2,900	1,327	1,327	1,327	1,308	1,350
SHEAR STRESSES, F'v psi	290	170	170	170	170	170
MODULUS OF ELASTICITY ksi	2,200	1,700	1,700	1,700	1,700	1,700
UNIFORM DISTRIBUTED LOAD OVER FULL SPAN						
ROOF TRIB. WIDTH ft			13.00	8.00		10.50
CEILING TRIB. WIDTH ft						
FLOOR TRIB. WIDTH ft	9.00	5.50		5.50		
DECK TRIB. WIDTH ft						
WALL HEIGHT ft			4.00	12.00		
WALL TYPE (EXT, INT)			EXT	EXT		
AUTO CALC BEAM WEIGHT						
TOTAL DEAD LOAD (D plf	117.00	71.50	241.03	391.52	-	136.53
TOTAL ROOF LIVE LOAD (L _r plf	-	-	260.00	160.00	-	210.00
TOTAL FLOOR LIVE LOAD (L plf	360.00	220.00	-	220.00	-	-
PARTIALLY DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
CEILING TRIB. WIDTH ft						
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft						
WALL TYPE (EXT, INT)						
TOTAL DEAD LOAD (D plf	-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf	-	-	-	-	-	-
TOTAL FLOOR LIVE LOAD (L plf	-	-	-	-	-	-
FROM (LEFT SUPPORT) ft						
LENGTH OF LOAD ft						
TRIANGULAR DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
TOTAL DEAD LOAD (D plf	-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf	-	-	-	-	-	-
CONCENTRATED LOADS						
P1				RDG2	RDG2	
DEAD LOAD (P D) lbs				610.0	2,040.0	
ROOF LIVE LOAD (P L _r) lbs				1,020.0	2,920.0	
FLOOR LIVE LOAD (P L) lbs				-	-	
LATERAL, P (W or 0.7E) lbs				-	-	
DIST. (LEFT SUPPORT) ft				5.00	9.25	
P2						
DEAD LOAD (P D) lbs						
ROOF LIVE LOAD (P L _r) lbs						
FLOOR LIVE LOAD (P L) lbs						
LATERAL, P (W or 0.7E) lbs						
DIST. (LEFT SUPPORT) ft						

SIMPLY SUPPORTED BEAM DESIGN

		FB1	FB2	HDR3	HDR4	CB1	CB2
SPAN	ft	20.00	20.00	17.00	10.50	18.50	13.00
MATERIAL (WOOD, STEEL)		WOOD Prllm	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	7x11.875	6x14	6x14	6x14	6x16	6x12
GRADE		Prllm2.2E	No.1	No.1	No.1	No.1	No.1
P3							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
P4							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
LOAD CASE 1: D							
MAX. BENDING MOMENT lb-ft		5,850	3,575	8,707	6,973	9,435	2,884
MAX. DEFLECTION in		0.196	0.134	0.236	0.069	0.160	0.074
LEFT REACTION lbs		1,170	715	2,049	2,375	1,020	887
RIGHT REACTION lbs		1,170	715	2,049	2,346	1,020	887
LOAD CASE: Lr							
MAX. BENDING MOMENT lb-ft		-	-	9,393	4,853	13,505	4,436
MAX. DEFLECTION in		-	-	0.255	0.045	0.229	0.114
LEFT REACTION lbs		-	-	2,210	1,374	1,460	1,365
RIGHT REACTION lbs		-	-	2,210	1,326	1,460	1,365
LOAD CASE: L							
MAX. BENDING MOMENT lb-ft		18,000	11,000	-	3,032	-	-
MAX. DEFLECTION in		0.603	0.413	-	0.031	-	-
LEFT REACTION lbs		3,600	2,200	-	1,155	-	-
RIGHT REACTION lbs		3,600	2,200	-	1,155	-	-
LOAD CASE 2: D + Lr							
		Wood Load Duration Factor = 1.25					
MAX. BENDING MOMENT lb-ft		4,680	2,860	14,480	9,461	18,352	5,856
MAX. DEFLECTION in		0.196	0.134	0.491	0.114	0.390	0.188
LEFT REACTION lbs		936	572	3,407	2,999	1,984	1,802
RIGHT REACTION lbs		936	572	3,407	2,937	1,984	1,802
LOAD CASE 3: D + L							
		Wood Load Duration Factor = 1.0					
MAX. BENDING MOMENT lb-ft		23,850	14,575	8,707	10,000	9,435	2,884
MAX. DEFLECTION in		0.799	0.547	0.236	0.100	0.160	0.074
LEFT REACTION lbs		4,770	2,915	2,049	3,530	1,020	887
RIGHT REACTION lbs		4,770	2,915	2,049	3,501	1,020	887
LOAD CASE 4: D + 0.75 (Lr + L)							
		Wood Load Duration Factor = 1.25					
MAX. BENDING MOMENT lb-ft		15,480	9,460	12,601	10,307	15,651	4,969
MAX. DEFLECTION in		0.648	0.444	0.427	0.126	0.332	0.159
LEFT REACTION lbs		3,870	2,365	3,706	4,272	2,115	1,911
RIGHT REACTION lbs		3,870	2,365	3,706	4,206	2,115	1,911
LOAD CASE 5: (1.0 + 0.14S _{ds})D + 0.7ΩE							
		Wood Load Duration Factor = 1.6					
MAX. BENDING MOMENT lb-ft		4,467	2,730	6,649	5,325	7,205	2,202
MAX. DEFLECTION in		0.200	0.137	0.241	0.070	0.163	0.075
LEFT REACTION lbs		1,191	728	2,086	2,418	1,038	904
RIGHT REACTION lbs		1,191	728	2,086	2,388	1,038	904

SIMPLY SUPPORTED BEAM DESIGN

		FB1	FB2	HDR3	HDR4	CB1	CB2
SPAN	ft	20.00	20.00	17.00	10.50	18.50	13.00
MATERIAL (WOOD, STEEL)		WOOD Prllm	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	7x11.875	6x14	6x14	6x14	6x16	6x12
GRADE		Prllm2.2E	No.1	No.1	No.1	No.1	No.1
LOAD CASE 6: $(1.0 + 0.105S_{Ds})D + 0.525\Omega E + 0.75(L + L_r)$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	12,702	7,762	10,750	8,777	13,208	4,182
MAX. DEFLECTION	in	0.567	0.389	0.389	0.115	0.299	0.143
LEFT REACTION	lbs	3,387	2,070	3,373	3,889	1,904	1,716
RIGHT REACTION	lbs	3,387	2,070	3,373	3,831	1,904	1,716
LOAD CASE 7: $(0.6 - 0.14S_{Ds})D + 0.7\Omega E$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	1,383	845	2,058	1,648	2,230	682
MAX. DEFLECTION	in	0.062	0.042	0.074	0.022	0.051	0.023
LEFT REACTION	lbs	369	225	646	749	322	280
RIGHT REACTION	lbs	369	225	646	739	322	280
CHECK BENDING STRESSES							
MAX. BENDING MOMENT	lb-ft	23,850.0	14,575.0	14,479.8	10,306.7	18,352.0	5,856.3
SECTION MODULUS, S	in ³	164.5	167.1	167.1	167.1	220.2	121.2
BENDING STRESS	psi	1,739.6	1,046.9	1,040.1	740.3	1,000.0	579.7
CHECK BENDING STRESS RATIO		60% OK	79% OK	78% OK	56% OK	76% OK	43% OK
CHECK SHEAR STRESSES							
MAX. SHEAR FORCE	lbs	4,770.0	2,915.00	3,706.28	4,271.97	2,115.00	1,911.17
SECTION AREA, A	in ²	83.13	74.25	74.25	74.25	85.25	63.25
SHEAR STRESS, fv	psi	86.08	58.89	74.87	86.30	37.21	45.32
CHECK SHEAR STRESS RATIO		30% OK	35% OK	44% OK	51% OK	22% OK	27% OK
CHECK DEFLECTION							
MOMENT OF INERTIA, I	in ⁴	976.83	1,127.67	1,127.67	1,127.67	1,706.78	697.07
MAX. DEFLECTION, DL	in	0.196	0.134	0.236	0.069	0.160	0.074
MAX. DEFLECTION, LL	in	0.603	0.413	0.255	0.057	0.229	0.114
DEFLECTION LIMITS, LL		L / 360	L / 360	L / 360	L / 360	L / 360	L / 360
CHECK DEFLECTION Δ_{LL}		< 0.67 OK	< 0.67 OK	< 0.57 OK	< 0.35 OK	< 0.62 OK	< 0.43 OK
MAX. DEFLECTION, DL+LL	in	0.799	0.547	0.491	0.126	0.390	0.188
DEFLECTION LIMITS, DL+LL		L / 240	L / 240	L / 240	L / 240	L / 240	L / 240
CHECK DEFLECTION $\Delta(DL+LL)$		< 1 OK	< 1 OK	< 0.85 OK	< 0.53 OK	< 0.93 OK	< 0.65 OK

SIMPLY SUPPORTED BEAM DESIGN

	CB3	CB4	CB5	HDR5	HDR6	HDR7
SPAN ft	25.00	10.50	22.00	10.50	10.50	4.00
MATERIAL (WOOD, STEEL)	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION in	6x12	6x12	6x16	6x12	6x10	6x10
GRADE	No.1	No.1	No.1	No.1	No.1	No.1
ALLOWABLE STRESSES						
REPETITIVE MEMBER	NO	NO	NO	NO	NO	NO
REPETITIVE FACTOR, Cr			1.00	1.00	1.00	1.00
SIZE FACTOR, Cf			0.97	1.00	1.00	1.00
BENDING STRESSES, F'b psi			1,308	1,350	1,350	1,350
SHEAR STRESSES, F'v psi			170	170	170	170
MODULUS OF ELASTICITY ksi			1,700	1,700	1,700	1,700
UNIFORM DISTRIBUTED LOAD OVER FULL SPAN						
ROOF TRIB. WIDTH ft	5.00	1.33		5.50	1.33	5.50
CEILING TRIB. WIDTH ft		6.50		6.50		5.50
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft		4.00		4.00	4.00	4.00
WALL TYPE (EXT, INT)		EXT		EXT	EXT	EXT
AUTO CALC BEAM WEIGHT						
TOTAL DEAD LOAD (D plf)			-	182.51	89.29	176.51
TOTAL ROOF LIVE LOAD (L _r plf)			-	110.00	26.60	110.00
TOTAL FLOOR LIVE LOAD (L plf)			-	130.00	-	110.00
PARTIALLY DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
CEILING TRIB. WIDTH ft						
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft						
WALL TYPE (EXT, INT)						
TOTAL DEAD LOAD (D plf)			-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf)			-	-	-	-
TOTAL FLOOR LIVE LOAD (L plf)			-	-	-	-
FROM (LEFT SUPPORT) ft						
LENGTH OF LOAD ft						
TRIANGULAR DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
TOTAL DEAD LOAD (D plf)			-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf)			-	-	-	-
CONCENTRATED LOADS						
P1			RDG3	RDG2	RDG3	RDG4
DEAD LOAD (P D) lbs		610.0	1,720.0	610.0	550.0	350.0
ROOF LIVE LOAD (P L _r) lbs		1,020.0	2,600.0	1,020.0	930.0	1,040.0
FLOOR LIVE LOAD (P L) lbs		-	-	-	-	-
LATERAL, P (W or 0.7E) lbs		-	-	-	-	-
DIST. (LEFT SUPPORT) ft		5.25	11.00	5.25	5.25	2.00
P2						
DEAD LOAD (P D) lbs						
ROOF LIVE LOAD (P L _r) lbs						
FLOOR LIVE LOAD (P L) lbs						
LATERAL, P (W or 0.7E) lbs						
DIST. (LEFT SUPPORT) ft						

SIMPLY SUPPORTED BEAM DESIGN

		CB3	CB4	CB5	HDR5	HDR6	HDR7
SPAN	ft	25.00	10.50	22.00	10.50	10.50	4.00
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	6x12	6x12	6x16	6x12	6x10	6x10
GRADE		No.1	No.1	No.1	No.1	No.1	No.1
P3							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
P4							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
LOAD CASE 1: D							
MAX. BENDING MOMENT lb-ft				9,460	4,117	2,674	703
MAX. DEFLECTION in				0.227	0.064	0.071	0.003
LEFT REACTION lbs				860	1,263	744	528
RIGHT REACTION lbs				860	1,263	744	528
LOAD CASE: Lr							
MAX. BENDING MOMENT lb-ft				14,300	4,193	2,808	1,260
MAX. DEFLECTION in				0.343	0.061	0.069	0.005
LEFT REACTION lbs				1,300	1,088	605	740
RIGHT REACTION lbs				1,300	1,088	605	740
LOAD CASE: L							
MAX. BENDING MOMENT lb-ft				-	1,792	-	220
MAX. DEFLECTION in				-	0.030	-	0.001
LEFT REACTION lbs				-	683	-	220
RIGHT REACTION lbs				-	683	-	220
LOAD CASE 2: D + Lr					Wood Load Duration Factor = 1.25		
MAX. BENDING MOMENT lb-ft				19,008	6,648	4,386	1,570
MAX. DEFLECTION in				0.571	0.125	0.140	0.007
LEFT REACTION lbs				1,728	1,881	1,079	1,014
RIGHT REACTION lbs				1,728	1,881	1,079	1,014
LOAD CASE 3: D + L					Wood Load Duration Factor = 1.0		
MAX. BENDING MOMENT lb-ft				9,460	5,908	2,674	923
MAX. DEFLECTION in				0.227	0.094	0.071	0.004
LEFT REACTION lbs				860	1,946	744	748
RIGHT REACTION lbs				860	1,946	744	748
LOAD CASE 4: D + 0.75 (Lr + L)					Wood Load Duration Factor = 1.25		
MAX. BENDING MOMENT lb-ft				16,148	6,884	3,824	1,450
MAX. DEFLECTION in				0.485	0.132	0.123	0.007
LEFT REACTION lbs				1,835	2,591	1,197	1,248
RIGHT REACTION lbs				1,835	2,591	1,197	1,248
LOAD CASE 5: (1.0 + 0.14S _{DS})D + 0.7ΩE					Wood Load Duration Factor = 1.6		
MAX. BENDING MOMENT lb-ft				7,224	3,143	2,042	537
MAX. DEFLECTION in				0.231	0.065	0.072	0.003
LEFT REACTION lbs				876	1,286	757	538
RIGHT REACTION lbs				876	1,286	757	538

SIMPLY SUPPORTED BEAM DESIGN

		CB3	CB4	CB5	HDR5	HDR6	HDR7
SPAN	ft	25.00	10.50	22.00	10.50	10.50	4.00
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L
BEAM SECTION	in	6x12	6x12	6x16	6x12	6x10	6x10
GRADE		No.1	No.1	No.1	No.1	No.1	No.1
LOAD CASE 6: $(1.0 + 0.105S_{Ds})D + 0.525\Omega E + 0.75(L + L_r)$				Wood Load Duration Factor = 1.6			
MAX. BENDING MOMENT	lb-ft			13,599	5,806	3,266	1,206
MAX. DEFLECTION	in			0.436	0.119	0.112	0.006
LEFT REACTION	lbs			1,648	2,334	1,101	1,113
RIGHT REACTION	lbs			1,648	2,334	1,101	1,113
LOAD CASE 7: $(0.6 - 0.14S_{Ds})D + 0.7\Omega E$				Wood Load Duration Factor = 1.6			
MAX. BENDING MOMENT	lb-ft			2,236	973	632	166
MAX. DEFLECTION	in			0.072	0.020	0.022	0.001
LEFT REACTION	lbs			271	398	234	166
RIGHT REACTION	lbs			271	398	234	166
CHECK BENDING STRESSES							
MAX. BENDING MOMENT	lb-ft			19,008.0	6,884.2	4,385.7	1,570.4
SECTION MODULUS, S	in ³			220.2	121.2	82.7	82.7
BENDING STRESS	psi			1,035.7	681.4	636.2	227.8
CHECK BENDING STRESS RATIO				79% OK	50% OK	47% OK	17% OK
CHECK SHEAR STRESSES							
MAX. SHEAR FORCE	lbs			1,835.00	2,590.70	1,197.28	1,248.03
SECTION AREA, A	in ²			85.25	63.25	52.25	52.25
SHEAR STRESS, fv	psi			32.29	61.44	34.37	35.83
CHECK SHEAR STRESS RATIO				19% OK	36% OK	20% OK	21% OK
CHECK DEFLECTION							
MOMENT OF INERTIA, I	in ⁴			1,706.78	697.07	392.96	392.96
MAX. DEFLECTION, DL	in			0.227	0.064	0.071	0.003
MAX. DEFLECTION, LL	in			0.343	0.068	0.069	0.005
DEFLECTION LIMITS, LL		L / 360	L / 360	L / 360	L / 360	L / 360	L / 360
CHECK DEFLECTION ALL				< 0.73 OK	< 0.35 OK	< 0.35 OK	< 0.13 OK
MAX. DEFLECTION, DL+LL	in			0.571	0.132	0.140	0.007
DEFLECTION LIMITS, DL+LL		L / 240	L / 240	L / 240	L / 240	L / 240	L / 240
CHECK DEFLECTION Δ(DL+LL)				< 1.1 OK	< 0.53 OK	< 0.53 OK	< 0.2 OK

SIMPLY SUPPORTED BEAM DESIGN

	CB6	CB7	CB8	FB3	STRINGER	FB4
SPAN ft	5.50	9.50	3.50	8.00	18.50	5.50
MATERIAL (WOOD, STEEL)	WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	STEEL HSS	WOOD DF-L
BEAM SECTION in	6x12	4x12	6x12	6x12	HSS12x 3x1/4	6x12
GRADE	No.1	No.1	No.1	No.1	Fy=46 ksi	No.1
ALLOWABLE STRESSES						
REPETITIVE MEMBER	NO	NO	NO	NO	NO	NO
REPETITIVE FACTOR, Cr	1.00	1.00	1.00	1.00	1.00	1.00
SIZE FACTOR, Cf	1.00	1.10	1.00	1.00	1.00	1.00
BENDING STRESSES, F'b psi	1,350	1,100	1,350	1,350	30,360	1,350
SHEAR STRESSES, F'v psi	170	180	170	170	18,400	170
MODULUS OF ELASTICITY ksi	1,700	1,700	1,700	1,700	29,000	1,700
UNIFORM DISTRIBUTED LOAD OVER FULL SPAN						
ROOF TRIB. WIDTH ft	1.33	9.00	6.00	9.00		
CEILING TRIB. WIDTH ft	4.50					
FLOOR TRIB. WIDTH ft				5.50		
DECK TRIB. WIDTH ft					3.00	4.00
WALL HEIGHT ft			4.00	12.00	3.50	
WALL TYPE (EXT, INT)			EXT	EXT	EXT	
AUTO CALC BEAM WEIGHT						
TOTAL DEAD LOAD (D plf)	44.29	117.02	150.02	404.52	102.00	52.00
TOTAL ROOF LIVE LOAD (L _r plf)	26.60	180.00	120.00	180.00	-	-
TOTAL FLOOR LIVE LOAD (L plf)	90.00	-	-	220.00	300.00	400.00
PARTIALLY DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
CEILING TRIB. WIDTH ft						
FLOOR TRIB. WIDTH ft						
DECK TRIB. WIDTH ft						
WALL HEIGHT ft						
WALL TYPE (EXT, INT)						
TOTAL DEAD LOAD (D plf)	-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf)	-	-	-	-	-	-
TOTAL FLOOR LIVE LOAD (L plf)	-	-	-	-	-	-
FROM (LEFT SUPPORT) ft						
LENGTH OF LOAD ft						
TRIANGULAR DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
TOTAL DEAD LOAD (D plf)	-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf)	-	-	-	-	-	-
CONCENTRATED LOADS						
P1	RDG4		CB2	CB2		STRINGER
DEAD LOAD (P D) lbs	2,930.0		887.4	887.4		943.5
ROOF LIVE LOAD (P L _r) lbs	4,140.0		1,365.0	1,365.0		-
FLOOR LIVE LOAD (P L) lbs	-		-	-		2,775.0
LATERAL, P (W or 0.7E) lbs	-		-	-		-
DIST. (LEFT SUPPORT) ft	3.50		1.75	4.00		1.00
P2						STRINGER
DEAD LOAD (P D) lbs						943.5
ROOF LIVE LOAD (P L _r) lbs						-
FLOOR LIVE LOAD (P L) lbs						2,775.0
LATERAL, P (W or 0.7E) lbs						-
DIST. (LEFT SUPPORT) ft						4.00

SIMPLY SUPPORTED BEAM DESIGN

		CB6	CB7	CB8	FB3	STRINGER	FB4
SPAN	ft	5.50	9.50	3.50	8.00	18.50	5.50
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	STEEL HSS	WOOD DF-L
BEAM SECTION	in	6x12	4x12	6x12	6x12	HSS12x 3x1/4	6x12
GRADE		No.1	No.1	No.1	No.1	Fy=46 ksi	No.1
P3							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
P4							
DEAD LOAD (P D) lbs							
ROOF LIVE LOAD (P Lr) lbs							
FLOOR LIVE LOAD (P L) lbs							
LATERAL, P (W or 0.7E) lbs							
DIST. (LEFT SUPPORT) ft							
LOAD CASE 1: D							
MAX. BENDING MOMENT lb-ft		3,846	1,320	1,006	5,011	4,364	1,442
MAX. DEFLECTION in		0.014	0.030	0.002	0.045	0.090	0.007
LEFT REACTION lbs		1,187	556	706	2,062	944	1,172
RIGHT REACTION lbs		1,986	556	706	2,062	944	1,001
LOAD CASE: Lr							
MAX. BENDING MOMENT lb-ft		5,309	2,031	1,378	4,170	-	-
MAX. DEFLECTION in		0.019	0.047	0.002	0.035	-	-
LEFT REACTION lbs		1,579	855	893	1,403	-	-
RIGHT REACTION lbs		2,708	855	893	1,403	-	-
LOAD CASE: L							
MAX. BENDING MOMENT lb-ft		340	-	-	1,760	12,834	5,061
MAX. DEFLECTION in		0.002	-	-	0.017	0.265	0.025
LEFT REACTION lbs		248	-	-	880	2,775	4,127
RIGHT REACTION lbs		248	-	-	880	2,775	3,623
LOAD CASE 2: D + Lr					Wood Load Duration Factor = 1.25		
MAX. BENDING MOMENT lb-ft		7,324	2,681	1,907	7,345	4,364	1,153
MAX. DEFLECTION in		0.034	0.077	0.004	0.080	0.090	0.007
LEFT REACTION lbs		2,213	1,129	1,279	2,771	755	938
RIGHT REACTION lbs		3,755	1,129	1,279	2,771	755	801
LOAD CASE 3: D + L					Wood Load Duration Factor = 1.0		
MAX. BENDING MOMENT lb-ft		4,160	1,320	1,006	6,771	17,198	6,484
MAX. DEFLECTION in		0.016	0.030	0.002	0.062	0.355	0.032
LEFT REACTION lbs		1,435	556	706	2,942	3,719	5,300
RIGHT REACTION lbs		2,234	556	706	2,942	3,719	4,623
LOAD CASE 4: D + 0.75 (Lr + L)					Wood Load Duration Factor = 1.25		
MAX. BENDING MOMENT lb-ft		6,451	2,275	1,632	7,567	13,989	4,175
MAX. DEFLECTION in		0.030	0.065	0.003	0.085	0.289	0.025
LEFT REACTION lbs		2,557	1,197	1,376	3,774	3,025	4,268
RIGHT REACTION lbs		4,203	1,197	1,376	3,774	3,025	3,718
LOAD CASE 5: (1.0 + 0.14S _{ds})D + 0.7ΩE					Wood Load Duration Factor = 1.6		
MAX. BENDING MOMENT lb-ft		2,937	1,008	768	3,826	4,443	1,101
MAX. DEFLECTION in		0.014	0.031	0.002	0.046	0.092	0.007
LEFT REACTION lbs		1,209	566	719	2,099	961	1,194
RIGHT REACTION lbs		2,022	566	719	2,099	961	1,019

SIMPLY SUPPORTED BEAM DESIGN

		CB6	CB7	CB8	FB3	STRINGER	FB4
SPAN	ft	5.50	9.50	3.50	8.00	18.50	5.50
MATERIAL (WOOD, STEEL)		WOOD DF-L	WOOD DF-L	WOOD DF-L	WOOD DF-L	STEEL HSS	WOOD DF-L
BEAM SECTION	in	6x12	4x12	6x12	6x12	HSS12x 3x1/4	6x12
GRADE		No.1	No.1	No.1	No.1	Fy=46 ksi	No.1
LOAD CASE 6: $(1.0 + 0.105S_{Ds})D + 0.525\Omega E + 0.75(L + L_r)$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	5,439	1,914	1,379	6,432	12,263	3,410
MAX. DEFLECTION	in	0.027	0.059	0.003	0.077	0.253	0.022
LEFT REACTION	lbs	2,295	1,075	1,244	3,430	2,651	3,719
RIGHT REACTION	lbs	3,778	1,075	1,244	3,430	2,651	3,237
LOAD CASE 7: $(0.6 - 0.14S_{Ds})D + 0.7\Omega E$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	909	312	238	1,185	1,375	341
MAX. DEFLECTION	in	0.004	0.010	0.000	0.014	0.028	0.002
LEFT REACTION	lbs	374	175	223	650	297	370
RIGHT REACTION	lbs	626	175	223	650	297	315
CHECK BENDING STRESSES							
MAX. BENDING MOMENT	lb-ft	7,324.2	2,680.6	1,907.5	7,566.8	17,198.1	6,483.5
SECTION MODULUS, S	in ³	121.2	73.8	121.2	121.2	17.2	121.2
BENDING STRESS	psi	725.0	435.7	188.8	749.0	11,998.6	641.8
CHECK BENDING STRESS RATIO		54% OK	40% OK	14% OK	55% OK	40% OK	48% OK
CHECK SHEAR STRESSES							
MAX. SHEAR FORCE	lbs	4,202.7	1,197.11	1,375.61	3,773.68	3,718.50	5,299.55
SECTION AREA, A	in ²	63.25	39.38	63.25	63.25	6.63	63.25
SHEAR STRESS, fv	psi	99.67	45.60	32.62	89.49	841.29	125.68
CHECK SHEAR STRESS RATIO		59% OK	25% OK	19% OK	53% OK	5% OK	74% OK
CHECK DEFLECTION							
MOMENT OF INERTIA, I	in ⁴	697.07	415.28	697.07	697.07	103.00	697.07
MAX. DEFLECTION, DL	in	0.014	0.030	0.002	0.045	0.090	0.007
MAX. DEFLECTION, LL	in	0.019	0.047	0.002	0.039	0.265	0.025
DEFLECTION LIMITS, LL		L / 360	L / 360	L / 360	L / 360	L / 360	L / 360
CHECK DEFLECTION ALL		< 0.18 OK	< 0.32 OK	< 0.12 OK	< 0.27 OK	< 0.62 OK	< 0.18 OK
MAX. DEFLECTION, DL+LL	in	0.034	0.077	0.004	0.085	0.355	0.032
DEFLECTION LIMITS, DL+LL		L / 240	L / 240	L / 240	L / 240	L / 240	L / 240
CHECK DEFLECTION Δ(DL+LL)		< 0.28 OK	< 0.48 OK	< 0.18 OK	< 0.4 OK	< 0.93 OK	< 0.28 OK

SIMPLY SUPPORTED BEAM DESIGN

	FB5	FB6	FB7	FB8	FB9	FB11
SPAN ft	10.50	6.50	7.50	21.00	20.00	10.50
MATERIAL (WOOD, STEEL)	WOOD Prllm	WOOD Prllm	WOOD DF-L	WOOD Prllm	WOOD Prllm	WOOD DF-L
BEAM SECTION in	5.25x11.25	5.25x11.25	6x12	5.25x16	7x11.25	6x12
GRADE	Prllm2.0E	Prllm2.2E	No.1	Prllm2.2E	Prllm2.2E	No.1
ALLOWABLE STRESSES						
REPETITIVE MEMBER	NO	NO	NO	NO	NO	NO
REPETITIVE FACTOR, Cr	1.00	1.00	1.00	1.00	1.00	1.00
SIZE FACTOR, Cf	1.00	1.00	1.00	1.00	1.00	1.00
BENDING STRESSES, F'b psi	2,900	2,900	1,350	2,900	2,900	1,350
SHEAR STRESSES, F'v psi	290	290	170	290	290	170
MODULUS OF ELASTICITY ksi	2,000	2,200	1,700	2,200	2,200	1,700
UNIFORM DISTRIBUTED LOAD OVER FULL SPAN						
ROOF TRIB. WIDTH ft		8.00	8.00	1.33	8.50	1.33
CEILING TRIB. WIDTH ft						
FLOOR TRIB. WIDTH ft	1.33	1.33	2.50		1.50	1.33
DECK TRIB. WIDTH ft						
WALL HEIGHT ft	3.50	8.00	8.00	8.00	8.00	8.00
WALL TYPE (EXT, INT)	EXT	EXT	EXT	EXT	EXT	EXT
AUTO CALC BEAM WEIGHT						
TOTAL DEAD LOAD (D plf)	80.29	265.31	280.52	161.29	274.02	178.58
TOTAL ROOF LIVE LOAD (L _r plf)	-	160.00	160.00	26.60	170.00	26.60
TOTAL FLOOR LIVE LOAD (L plf)	53.20	53.20	100.00	-	60.00	53.20
PARTIALLY DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
CEILING TRIB. WIDTH ft						
FLOOR TRIB. WIDTH ft			3.00			
DECK TRIB. WIDTH ft						
WALL HEIGHT ft						
WALL TYPE (EXT, INT)						
TOTAL DEAD LOAD (D plf)	-	-	39.00	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf)	-	-	-	-	-	-
TOTAL FLOOR LIVE LOAD (L plf)	-	-	120.00	-	-	-
FROM (LEFT SUPPORT) ft			5.50			
LENGTH OF LOAD ft			2.00			
TRIANGULAR DISTRIBUTED LOAD						
ROOF TRIB. WIDTH ft						
TOTAL DEAD LOAD (D plf)	-	-	-	-	-	-
TOTAL ROOF LIVE LOAD (L _r plf)	-	-	-	-	-	-
CONCENTRATED LOADS						
P1	FB4	FB8	FB5	HDR1		HDR1
DEAD LOAD (P D) lbs	1,000.7	2,851.5	1,041.0	850.5		850.5
ROOF LIVE LOAD (P L _r) lbs	-	1,647.3	-	1,308.3		1,308.3
FLOOR LIVE LOAD (P L) lbs	3,622.7	837.6	2,521.9	-		-
LATERAL, P (W or 0.7E) lbs	-	-	-	-		-
DIST. (LEFT SUPPORT) ft	6.50	1.00	5.50	9.00		1.50
P2				HDR1		
DEAD LOAD (P D) lbs				850.5		
ROOF LIVE LOAD (P L _r) lbs				1,308.3		
FLOOR LIVE LOAD (P L) lbs				-		
LATERAL, P (W or 0.7E) lbs				-		
DIST. (LEFT SUPPORT) ft				11.50		

SIMPLY SUPPORTED BEAM DESIGN

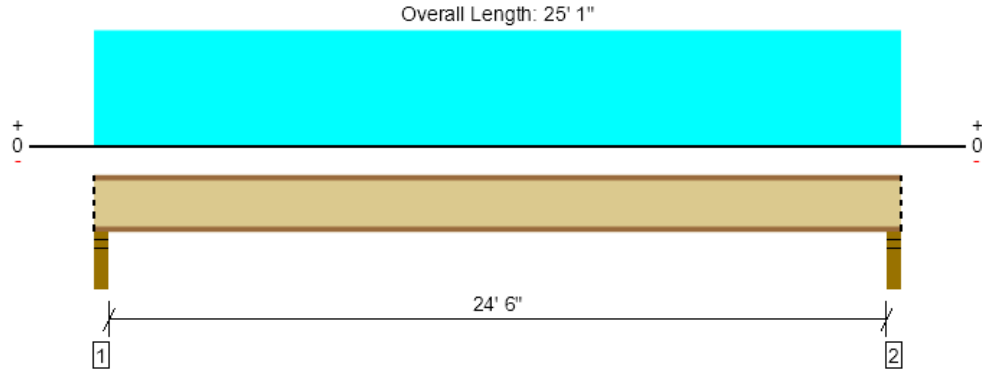
		FB5	FB6	FB7	FB8	FB9	FB11
SPAN	ft	10.50	6.50	7.50	21.00	20.00	10.50
MATERIAL (WOOD, STEEL)		WOOD Prllm	WOOD Prllm	WOOD DF-L	WOOD Prllm	WOOD Prllm	WOOD DF-L
BEAM SECTION	in	5.25x11.25	5.25x11.25	6x12	5.25x16	7x11.25	6x12
GRADE		Prllm2.0E	Prllm2.2E	No.1	Prllm2.2E	Prllm2.2E	No.1
P3					FB4		
DEAD LOAD (P D) lbs					1,172.3		
ROOF LIVE LOAD (P Lr) lbs					-		
FLOOR LIVE LOAD (P L) lbs					4,127.3		
LATERAL, P (W or 0.7E) lbs					-		
DIST. (LEFT SUPPORT) ft					17.00		
P4					FB7		
DEAD LOAD (P D) lbs					1,340.0		
ROOF LIVE LOAD (P Lr) lbs					600.0		
FLOOR LIVE LOAD (P L) lbs					1,079.5		
LATERAL, P (W or 0.7E) lbs					-		
DIST. (LEFT SUPPORT) ft					20.00		
LOAD CASE 1: D							
MAX. BENDING MOMENT lb-ft		3,515	3,189	3,200	19,974	13,701	3,140
MAX. DEFLECTION in		0.049	0.017	0.027	0.390	0.540	0.054
LEFT REACTION lbs		803	3,275	1,340	2,851	2,740	1,667
RIGHT REACTION lbs		1,041	1,301	1,883	4,749	2,740	1,059
LOAD CASE: Lr							
MAX. BENDING MOMENT lb-ft		-	1,869	1,125	13,908	8,500	1,845
MAX. DEFLECTION in		-	0.010	0.010	0.253	0.335	0.026
LEFT REACTION lbs		-	1,914	600	1,647	1,700	1,261
RIGHT REACTION lbs		-	773	600	2,128	1,700	327
LOAD CASE: L							
MAX. BENDING MOMENT lb-ft		9,639	855	4,371	14,071	3,000	733
MAX. DEFLECTION in		0.124	0.004	0.031	0.207	0.118	0.012
LEFT REACTION lbs		1,659	882	1,080	838	600	279
RIGHT REACTION lbs		2,522	302	2,432	4,369	600	279
LOAD CASE 2: D + Lr					Wood Load Duration Factor = 1.25		
MAX. BENDING MOMENT lb-ft		2,812	4,046	3,417	27,106	17,761	3,743
MAX. DEFLECTION in		0.049	0.027	0.037	0.643	0.875	0.079
LEFT REACTION lbs		642	4,151	1,552	3,599	3,552	2,342
RIGHT REACTION lbs		833	1,659	1,986	5,501	3,552	1,108
LOAD CASE 3: D + L					Wood Load Duration Factor = 1.0		
MAX. BENDING MOMENT lb-ft		13,154	4,032	7,517	29,685	16,701	3,863
MAX. DEFLECTION in		0.172	0.021	0.058	0.595	0.658	0.066
LEFT REACTION lbs		2,462	4,157	2,419	3,689	3,340	1,946
RIGHT REACTION lbs		3,563	1,603	4,315	9,118	3,340	1,338
LOAD CASE 4: D + 0.75 (Lr + L)					Wood Load Duration Factor = 1.25		
MAX. BENDING MOMENT lb-ft		8,595	4,177	5,684	30,023	17,861	3,842
MAX. DEFLECTION in		0.142	0.028	0.057	0.733	0.880	0.082
LEFT REACTION lbs		2,047	5,372	2,600	4,715	4,465	2,822
RIGHT REACTION lbs		2,932	2,107	4,157	9,622	4,465	1,513
LOAD CASE 5: (1.0 + 0.14S_{ds})D + 0.7ΩE					Wood Load Duration Factor = 1.6		
MAX. BENDING MOMENT lb-ft		2,684	2,435	2,444	15,252	10,462	2,398
MAX. DEFLECTION in		0.049	0.017	0.027	0.397	0.550	0.055
LEFT REACTION lbs		817	3,334	1,364	2,903	2,790	1,697
RIGHT REACTION lbs		1,060	1,325	1,917	4,835	2,790	1,078

SIMPLY SUPPORTED BEAM DESIGN

		FB5	FB6	FB7	FB8	FB9	FB11
SPAN	ft	10.50	6.50	7.50	21.00	20.00	10.50
MATERIAL (WOOD, STEEL)		WOOD Prllm	WOOD Prllm	WOOD DF-L	WOOD Prllm	WOOD Prllm	WOOD DF-L
BEAM SECTION	in	5.25x11.25	5.25x11.25	6x12	5.25x16	7x11.25	6x12
GRADE		Prllm2.0E	Prllm2.2E	No.1	Prllm2.2E	Prllm2.2E	No.1
LOAD CASE 6: $(1.0 + 0.105S_{Ds})D + 0.525\Omega E + 0.75(L + L_r)$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	7,080	3,595	4,767	25,532	15,378	3,327
MAX. DEFLECTION	in	0.125	0.026	0.051	0.665	0.808	0.076
LEFT REACTION	lbs	1,817	4,930	2,352	4,324	4,101	2,583
RIGHT REACTION	lbs	2,588	1,936	3,725	8,676	4,101	1,408
LOAD CASE 7: $(0.6 - 0.14S_{Ds})D + 0.7\Omega E$ Wood Load Duration Factor = 1.6							
MAX. BENDING MOMENT	lb-ft	831	754	757	4,722	3,239	742
MAX. DEFLECTION	in	0.015	0.005	0.008	0.123	0.170	0.017
LEFT REACTION	lbs	253	1,032	422	899	864	525
RIGHT REACTION	lbs	328	410	594	1,497	864	334
CHECK BENDING STRESSES							
MAX. BENDING MOMENT	lb-ft	13,153.6	4,177.5	7,517.2	30,023.3	17,860.9	3,862.7
SECTION MODULUS, S	in ³	110.7	110.7	121.2	224.0	147.7	121.2
BENDING STRESS	psi	1,425.3	452.7	744.1	1,608.4	1,451.5	382.4
CHECK BENDING STRESS RATIO		49% OK	16% OK	55% OK	55% OK	50% OK	28% OK
CHECK SHEAR STRESSES							
MAX. SHEAR FORCE	lbs	3,563.0	5,371.61	4,315.39	9,621.79	4,465.21	2,821.82
SECTION AREA, A	in ²	59.06	59.06	63.25	84.00	78.75	63.25
SHEAR STRESS, fv	psi	90.49	136.42	102.34	171.82	85.05	66.92
CHECK SHEAR STRESS RATIO		31% OK	47% OK	60% OK	59% OK	29% OK	39% OK
CHECK DEFLECTION							
MOMENT OF INERTIA, I	in ⁴	622.92	622.92	697.07	1,792.00	830.57	697.07
MAX. DEFLECTION, DL	in	0.049	0.017	0.027	0.390	0.540	0.054
MAX. DEFLECTION, LL	in	0.124	0.011	0.031	0.343	0.340	0.028
DEFLECTION LIMITS, LL		L / 360	L / 360	L / 360	L / 360	L / 360	L / 360
CHECK DEFLECTION ΔLL		< 0.35 OK	< 0.22 OK	< 0.25 OK	< 0.7 OK	< 0.67 OK	< 0.35 OK
MAX. DEFLECTION, DL+LL	in	0.172	0.028	0.058	0.733	0.880	0.082
DEFLECTION LIMITS, DL+LL		L / 240	L / 240	L / 240	L / 240	L / 240	L / 240
CHECK DEFLECTION Δ(DL+LL)		< 0.53 OK	< 0.33 OK	< 0.38 OK	< 1.05 OK	< 1 OK	< 0.53 OK

Level, Roof: Joist

1 piece(s) 14" TJI® 360 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	552 @ 2 1/2"	1881 (3.50")	Passed (29%)	1.25	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	539 @ 3 1/2"	2131	Passed (25%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	3346 @ 12' 6 1/2"	7725	Passed (43%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.567 @ 12' 6 1/2"	0.822	Passed (L/522)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.935 @ 12' 6 1/2"	1.233	Passed (L/317)	--	1.0 D + 1.0 Lr (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

Member Length : 25' 1"
System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 0/12

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Roof Live	Factored	
1 - Stud wall - SPF	3.50"	3.50"	1.75"	217	334	552	Blocking
2 - Stud wall - SPF	3.50"	3.50"	1.75"	217	334	552	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 2" o/c	
Bottom Edge (Lu)	25' 1" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

Vertical Load	Location	Spacing	Dead (0.90)	Roof Live (1.25)	Comments
1 - Uniform (PSF)	0 to 25' 1"	16"	13.0	20.0	Default Load

Weyerhaeuser Notes

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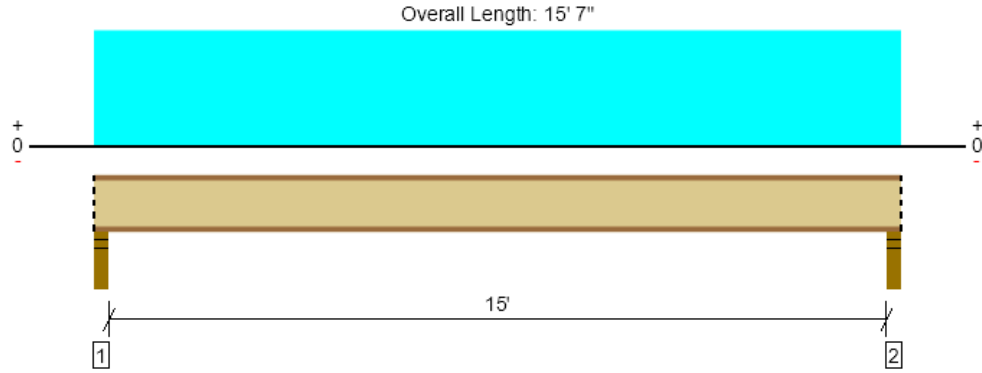
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
feby awad N/A (714) 642-1004 febynawad@gmail.com	



Level, FLOOR: Joist

1 piece(s) 14" TJI® 360 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1454 @ 2 1/2"	1881 (3.50")	Passed (77%)	1.25	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	1400 @ 3 1/2"	2444	Passed (57%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	5367 @ 7' 9 1/2"	9169	Passed (59%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.397 @ 7' 9 1/2"	0.758	Passed (L/458)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.445 @ 7' 9 1/2"	1.011	Passed (L/409)	--	1.0 D + 1.0 Lr (All Spans)

- Deflection criteria: LL (L/240) and TL (L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.

Member Length : 15' 7"
System : Roof
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2021
Design Methodology : ASD
Member Pitch : 0/12

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Roof Live	Factored	
1 - Stud wall - SPF	3.50"	3.50"	2.09"	156	1299	1454	Blocking
2 - Stud wall - SPF	3.50"	3.50"	2.09"	156	1299	1454	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' 5" o/c	
Bottom Edge (Lu)	15' 7" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

Vertical Load	Location	Spacing	Dead (0.90)	Roof Live (1.25)	Comments
1 - Uniform (PSF)	0 to 15' 7"	16"	15.0	125.0	Default Load

Weyerhaeuser Notes

Weyerhaeuser warrants that the sizing of its products will be in accordance with Weyerhaeuser product design criteria and published design values. Weyerhaeuser expressly disclaims any other warranties related to the software. Use of this software is not intended to circumvent the need for a design professional as determined by the authority having jurisdiction. The designer of record, builder or framer is responsible to assure that this calculation is compatible with the overall project. Accessories (Rim Board, Blocking Panels and Squash Blocks) are not designed by this software. Products manufactured at Weyerhaeuser facilities are third-party certified to sustainable forestry standards. Weyerhaeuser Engineered Lumber Products have been evaluated by ICC-ES under evaluation reports ESR-1153 and ESR-1387 and/or tested in accordance with applicable ASTM standards. For current code evaluation reports, Weyerhaeuser product literature and installation details refer to www.weyerhaeuser.com/woodproducts/document-library.

The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Atef Gerges FA Structural (714) 519-5027 atefg@fastructural.com	



8/18/2025 9:13:07 PM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: RJ STORAGE
Page 44

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

(c) ENERCALC INC 1983-2023

DESCRIPTION: RDG2

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-22 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design

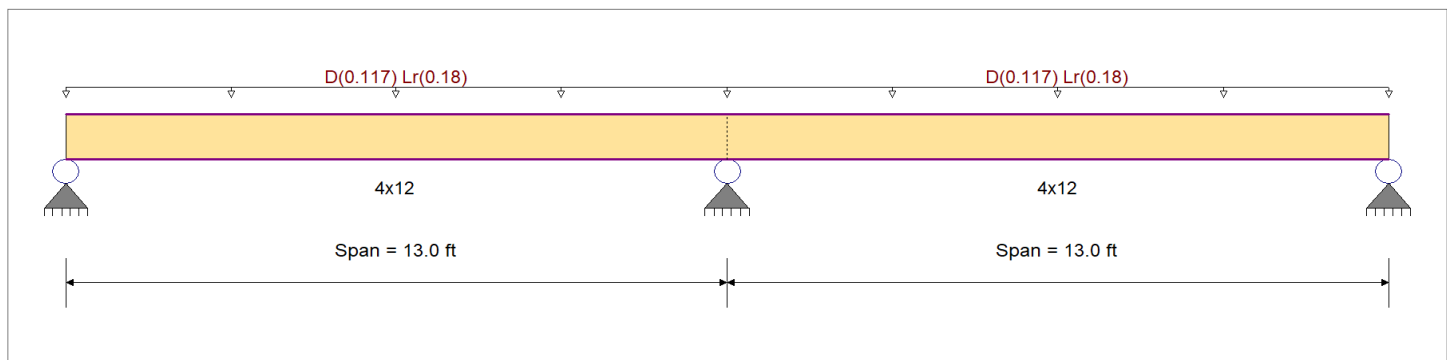
Load Combination : ASCE 7-22 / IBC 2024

Wood Species : Douglas Fir-Larch

Wood Grade : No.1

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

Fb +	1,350.0 psi	E : Modulus of Elasticity	
Fb -	1,350.0 psi	Ebend- xx	1,600.0ksi
Fc - Prll	925.0 psi	Eminbend - xx	580.0ksi
Fc - Perp	625.0 psi		
Fv	170.0 psi		
Ft	675.0 psi	Density	31.210pcf



Applied Loads

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

Beam self weight calculated and added to loading

Load for Span Number 1

Uniform Load : D = 0.0130, Lr = 0.020 ksf, Tributary Width = 9.0 ft, (RR)

Load for Span Number 2

Uniform Load : D = 0.0130, Lr = 0.020 ksf, Tributary Width = 9.0 ft, (RR)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.565	1	Maximum Shear Stress Ratio	=	0.397	1
Section used for this span		4x12		Section used for this span		4x12	
fb: Actual	=	1,049.10psi		fv: Actual	=	84.43 psi	
F'b	=	1,856.25psi		F'v	=	212.50 psi	
Load Combination	+D+Lr+H, LL Comb Run (LL)			Load Combination	+D+Lr+H, LL Comb Run (LL)		
Location of maximum on span	=	13.000ft		Location of maximum on span	=	12.128 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.124 in	Ratio =	1263 >=360	Span: 2 : Lr Only, LL Comb Run (*L)			
Max Upward Transient Deflection	-0.054 in	Ratio =	2868 >=360	Span: 2 : Lr Only, LL Comb Run (L*)			
Max Downward Total Deflection	0.174 in	Ratio =	896 >=240	Span: 2 : +D+Lr+H, LL Comb Run (*L)			
Max Upward Total Deflection	-0.025 in	Ratio =	6307 >=240	Span: 2 : +D+Lr+H, LL Comb Run (L*)			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
+D+H														0.0	0.00	0.0	0.0
Length = 13.0 ft	1	0.323	0.227	0.90	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,336.5	0.91	34.7	153.0
Length = 13.0 ft	2	0.323	0.227	0.90	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,336.5	0.91	34.7	153.0
+D+L+H, LL Comb Run (*L)														0.0	0.00	0.0	0.0
Length = 13.0 ft	1	0.290	0.204	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,485.0	0.91	34.7	170.0
Length = 13.0 ft	2	0.290	0.204	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,485.0	0.91	34.7	170.0
+D+L+H, LL Comb Run (L*)														0.0	0.00	0.0	0.0
Length = 13.0 ft	1	0.290	0.204	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,485.0	0.91	34.7	170.0

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 Phone : (949) 337-7075

Project Title: 85-25 - E 20th St
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

(c) ENERCALC INC 1983-2023

DESCRIPTION: RDG2

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values			
Segment	Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
Length = 13.0 ft		2	0.290	0.204	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,485.0	0.91	34.7	170.0
+D+L+H, LL Comb Run (LL)						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0
Length = 13.0 ft		1	0.290	0.204	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,485.0	0.91	34.7	170.0
Length = 13.0 ft		2	0.290	0.204	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,485.0	0.91	34.7	170.0
+D+Lr+H, LL Comb Run (*L)						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0
Length = 13.0 ft		1	0.399	0.371	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	4.55	740.1	1,856.3	2.07	78.9	212.5
Length = 13.0 ft		2	0.399	0.371	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	4.55	740.1	1,856.3	2.07	78.9	212.5
+D+Lr+H, LL Comb Run (L*)						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0
Length = 13.0 ft		1	0.399	0.371	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	4.55	740.1	1,856.3	2.07	78.9	212.5
Length = 13.0 ft		2	0.399	0.371	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	4.55	740.1	1,856.3	1.06	78.9	212.5
+D+Lr+H, LL Comb Run (LL)						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0
Length = 13.0 ft		1	0.565	0.397	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	6.45	1,049.1	1,856.3	2.22	84.4	212.5
Length = 13.0 ft		2	0.565	0.397	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	6.45	1,049.1	1,856.3	2.22	84.4	212.5
+D+0.70S+H						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0
Length = 13.0 ft		1	0.252	0.177	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,707.8	0.91	34.7	195.5
Length = 13.0 ft		2	0.252	0.177	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,707.8	0.91	34.7	195.5
+D+0.750Lr+0.750L+H, LL Cc						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0
Length = 13.0 ft		1	0.357	0.319	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	4.08	662.8	1,856.3	1.78	67.8	212.5
Length = 13.0 ft		2	0.357	0.319	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	4.08	662.8	1,856.3	1.78	67.8	212.5
+D+0.750Lr+0.750L+H, LL Cc						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0
Length = 13.0 ft		1	0.357	0.319	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	4.08	662.8	1,856.3	1.78	67.8	212.5
Length = 13.0 ft		2	0.357	0.319	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	4.08	662.8	1,856.3	1.02	67.8	212.5
+D+0.750Lr+0.750L+H, LL Cc						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0
Length = 13.0 ft		1	0.482	0.339	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	5.50	894.6	1,856.3	1.89	72.0	212.5
Length = 13.0 ft		2	0.482	0.339	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	5.50	894.6	1,856.3	1.89	72.0	212.5
+D+0.750L+0.5250S+H, LL C						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0
Length = 13.0 ft		1	0.252	0.177	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,707.8	0.91	34.7	195.5
Length = 13.0 ft		2	0.252	0.177	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,707.8	0.91	34.7	195.5
+D+0.750L+0.5250S+H, LL C						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0
Length = 13.0 ft		1	0.252	0.177	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,707.8	0.91	34.7	195.5
Length = 13.0 ft		2	0.252	0.177	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,707.8	0.91	34.7	195.5
+D+0.750L+0.5250S+H, LL C						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0
Length = 13.0 ft		1	0.252	0.177	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,707.8	0.91	34.7	195.5
Length = 13.0 ft		2	0.252	0.177	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	1,707.8	0.91	34.7	195.5
+D+0.60W+H						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0
Length = 13.0 ft		1	0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
Length = 13.0 ft		2	0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
+D+0.750Lr+0.750L+0.450W+						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0
Length = 13.0 ft		1	0.279	0.249	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	4.08	662.8	2,376.0	1.78	67.8	272.0
Length = 13.0 ft		2	0.279	0.249	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	4.08	662.8	2,376.0	1.78	67.8	272.0
+D+0.750Lr+0.750L+0.450W+						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0
Length = 13.0 ft		1	0.279	0.249	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	4.08	662.8	2,376.0	1.78	67.8	272.0
Length = 13.0 ft		2	0.279	0.249	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	4.08	662.8	2,376.0	1.02	67.8	272.0
+D+0.750Lr+0.750L+0.450W+						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0
Length = 13.0 ft		1	0.377	0.265	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	5.50	894.6	2,376.0	1.89	72.0	272.0
Length = 13.0 ft		2	0.377	0.265	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	5.50	894.6	2,376.0	1.89	72.0	272.0
+D+0.750L+0.5250S+0.450W						1.00	1.00	1.00	1.100	1.00	1.00	1.00		0.0	0.00	0.0	0.0	0.0

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 : CA 92648
 Phone : (949) 337-7075

Project Title: 85-25 - E 20th St
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

(c) ENERCALC INC 1983-2023

DESCRIPTION: RDG2

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F ^b	V	fv	F ^v
Length = 13.0 ft	1		0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
Length = 13.0 ft	2		0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
+D+0.750L+0.5250S+0.450W						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.0 ft	1		0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
Length = 13.0 ft	2		0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
+D+0.750L+0.5250S+0.450W						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.0 ft	1		0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
Length = 13.0 ft	2		0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
+0.60D+0.60W+0.60H						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.0 ft	1		0.109	0.077	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	258.6	2,376.0	0.55	20.8	272.0
Length = 13.0 ft	2		0.109	0.077	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	258.6	2,376.0	0.55	20.8	272.0
+D+0.70E+H						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.0 ft	1		0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
Length = 13.0 ft	2		0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
+D+0.750L+0.10S+0.5250E+I						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.0 ft	1		0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
Length = 13.0 ft	2		0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
+D+0.750L+0.10S+0.5250E+I						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.0 ft	1		0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
Length = 13.0 ft	2		0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
+D+0.750L+0.10S+0.5250E+I						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.0 ft	1		0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
Length = 13.0 ft	2		0.181	0.128	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.65	431.0	2,376.0	0.91	34.7	272.0
+0.60D+0.70E+0.60H						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.0 ft	1		0.109	0.077	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	258.6	2,376.0	0.55	20.8	272.0
Length = 13.0 ft	2		0.109	0.077	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	258.6	2,376.0	0.55	20.8	272.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr+H, LL Comb Run (L*)	1	0.1740	5.955		0.0000	0.000
+D+Lr+H, LL Comb Run (*L)	2	0.1731	7.117		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	1.636	4.965	1.636
Max Upward from Load Combinations	1.636	4.965	1.636
Max Upward from Load Cases	1.024	2.925	1.024
Max Downward from all Load Conditions	-0.146		-0.146
Max Downward from Load Cases (Resis)	-0.146		-0.146
+D+H	0.612	2.040	0.612
+D+L+H, LL Comb Run (L*)	0.612	2.040	0.612
+D+L+H, LL Comb Run (L*)	0.612	2.040	0.612
+D+L+H, LL Comb Run (LL)	0.612	2.040	0.612
+D+Lr+H, LL Comb Run (L*)	0.466	3.502	1.636
+D+Lr+H, LL Comb Run (L*)	1.636	3.502	0.466
+D+Lr+H, LL Comb Run (LL)	1.489	4.965	1.489
+D+0.70S+H	0.612	2.040	0.612
+D+0.750Lr+0.750L+H, LL Comb Run (*)	0.502	3.137	1.380
+D+0.750Lr+0.750L+H, LL Comb Run (L)	1.380	3.137	0.502
+D+0.750Lr+0.750L+H, LL Comb Run (L)	1.270	4.234	1.270
+D+0.750L+0.5250S+H, LL Comb Run (*)	0.612	2.040	0.612
+D+0.750L+0.5250S+H, LL Comb Run (L)	0.612	2.040	0.612
+D+0.750L+0.5250S+H, LL Comb Run (L)	0.612	2.040	0.612

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 Phone : (949) 337-7075

Project Title: 85-25 - E 20th St
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

(c) ENERCALC INC 1983-2023

DESCRIPTION: RDG2

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
+D+0.60W+H	0.612	2.040	0.612
+D+0.750Lr+0.750L+0.450W+H, LL Comb	0.502	3.137	1.380
+D+0.750Lr+0.750L+0.450W+H, LL Comb	1.380	3.137	0.502
+D+0.750Lr+0.750L+0.450W+H, LL Comb	1.270	4.234	1.270
+D+0.750L+0.5250S+0.450W+H, LL Comb	0.612	2.040	0.612
+D+0.750L+0.5250S+0.450W+H, LL Comb	0.612	2.040	0.612
+D+0.750L+0.5250S+0.450W+H, LL Comb	0.612	2.040	0.612
+0.60D+0.60W+0.60H	0.367	1.224	0.367
+D+0.70E+H	0.612	2.040	0.612
+D+0.750L+0.10S+0.5250E+H, LL Comb	0.612	2.040	0.612
+D+0.750L+0.10S+0.5250E+H, LL Comb	0.612	2.040	0.612
+D+0.750L+0.10S+0.5250E+H, LL Comb	0.612	2.040	0.612
+0.60D+0.70E+0.60H	0.367	1.224	0.367
D Only	0.612	2.040	0.612
Lr Only, LL Comb Run (*L)	-0.146	1.462	1.024
Lr Only, LL Comb Run (L*)	1.024	1.462	-0.146
Lr Only, LL Comb Run (LL)	0.878	2.925	0.878
H Only			

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

(c) ENERCALC, LLC 1982-2025

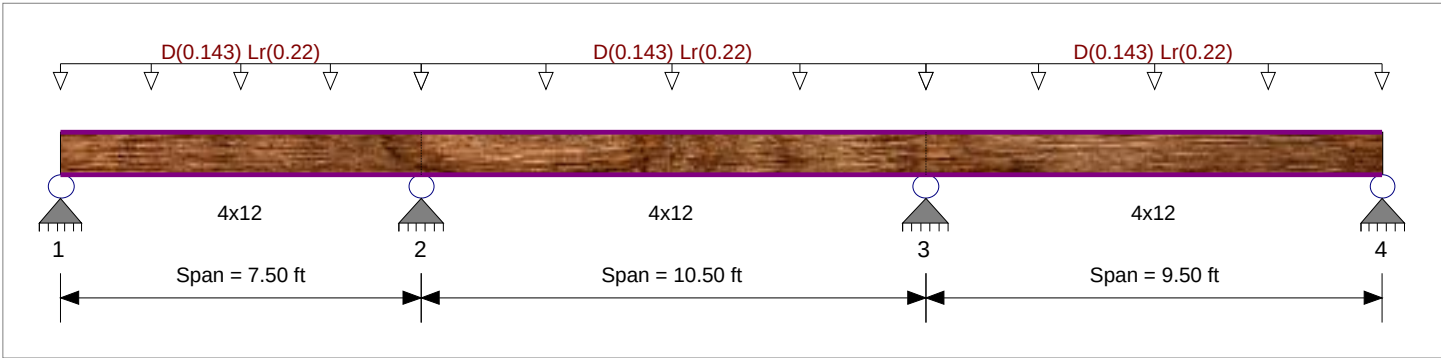
DESCRIPTION: RDG3

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019
Load Combination Set : ASCE 7-22 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design	Fb + 1,350.0 psi	E : Modulus of Elasticity
Load Combination : ASCE 7-22 / IBC 2024	Fb - 1,350.0 psi	Ebend- xx 1,600.0ksi
	Fc - Prll 925.0 psi	Eminbend - xx 580.0ksi
Wood Species : Douglas Fir-Larch	Fc - Perp 625.0 psi	
Wood Grade : No.1	Fv 170.0 psi	
	Ft 675.0 psi	Density 31.210pcf
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling		



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.0130, Lr = 0.020 ksf, Tributary Width = 11.0 ft, (RR)
Load for Span Number 2
Uniform Load : D = 0.0130, Lr = 0.020 ksf, Tributary Width = 11.0 ft, (RR)
Load for Span Number 3
Uniform Load : D = 0.0130, Lr = 0.020 ksf, Tributary Width = 11.0 ft, (RR)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.358 1	Maximum Shear Stress Ratio	=	0.335 : 1
Section used for this span		4x12	Section used for this span		4x12
fb: Actual	=	665.07 psi	fv: Actual	=	71.21 psi
F'b	=	1,856.25 psi	F'v	=	212.50 psi
Load Combination		+D+Lr+H, LL Comb Run (*LL)	Load Combination		+D+Lr+H, LL Comb Run (*LL)
Location of maximum on span	=	10.500ft	Location of maximum on span	=	10.500 ft
Span # where maximum occurs	=	Span # 2	Span # where maximum occurs	=	Span # 2
Maximum Deflection					
Max Downward Transient Deflection	0.045 in	Ratio = 2520 >=360	Span: 3 : Lr Only, LL Comb Run (L*L)		
Max Upward Transient Deflection	-0.026 in	Ratio = 4849 >=360	Span: 2 : Lr Only, LL Comb Run (L*L)		
Max Downward Total Deflection	0.064 in	Ratio = 1779 >=240	Span: 3 : +D+Lr+H, LL Comb Run (L*L)		
Max Upward Total Deflection	-0.015 in	Ratio = 8231 >=240	Span: 2 : +D+Lr+H, LL Comb Run (L*L)		

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
+D+H														0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.146	0.155	0.90	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	1,336.5	0.62	23.8	153.0
Length = 10.50 ft	2	0.194	0.188	0.90	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,336.5	0.75	28.7	153.0
Length = 9.50 ft	3	0.194	0.188	0.90	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,336.5	0.75	28.7	153.0

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: RDG3

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
+D+L+H, LL Comb Run (**L)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.131	0.140	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	1,485.0	0.62	23.8	170.0
Length = 10.50 ft	2	0.174	0.169	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,485.0	0.75	28.7	170.0
Length = 9.50 ft	3	0.174	0.169	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,485.0	0.75	28.7	170.0
+D+L+H, LL Comb Run (*L*)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.131	0.140	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	1,485.0	0.62	23.8	170.0
Length = 10.50 ft	2	0.174	0.169	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,485.0	0.75	28.7	170.0
Length = 9.50 ft	3	0.174	0.169	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,485.0	0.75	28.7	170.0
+D+L+H, LL Comb Run (*LL)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.131	0.140	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	1,485.0	0.62	23.8	170.0
Length = 10.50 ft	2	0.174	0.169	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,485.0	0.75	28.7	170.0
Length = 9.50 ft	3	0.174	0.169	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,485.0	0.75	28.7	170.0
+D+L+H, LL Comb Run (L**)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.131	0.140	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	1,485.0	0.62	23.8	170.0
Length = 10.50 ft	2	0.174	0.169	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,485.0	0.75	28.7	170.0
Length = 9.50 ft	3	0.174	0.169	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,485.0	0.75	28.7	170.0
+D+L+H, LL Comb Run (L*L)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.131	0.140	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	1,485.0	0.62	23.8	170.0
Length = 10.50 ft	2	0.174	0.169	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,485.0	0.75	28.7	170.0
Length = 9.50 ft	3	0.174	0.169	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,485.0	0.75	28.7	170.0
+D+L+H, LL Comb Run (LL*)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.131	0.140	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	1,485.0	0.62	23.8	170.0
Length = 10.50 ft	2	0.174	0.169	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,485.0	0.75	28.7	170.0
Length = 9.50 ft	3	0.174	0.169	1.00	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,485.0	0.75	28.7	170.0
+D+Lr+H, LL Comb Run (**L)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.072	0.098	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	0.82	134.1	1,856.3	0.54	20.7	212.5
Length = 10.50 ft	2	0.251	0.312	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.87	466.6	1,856.3	1.74	66.3	212.5
Length = 9.50 ft	3	0.252	0.312	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.88	467.9	1,856.3	1.74	66.3	212.5
+D+Lr+H, LL Comb Run (*L*)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.229	0.287	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.61	424.2	1,856.3	1.60	61.1	212.5
Length = 10.50 ft	2	0.246	0.294	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.82	457.6	1,856.3	1.64	62.6	212.5
Length = 9.50 ft	3	0.246	0.158	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.82	457.6	1,856.3	0.88	33.6	212.5
+D+Lr+H, LL Comb Run (*LL)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.196	0.259	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.24	363.7	1,856.3	1.45	55.1	212.5
Length = 10.50 ft	2	0.358	0.335	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	4.09	665.1	1,856.3	1.87	71.2	212.5
Length = 9.50 ft	3	0.358	0.335	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	4.09	665.1	1,856.3	1.87	71.2	212.5
+D+Lr+H, LL Comb Run (L**)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.166	0.236	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.90	308.0	1,856.3	1.32	50.2	212.5
Length = 10.50 ft	2	0.166	0.132	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.90	308.0	1,856.3	0.74	28.0	212.5
Length = 9.50 ft	3	0.124	0.132	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.41	229.3	1,856.3	0.74	28.0	212.5
+D+Lr+H, LL Comb Run (L*L)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.167	0.227	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.91	309.9	1,856.3	1.27	48.3	212.5
Length = 10.50 ft	2	0.235	0.309	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.69	436.8	1,856.3	1.72	65.6	212.5

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: RDG3

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values			
Segment	Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
Length = 9.50 ft		3	0.259	0.309	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.96	480.4	1,856.3	1.72	65.6	212.5
+D+Lr+H, LL Comb Run (LL*)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft		1	0.290	0.302	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.31	537.6	1,856.3	1.69	64.3	212.5
Length = 10.50 ft		2	0.290	0.302	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.31	537.6	1,856.3	1.69	64.3	212.5
Length = 9.50 ft		3	0.230	0.155	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.63	427.8	1,856.3	0.86	32.9	212.5
+D+Lr+H, LL Comb Run (LLL)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft		1	0.257	0.274	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.94	477.1	1,856.3	1.53	58.3	212.5
Length = 10.50 ft		2	0.342	0.332	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.91	635.3	1,856.3	1.85	70.5	212.5
Length = 9.50 ft		3	0.342	0.332	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.91	635.3	1,856.3	1.85	70.5	212.5
+D+0.70S+H						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft		1	0.114	0.122	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	1,707.8	0.62	23.8	195.5
Length = 10.50 ft		2	0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
Length = 9.50 ft		3	0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
+D+0.750Lr+0.750L+H, LL Comb Run (**L)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft		1	0.080	0.100	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	0.92	149.2	1,856.3	0.56	21.2	212.5
Length = 10.50 ft		2	0.223	0.268	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.55	414.7	1,856.3	1.49	56.9	212.5
Length = 9.50 ft		3	0.223	0.268	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.55	414.7	1,856.3	1.49	56.9	212.5
+D+0.750Lr+0.750L+H, LL Comb Run (*L*)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft		1	0.198	0.244	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.26	366.8	1,856.3	1.36	51.7	212.5
Length = 10.50 ft		2	0.220	0.252	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.51	407.9	1,856.3	1.41	53.6	212.5
Length = 9.50 ft		3	0.220	0.153	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.51	407.9	1,856.3	0.85	32.4	212.5
+D+0.750Lr+0.750L+H, LL Comb Run (*LL)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft		1	0.173	0.222	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.98	321.4	1,856.3	1.24	47.3	212.5
Length = 10.50 ft		2	0.304	0.285	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.47	563.6	1,856.3	1.59	60.6	212.5
Length = 9.50 ft		3	0.304	0.285	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.47	563.6	1,856.3	1.59	60.6	212.5
+D+0.750Lr+0.750L+H, LL Comb Run (L**)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft		1	0.151	0.204	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.72	279.7	1,856.3	1.14	43.3	212.5
Length = 10.50 ft		2	0.151	0.133	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.72	279.7	1,856.3	0.74	28.2	212.5
Length = 9.50 ft		3	0.128	0.133	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.46	236.8	1,856.3	0.74	28.2	212.5
+D+0.750Lr+0.750L+H, LL Comb Run (L*L)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft		1	0.137	0.197	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.56	254.1	1,856.3	1.10	41.9	212.5
Length = 10.50 ft		2	0.211	0.265	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.41	392.4	1,856.3	1.48	56.4	212.5
Length = 9.50 ft		3	0.216	0.265	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.47	400.8	1,856.3	1.48	56.4	212.5
+D+0.750Lr+0.750L+H, LL Comb Run (LL*)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft		1	0.243	0.255	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.78	451.9	1,856.3	1.42	54.1	212.5
Length = 10.50 ft		2	0.243	0.255	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.78	451.9	1,856.3	1.42	54.1	212.5
Length = 9.50 ft		3	0.208	0.150	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.37	385.6	1,856.3	0.84	31.9	212.5
+D+0.750Lr+0.750L+H, LL Comb Run (LLL)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft		1	0.219	0.234	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.50	406.5	1,856.3	1.30	49.7	212.5
Length = 10.50 ft		2	0.292	0.283	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.33	541.2	1,856.3	1.58	60.0	212.5
Length = 9.50 ft		3	0.292	0.283	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.33	541.2	1,856.3	1.58	60.0	212.5
+D+0.750L+0.5250S+H, LL Comb Run (**L)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft		1	0.114	0.122	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	1,707.8	0.62	23.8	195.5
Length = 10.50 ft		2	0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
Length = 9.50 ft		3	0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
+D+0.750L+0.5250S+H, LL Comb Run (*L*)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft		1	0.114	0.122	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	1,707.8	0.62	23.8	195.5

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: RDG3

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
Length = 10.50 ft	2		0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
Length = 9.50 ft	3		0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
+D+0.750L+0.5250S+H, LL Comb Run (*LL)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.114	0.122	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	1,707.8	0.62	23.8	195.5
Length = 10.50 ft	2		0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
Length = 9.50 ft	3		0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
+D+0.750L+0.5250S+H, LL Comb Run (L**)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.114	0.122	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	1,707.8	0.62	23.8	195.5
Length = 10.50 ft	2		0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
Length = 9.50 ft	3		0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
+D+0.750L+0.5250S+H, LL Comb Run (L*L)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.114	0.122	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	1,707.8	0.62	23.8	195.5
Length = 10.50 ft	2		0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
Length = 9.50 ft	3		0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
+D+0.750L+0.5250S+H, LL Comb Run (LL*)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.114	0.122	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	1,707.8	0.62	23.8	195.5
Length = 10.50 ft	2		0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
Length = 9.50 ft	3		0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
+D+0.750L+0.5250S+H, LL Comb Run (LLL)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.114	0.122	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	1,707.8	0.62	23.8	195.5
Length = 10.50 ft	2		0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
Length = 9.50 ft	3		0.152	0.147	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	1,707.8	0.75	28.7	195.5
+D+0.60W+H						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (**L)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.063	0.078	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	0.92	149.2	2,376.0	0.56	21.2	272.0
Length = 10.50 ft	2		0.175	0.209	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.55	414.7	2,376.0	1.49	56.9	272.0
Length = 9.50 ft	3		0.175	0.209	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.55	414.7	2,376.0	1.49	56.9	272.0
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (*L*)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.154	0.190	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.26	366.8	2,376.0	1.36	51.7	272.0
Length = 10.50 ft	2		0.172	0.197	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.51	407.9	2,376.0	1.41	53.6	272.0
Length = 9.50 ft	3		0.172	0.119	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.51	407.9	2,376.0	0.85	32.4	272.0
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (*LL)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.135	0.174	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.98	321.4	2,376.0	1.24	47.3	272.0
Length = 10.50 ft	2		0.237	0.223	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.47	563.6	2,376.0	1.59	60.6	272.0
Length = 9.50 ft	3		0.237	0.223	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.47	563.6	2,376.0	1.59	60.6	272.0
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (L**)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.118	0.159	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.72	279.7	2,376.0	1.14	43.3	272.0
Length = 10.50 ft	2		0.118	0.104	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.72	279.7	2,376.0	0.74	28.2	272.0
Length = 9.50 ft	3		0.100	0.104	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.46	236.8	2,376.0	0.74	28.2	272.0
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (L*L)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.107	0.154	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.56	254.1	2,376.0	1.10	41.9	272.0
Length = 10.50 ft	2		0.165	0.207	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.41	392.4	2,376.0	1.48	56.4	272.0
Length = 9.50 ft	3		0.169	0.207	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.47	400.8	2,376.0	1.48	56.4	272.0
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (LL*)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

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DESCRIPTION: RDG3

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios									Moment Values			Shear Values			
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
Length = 7.50 ft	1		0.190	0.199	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.78	451.9	2,376.0	1.42	54.1	272.0
Length = 10.50 ft	2		0.190	0.199	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.78	451.9	2,376.0	1.42	54.1	272.0
Length = 9.50 ft	3		0.162	0.117	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.37	385.6	2,376.0	0.84	31.9	272.0
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (LLL)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.171	0.183	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.50	406.5	2,376.0	1.30	49.7	272.0
Length = 10.50 ft	2		0.228	0.221	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.33	541.2	2,376.0	1.58	60.0	272.0
Length = 9.50 ft	3		0.228	0.221	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.33	541.2	2,376.0	1.58	60.0	272.0
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (**L)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (*L*)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (*LL)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (L**)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (L*L)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (LL*)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (LLL)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
+0.60D+0.60W+0.60H						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.049	0.052	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	0.72	116.8	2,376.0	0.37	14.3	272.0
Length = 10.50 ft	2		0.065	0.063	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	0.96	155.5	2,376.0	0.45	17.2	272.0
Length = 9.50 ft	3		0.065	0.063	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	0.96	155.5	2,376.0	0.45	17.2	272.0
+D+0.70E+H						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (**L)						1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1		0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3		0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: RDG3

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (*L*)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2	0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3	0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (*LL)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2	0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3	0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (L**)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2	0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3	0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (L*L)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2	0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3	0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (LL*)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2	0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3	0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (LLL)					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.082	0.087	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.20	194.6	2,376.0	0.62	23.8	272.0
Length = 10.50 ft	2	0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
Length = 9.50 ft	3	0.109	0.106	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.59	259.1	2,376.0	0.75	28.7	272.0
+0.60D+0.70E+0.60H					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.50 ft	1	0.049	0.052	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	0.72	116.8	2,376.0	0.37	14.3	272.0
Length = 10.50 ft	2	0.065	0.063	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	0.96	155.5	2,376.0	0.45	17.2	272.0
Length = 9.50 ft	3	0.065	0.063	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	0.96	155.5	2,376.0	0.45	17.2	272.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr+H, LL Comb Run (L*L)	0.0264	3.529	+D+Lr+H, LL Comb Run (*L*)	-0.0036	7.122
2	+D+Lr+H, LL Comb Run (*L*)	0.0562	5.294	+D+Lr+H, LL Comb Run (L*L)	-0.0047	10.147
3	+D+Lr+H, LL Comb Run (L*L)	0.0641	5.189		0.0000	10.147

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions	1.190	3.849	4.323	1.482
Max Upward from Load Combinations	1.190	3.849	4.323	1.482
Max Upward from Load Cases	0.782	2.364	2.602	0.930
Max Downward from all Load Conditions (Resisting Uplift)	-0.188	-0.207	-0.103	-0.129
Max Downward from Load Cases (Resisting Uplift)	-0.188	-0.207	-0.103	-0.129
+D+H	0.409	1.486	1.721	0.552
+D+L+H, LL Comb Run (**L)	0.409	1.486	1.721	0.552
+D+L+H, LL Comb Run (*L*)	0.409	1.486	1.721	0.552
+D+L+H, LL Comb Run (*LL)	0.409	1.486	1.721	0.552
+D+L+H, LL Comb Run (L**)	0.409	1.486	1.721	0.552
+D+L+H, LL Comb Run (L*L)	0.409	1.486	1.721	0.552
+D+L+H, LL Comb Run (LL*)	0.409	1.486	1.721	0.552
+D+L+H, LL Comb Run (LLL)	0.409	1.486	1.721	0.552
+D+Lr+H, LL Comb Run (**L)	0.458	1.279	3.057	1.463
+D+Lr+H, LL Comb Run (*L*)	0.220	2.847	2.986	0.423

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Project Title: 85-25 - E 20th St
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: RDG3

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
+D+Lr+H, LL Comb Run (*LL)	0.270	2.641	4.323	1.334
+D+Lr+H, LL Comb Run (L**)	1.141	2.488	1.618	0.571
+D+Lr+H, LL Comb Run (L*L)	1.190	2.281	2.954	1.482
+D+Lr+H, LL Comb Run (LL*)	0.952	3.849	2.883	0.443
+D+Lr+H, LL Comb Run (LLL)	1.002	3.642	4.219	1.353
+D+0.70S+H	0.409	1.486	1.721	0.552
+D+0.750Lr+0.750L+H, LL Comb Run (**L)	0.446	1.331	2.723	1.235
+D+0.750Lr+0.750L+H, LL Comb Run (*L*)	0.267	2.507	2.670	0.456
+D+0.750Lr+0.750L+H, LL Comb Run (*LL)	0.305	2.352	3.672	1.139
+D+0.750Lr+0.750L+H, LL Comb Run (L**)	0.958	2.237	1.644	0.566
+D+0.750Lr+0.750L+H, LL Comb Run (L*L)	0.995	2.082	2.646	1.249
+D+0.750Lr+0.750L+H, LL Comb Run (LL*)	0.816	3.258	2.592	0.470
+D+0.750Lr+0.750L+H, LL Comb Run (LLL)	0.854	3.103	3.595	1.153
+D+0.750L+0.5250S+H, LL Comb Run (**L)	0.409	1.486	1.721	0.552
+D+0.750L+0.5250S+H, LL Comb Run (*L*)	0.409	1.486	1.721	0.552
+D+0.750L+0.5250S+H, LL Comb Run (*LL)	0.409	1.486	1.721	0.552
+D+0.750L+0.5250S+H, LL Comb Run (L**)	0.409	1.486	1.721	0.552
+D+0.750L+0.5250S+H, LL Comb Run (L*L)	0.409	1.486	1.721	0.552
+D+0.750L+0.5250S+H, LL Comb Run (LL*)	0.409	1.486	1.721	0.552
+D+0.750L+0.5250S+H, LL Comb Run (LLL)	0.409	1.486	1.721	0.552
+D+0.60W+H	0.409	1.486	1.721	0.552
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (**L)	0.446	1.331	2.723	1.235
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (*L*)	0.267	2.507	2.670	0.456
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (*LL)	0.305	2.352	3.672	1.139
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (L**)	0.958	2.237	1.644	0.566
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (L*L)	0.995	2.082	2.646	1.249
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (LL*)	0.816	3.258	2.592	0.470
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (LLL)	0.854	3.103	3.595	1.153
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (**L)	0.409	1.486	1.721	0.552
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (*L*)	0.409	1.486	1.721	0.552
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (*LL)	0.409	1.486	1.721	0.552
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (L**)	0.409	1.486	1.721	0.552
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (L*L)	0.409	1.486	1.721	0.552
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (LL*)	0.409	1.486	1.721	0.552
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (LLL)	0.409	1.486	1.721	0.552
+0.60D+0.60W+0.60H	0.245	0.891	1.033	0.331
+D+0.70E+H	0.409	1.486	1.721	0.552
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (**L)	0.409	1.486	1.721	0.552
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (*L*)	0.409	1.486	1.721	0.552
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (*LL)	0.409	1.486	1.721	0.552
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (L**)	0.409	1.486	1.721	0.552
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (L*L)	0.409	1.486	1.721	0.552
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (LL*)	0.409	1.486	1.721	0.552
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (LLL)	0.409	1.486	1.721	0.552
+0.60D+0.70E+0.60H	0.245	0.891	1.033	0.331
D Only	0.409	1.486	1.721	0.552
Lr Only, LL Comb Run (**L)	0.050	-0.207	1.336	0.911
Lr Only, LL Comb Run (*L*)	-0.188	1.362	1.265	-0.129
Lr Only, LL Comb Run (*LL)	-0.139	1.155	2.602	0.782
Lr Only, LL Comb Run (L**)	0.732	1.002	-0.103	0.019
Lr Only, LL Comb Run (L*L)	0.782	0.795	1.233	0.930
Lr Only, LL Comb Run (LL*)	0.544	2.364	1.162	-0.109
Lr Only, LL Comb Run (LLL)	0.593	2.157	2.499	0.801
H Only				

Wood Beam

Project File: 85-25_Storage.ecb

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

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DESCRIPTION: RDG4

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019

Load Combination Set : ASCE 7-22 / IBC 2024

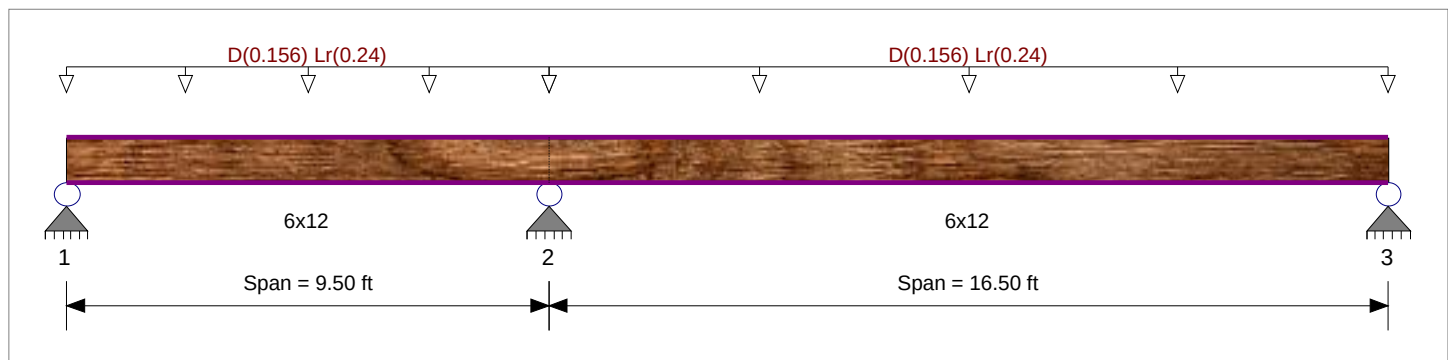
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-22 / IBC 2024

Wood Species : Douglas Fir-Larch
Wood Grade : No.1

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

Fb + 1,350.0 psi E : Modulus of Elasticity
Fb - 1,350.0 psi Ebend- xx 1,600.0ksi
Fc - Prll 925.0 psi Eminbend - xx 580.0ksi
Fc - Perp 625.0 psi
Fv 170.0 psi
Ft 675.0 psi Density 31.210pcf



Applied Loads

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

Beam self weight calculated and added to loading

Load for Span Number 1

Uniform Load : D = 0.0130, Lr = 0.020 ksf, Tributary Width = 12.0 ft, (RR)

Load for Span Number 2

Uniform Load : D = 0.0130, Lr = 0.020 ksf, Tributary Width = 12.0 ft, (RR)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.618	1	Maximum Shear Stress Ratio	=	0.406	1
Section used for this span		6x12		Section used for this span		6x12	
fb: Actual	=	1,043.04psi		fv: Actual	=	86.35 psi	
F'b	=	1,687.50psi		F'v	=	212.50 psi	
Load Combination		+D+Lr+H, LL Comb Run (LL)		Load Combination		+D+Lr+H, LL Comb Run (LL)	
Location of maximum on span	=	9.500ft		Location of maximum on span	=	9.500 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.225 in	Ratio =	879 >=360	Span: 2 : Lr Only, LL Comb Run (*L)			
Max Upward Transient Deflection	-0.047 in	Ratio =	2434 >=360	Span: 1 : Lr Only, LL Comb Run (*L)			
Max Downward Total Deflection	0.366 in	Ratio =	540 >=240	Span: 2 : +D+Lr+H, LL Comb Run (*L)			
Max Upward Total Deflection	-0.059 in	Ratio =	1921 >=240	Span: 1 : +D+Lr+H, LL Comb Run (*L)			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
+D+H																	
Length = 9.50 ft	1	0.356	0.234	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,215.0	1.51	35.8	153.0
Length = 16.50 ft	2	0.356	0.234	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,215.0	1.51	35.8	153.0
+D+L+H, LL Comb Run (*L)																	
Length = 9.50 ft	1	0.320	0.210	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,350.0	1.51	35.8	170.0
Length = 16.50 ft	2	0.320	0.210	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,350.0	1.51	35.8	170.0

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: RDG4

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
+D+L+H, LL Comb Run (L*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.320	0.210	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,350.0	1.51	35.8	170.0
Length = 16.50 ft	2	0.320	0.210	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,350.0	1.51	35.8	170.0
+D+L+H, LL Comb Run (LL)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.320	0.210	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,350.0	1.51	35.8	170.0
Length = 16.50 ft	2	0.320	0.210	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,350.0	1.51	35.8	170.0
+D+Lr+H, LL Comb Run (L*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.560	0.400	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	9.55	945.1	1,687.5	3.58	84.9	212.5
Length = 16.50 ft	2	0.562	0.400	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	9.58	948.0	1,687.5	3.58	84.9	212.5
+D+Lr+H, LL Comb Run (LL)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.618	0.406	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	10.54	1,043.0	1,687.5	3.64	86.3	212.5
Length = 16.50 ft	2	0.618	0.406	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	10.54	1,043.0	1,687.5	3.64	86.3	212.5
+D+0.70S+H					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.278	0.183	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,552.5	1.51	35.8	195.5
Length = 16.50 ft	2	0.278	0.183	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,552.5	1.51	35.8	195.5
+D+0.750Lr+0.750L+H, LL Comb Run (L*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.484	0.342	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.25	816.8	1,687.5	3.06	72.6	212.5
Length = 16.50 ft	2	0.484	0.342	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.25	816.8	1,687.5	3.06	72.6	212.5
+D+0.750Lr+0.750L+H, LL Comb Run (L*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.300	0.208	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.11	505.5	1,687.5	1.86	44.2	212.5
Length = 16.50 ft	2	0.300	0.173	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.11	505.5	1,687.5	1.55	36.8	212.5
+D+0.750Lr+0.750L+H, LL Comb Run (LL)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.528	0.347	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.99	890.3	1,687.5	3.11	73.7	212.5
Length = 16.50 ft	2	0.528	0.347	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.99	890.3	1,687.5	3.11	73.7	212.5
+D+0.750L+0.5250S+H, LL Comb Run (L*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.278	0.183	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,552.5	1.51	35.8	195.5
Length = 16.50 ft	2	0.278	0.183	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,552.5	1.51	35.8	195.5
+D+0.750L+0.5250S+H, LL Comb Run (L*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.278	0.183	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,552.5	1.51	35.8	195.5
Length = 16.50 ft	2	0.278	0.183	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,552.5	1.51	35.8	195.5
+D+0.750L+0.5250S+H, LL Comb Run (LL)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.278	0.183	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,552.5	1.51	35.8	195.5
Length = 16.50 ft	2	0.278	0.183	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	1,552.5	1.51	35.8	195.5
+D+0.60W+H					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
Length = 16.50 ft	2	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (L*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.378	0.267	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.25	816.8	2,160.0	3.06	72.6	272.0
Length = 16.50 ft	2	0.378	0.267	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.25	816.8	2,160.0	3.06	72.6	272.0
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (L*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.234	0.163	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.11	505.5	2,160.0	1.86	44.2	272.0
Length = 16.50 ft	2	0.234	0.135	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.11	505.5	2,160.0	1.55	36.8	272.0
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (LL)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.412	0.271	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.99	890.3	2,160.0	3.11	73.7	272.0

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: RDG4

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
Length = 16.50 ft	2	0.412	0.271	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.99	890.3	2,160.0	3.11	73.7	272.0
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (*L)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
Length = 16.50 ft	2	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (L*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
Length = 16.50 ft	2	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (LL)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
Length = 16.50 ft	2	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
+0.60D+0.60W+0.60H					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.120	0.079	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.62	259.2	2,160.0	0.90	21.5	272.0
Length = 16.50 ft	2	0.120	0.079	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.62	259.2	2,160.0	0.90	21.5	272.0
+D+0.70E+H					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
Length = 16.50 ft	2	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (*L)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
Length = 16.50 ft	2	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (L*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
Length = 16.50 ft	2	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (LL)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
Length = 16.50 ft	2	0.200	0.131	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	432.0	2,160.0	1.51	35.8	272.0
+0.60D+0.70E+0.60H					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.50 ft	1	0.120	0.079	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.62	259.2	2,160.0	0.90	21.5	272.0
Length = 16.50 ft	2	0.120	0.079	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.62	259.2	2,160.0	0.90	21.5	272.0

Overall Maximum Deflections

Span	Load Combination	Max. "v" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1		0.0000	0.000	+D+Lr+H, LL Comb Run (*L)	-0.0593	5.891
2	+D+Lr+H, LL Comb Run (*L)	0.3662	9.034		0.0000	5.891

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	1.383	7.074	2.801
Max Upward from Load Combinations	1.383	7.074	2.801
Max Upward from Load Cases	1.036	4.144	1.666
Max Downward from all Load Conditions (Resisting Uplift)	-0.546		-0.060
Max Downward from Load Combinations (Resisting Uplift)	-0.199		
Max Downward from Load Cases (Resisting Uplift)	-0.546		-0.060
+D+H	0.347	2.930	1.136
+D+L+H, LL Comb Run (*L)	0.347	2.930	1.136
+D+L+H, LL Comb Run (L*)	0.347	2.930	1.136
+D+L+H, LL Comb Run (LL)	0.347	2.930	1.136
+D+Lr+H, LL Comb Run (*L)	-0.199	5.770	2.801
+D+Lr+H, LL Comb Run (L*)	1.383	4.234	1.076
+D+Lr+H, LL Comb Run (LL)	0.837	7.074	2.741
+D+0.70S+H	0.347	2.930	1.136
+D+0.750Lr+0.750L+H, LL Comb Run (*L)	-0.063	5.060	2.385
+D+0.750Lr+0.750L+H, LL Comb Run (L*)	1.124	3.908	1.091

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Project Title: 85-25 - E 20th St
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.25.04.16

FA STRUCTURAL

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: RDG4

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
+D+0.750Lr+0.750L+H, LL Comb Run (LL)	0.714	6.038	2.340
+D+0.750L+0.5250S+H, LL Comb Run (*L)	0.347	2.930	1.136
+D+0.750L+0.5250S+H, LL Comb Run (L*)	0.347	2.930	1.136
+D+0.750L+0.5250S+H, LL Comb Run (LL)	0.347	2.930	1.136
+D+0.60W+H	0.347	2.930	1.136
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (*L)	-0.063	5.060	2.385
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (L*)	1.124	3.908	1.091
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run (LL)	0.714	6.038	2.340
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (*L)	0.347	2.930	1.136
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (L*)	0.347	2.930	1.136
+D+0.750L+0.5250S+0.450W+H, LL Comb Run (LL)	0.347	2.930	1.136
+0.60D+0.60W+0.60H	0.208	1.758	0.681
+D+0.70E+H	0.347	2.930	1.136
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (*L)	0.347	2.930	1.136
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (L*)	0.347	2.930	1.136
+D+0.750L+0.10S+0.5250E+H, LL Comb Run (LL)	0.347	2.930	1.136
+0.60D+0.70E+0.60H	0.208	1.758	0.681
D Only	0.347	2.930	1.136
Lr Only, LL Comb Run (*L)	-0.546	2.840	1.666
Lr Only, LL Comb Run (L*)	1.036	1.304	-0.060
Lr Only, LL Comb Run (LL)	0.490	4.144	1.606
H Only			

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

(c) ENERCALC INC 1983-2023

DESCRIPTION: CANT. FB10

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

Analysis Method : Allowable Stress Design

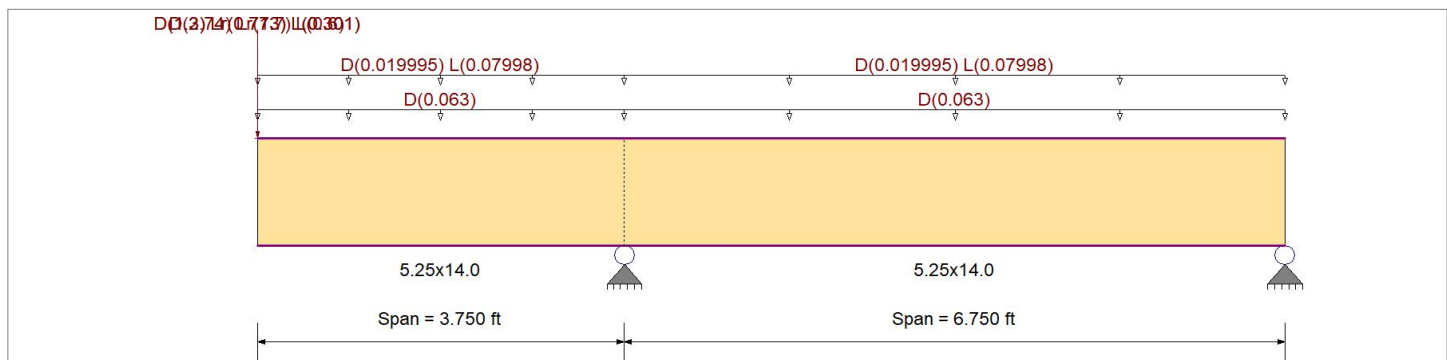
Load Combination : IBC 2021

Wood Species : iLevel Truss Joist

Wood Grade : Parallam PSL 2.2E

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

Fb +	2900 psi	E : Modulus of Elasticity	
Fb -	2900 psi	Ebend- xx	2200ksi
Fc - Prll	2900 psi	Eminbend - xx	1118.19ksi
Fc - Perp	750 psi		
Fv	290 psi		
Ft	2025 psi	Density	45.07 pcf



Applied Loads

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

Beam self weight calculated and added to loading

Load for Span Number 1

Uniform Load : D = 0.0180 ksf, Tributary Width = 3.50 ft, (WALL)

Uniform Load : D = 0.0150, L = 0.060 ksf, Tributary Width = 1.333 ft, (FJ)

Point Load : D = 1.30, Lr = 0.7730, L = 0.3010 k @ 0.0 ft, (FB6)

Point Load : D = 1.30, Lr = 0.7730, L = 0.3010 k @ 0.0 ft, (FB6)

Point Load : D = 2.740, Lr = 1.70, L = 0.60 k @ 0.0 ft, (FB9)

Load for Span Number 2

Uniform Load : D = 0.0180 ksf, Tributary Width = 3.50 ft, (WALL)

Uniform Load : D = 0.0150, L = 0.060 ksf, Tributary Width = 1.333 ft, (FJ)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.662	1	Maximum Shear Stress Ratio	=	0.513	1
Section used for this span	=	5.25x14.0		Section used for this span	=	5.25x14.0	
fb: Actual	=	2,358.16psi		fv: Actual	=	185.86 psi	
F'b	=	3,563.50psi		F'v	=	362.50 psi	
Load Combination +D+0.750Lr+0.750L+H, LL Comb Run (L*)				Load Combination +D+0.750Lr+0.750L+H, LL Comb Run (LL)			
Location of maximum on span	=	3.750ft		Location of maximum on span	=	2.598 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.105 in	Ratio =	858 >=360	Span: 2 : L Only, LL Comb Run (*L)			
Max Upward Transient Deflection	-0.024 in	Ratio =	3429 >=360	Span: 2 : Lr Only, LL Comb Run (L*)			
Max Downward Total Deflection	0.286 in	Ratio =	314 >=240	Span: 1 : +D+0.750Lr+0.750L+H, LL Comb Run (L*)			
Max Upward Total Deflection	-0.064 in	Ratio =	1274 >=240	Span: 2 : +D+0.750Lr+0.750L+H, LL Comb Run (L*)			

Maximum Forces & Stresses for Load Combinations

Maximum Forces & Stresses for Load Combinations																		
Load Combination		Max Stress Ratios										Moment Values			Shear Values			
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v	
+D+H														0.0	0.00	0.0	0.0	
Length = 3.750 ft	1	0.566	0.439	0.90	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	2,565.7	5.62	114.6	261.0	
Length = 6.750 ft	2	0.566	0.439	0.90	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	2,565.7	3.31	114.6	261.0	

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Project Title: 85-25 - E 20th St
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

(c) ENERCALC INC 1983-2023

DESCRIPTION: CANT. FB10

Maximum Forces & Stresses for Load Combinations

Load Combination			Max Stress Ratios										Moment Values			Shear Values		
Segment	Length	Span #	M	V	CD	CM	C _t	CLx	C _F	Cfu	C _i	C _r	M	fb	F'b	V	fv	F'v
+D+L+H, LL Comb Run (*L)						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.510	0.395	1.00	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	2,850.8	5.62	114.6	290.0
Length = 6.750 ft	2		0.510	0.395	1.00	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	2,850.8	3.49	114.6	290.0
+D+L+H, LL Comb Run (L*)						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.634	0.494	1.00	1.00	1.00	1.00	0.983	1.00	1.00	1.00	25.84	1,808.1	2,850.8	7.03	143.4	290.0
Length = 6.750 ft	2		0.634	0.494	1.00	1.00	1.00	1.00	0.983	1.00	1.00	1.00	25.84	1,808.1	2,850.8	4.07	143.4	290.0
+D+L+H, LL Comb Run (LL)						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.634	0.494	1.00	1.00	1.00	1.00	0.983	1.00	1.00	1.00	25.84	1,808.1	2,850.8	7.03	143.4	290.0
Length = 6.750 ft	2		0.634	0.494	1.00	1.00	1.00	1.00	0.983	1.00	1.00	1.00	25.84	1,808.1	2,850.8	4.25	143.4	290.0
+D+Lr+H, LL Comb Run (*L)						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.408	0.316	1.25	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	3,563.5	5.62	114.6	362.5
Length = 6.750 ft	2		0.408	0.316	1.25	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	3,563.5	3.31	114.6	362.5
+D+Lr+H, LL Comb Run (L*)						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.647	0.499	1.25	1.00	1.00	1.00	0.983	1.00	1.00	1.00	32.94	2,305.0	3,563.5	8.86	180.8	362.5
Length = 6.750 ft	2		0.647	0.499	1.25	1.00	1.00	1.00	0.983	1.00	1.00	1.00	32.94	2,305.0	3,563.5	5.12	180.8	362.5
+D+Lr+H, LL Comb Run (LL)						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.647	0.499	1.25	1.00	1.00	1.00	0.983	1.00	1.00	1.00	32.94	2,305.0	3,563.5	8.86	180.8	362.5
Length = 6.750 ft	2		0.647	0.499	1.25	1.00	1.00	1.00	0.983	1.00	1.00	1.00	32.94	2,305.0	3,563.5	5.12	180.8	362.5
+D+S+H						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.443	0.344	1.15	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	3,278.4	5.62	114.6	333.5
Length = 6.750 ft	2		0.443	0.344	1.15	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	3,278.4	3.31	114.6	333.5
+D+0.750Lr+0.750L+H, LL Cc						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.408	0.316	1.25	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	3,563.5	5.62	114.6	362.5
Length = 6.750 ft	2		0.408	0.316	1.25	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	3,563.5	3.45	114.6	362.5
+D+0.750Lr+0.750L+H, LL Cc						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.662	0.513	1.25	1.00	1.00	1.00	0.983	1.00	1.00	1.00	33.70	2,358.2	3,563.5	9.11	185.9	362.5
Length = 6.750 ft	2		0.662	0.513	1.25	1.00	1.00	1.00	0.983	1.00	1.00	1.00	33.70	2,358.2	3,563.5	5.23	185.9	362.5
+D+0.750Lr+0.750L+H, LL Cc						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.662	0.513	1.25	1.00	1.00	1.00	0.983	1.00	1.00	1.00	33.70	2,358.2	3,563.5	9.11	185.9	362.5
Length = 6.750 ft	2		0.662	0.513	1.25	1.00	1.00	1.00	0.983	1.00	1.00	1.00	33.70	2,358.2	3,563.5	5.37	185.9	362.5
+D+0.750L+0.750S+H, LL Co						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.443	0.344	1.15	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	3,278.4	5.62	114.6	333.5
Length = 6.750 ft	2		0.443	0.344	1.15	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	3,278.4	3.45	114.6	333.5
+D+0.750L+0.750S+H, LL Co						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.524	0.408	1.15	1.00	1.00	1.00	0.983	1.00	1.00	1.00	24.57	1,719.4	3,278.4	6.67	136.2	333.5
Length = 6.750 ft	2		0.524	0.408	1.15	1.00	1.00	1.00	0.983	1.00	1.00	1.00	24.57	1,719.4	3,278.4	3.88	136.2	333.5
+D+0.750L+0.750S+H, LL Co						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.524	0.408	1.15	1.00	1.00	1.00	0.983	1.00	1.00	1.00	24.57	1,719.4	3,278.4	6.67	136.2	333.5
Length = 6.750 ft	2		0.524	0.408	1.15	1.00	1.00	1.00	0.983	1.00	1.00	1.00	24.57	1,719.4	3,278.4	4.01	136.2	333.5
+D+0.60W+H						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.319	0.247	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	4,561.3	5.62	114.6	464.0
Length = 6.750 ft	2		0.319	0.247	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	4,561.3	3.31	114.6	464.0
+D+0.70E+H						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.319	0.247	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	4,561.3	5.62	114.6	464.0
Length = 6.750 ft	2		0.319	0.247	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	4,561.3	3.31	114.6	464.0
+D+0.750Lr+0.750L+0.450W+						1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1		0.319	0.247	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	4,561.3	5.62	114.6	464.0

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Project Title: 85-25 - E 20th St
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

(c) ENERCALC INC 1983-2023

DESCRIPTION: CANT. FB10

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F _b	V	fv	F _v
Length = 6.750 ft	2	0.319	0.247	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	4,561.3	3.45	114.6	464.0
+D+0.750Lr+0.750L+0.450W+					1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1	0.517	0.401	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	33.70	2,358.2	4,561.3	9.11	185.9	464.0
Length = 6.750 ft	2	0.517	0.401	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	33.70	2,358.2	4,561.3	5.23	185.9	464.0
+D+0.750Lr+0.750L+0.450W+					1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1	0.517	0.401	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	33.70	2,358.2	4,561.3	9.11	185.9	464.0
Length = 6.750 ft	2	0.517	0.401	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	33.70	2,358.2	4,561.3	5.37	185.9	464.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1	0.319	0.247	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	4,561.3	5.62	114.6	464.0
Length = 6.750 ft	2	0.319	0.247	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	4,561.3	3.45	114.6	464.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1	0.377	0.293	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	24.57	1,719.4	4,561.3	6.67	136.2	464.0
Length = 6.750 ft	2	0.377	0.293	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	24.57	1,719.4	4,561.3	3.88	136.2	464.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1	0.377	0.293	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	24.57	1,719.4	4,561.3	6.67	136.2	464.0
Length = 6.750 ft	2	0.377	0.293	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	24.57	1,719.4	4,561.3	4.01	136.2	464.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1	0.319	0.247	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	4,561.3	5.62	114.6	464.0
Length = 6.750 ft	2	0.319	0.247	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	20.77	1,453.3	4,561.3	3.45	114.6	464.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1	0.377	0.293	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	24.57	1,719.4	4,561.3	6.67	136.2	464.0
Length = 6.750 ft	2	0.377	0.293	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	24.57	1,719.4	4,561.3	3.88	136.2	464.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1	0.377	0.293	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	24.57	1,719.4	4,561.3	6.67	136.2	464.0
Length = 6.750 ft	2	0.377	0.293	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	24.57	1,719.4	4,561.3	4.01	136.2	464.0
+0.60D+0.60W+0.60H					1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1	0.191	0.148	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	12.46	872.0	4,561.3	3.37	68.8	464.0
Length = 6.750 ft	2	0.191	0.148	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	12.46	872.0	4,561.3	1.99	68.8	464.0
+0.60D+0.70E+0.60H					1.00	1.00	1.00	0.983	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 3.750 ft	1	0.191	0.148	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	12.46	872.0	4,561.3	3.37	68.8	464.0
Length = 6.750 ft	2	0.191	0.148	1.60	1.00	1.00	1.00	0.983	1.00	1.00	1.00	12.46	872.0	4,561.3	1.99	68.8	464.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750Lr+0.750L+0.450W+H, I	1	0.2856	0.000	+D+0.750Lr+0.750L+0.450W+H, I	0.0000	0.000
	2	0.0000	0.000		-0.0636	2.866

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions		14.852	0.270
Max Upward from Load Combinations		14.852	
Max Upward from Load Cases		9.172	0.270
Max Downward from all Load Conditions			-4.635
Max Downward from Load Combinations			-4.635
Max Downward from Load Cases (Resis			-2.719
+D+H		9.172	-2.719
+D+L+H, LL Comb Run (*L)		9.442	-2.449
+D+L+H, LL Comb Run (L*)		11.425	-3.470
+D+L+H, LL Comb Run (LL)		11.695	-3.200
+D+Lr+H, LL Comb Run (*L)		9.172	-2.719
+D+Lr+H, LL Comb Run (L*)		14.222	-4.523
+D+Lr+H, LL Comb Run (LL)		14.222	-4.523

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Project Title: 85-25 - E 20th St
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

(c) ENERCALC INC 1983-2023

DESCRIPTION: CANT. FB10

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
+D+S+H		9.172	-2.719
+D+0.750Lr+0.750L+H, LL Comb Run (*)		9.375	-2.517
+D+0.750Lr+0.750L+H, LL Comb Run (L		14.649	-4.635
+D+0.750Lr+0.750L+H, LL Comb Run (L		14.852	-4.433
+D+0.750L+0.750S+H, LL Comb Run (*L		9.375	-2.517
+D+0.750L+0.750S+H, LL Comb Run (L*		10.862	-3.283
+D+0.750L+0.750S+H, LL Comb Run (LL		11.065	-3.080
+D+0.60W+H		9.172	-2.719
+D+0.70E+H		9.172	-2.719
+D+0.750Lr+0.750L+0.450W+H, LL Comb		9.375	-2.517
+D+0.750Lr+0.750L+0.450W+H, LL Comb		14.649	-4.635
+D+0.750Lr+0.750L+0.450W+H, LL Comb		14.852	-4.433
+D+0.750L+0.750S+0.450W+H, LL Comb		9.375	-2.517
+D+0.750L+0.750S+0.450W+H, LL Comb		10.862	-3.283
+D+0.750L+0.750S+0.450W+H, LL Comb		11.065	-3.080
+D+0.750L+0.750S+0.5250E+H, LL Comb		9.375	-2.517
+D+0.750L+0.750S+0.5250E+H, LL Comb		10.862	-3.283
+D+0.750L+0.750S+0.5250E+H, LL Comb		11.065	-3.080
+0.60D+0.60W+0.60H		5.503	-1.632
+0.60D+0.70E+0.60H		5.503	-1.632
D Only		9.172	-2.719
Lr Only, LL Comb Run (L*)		5.049	-1.803
Lr Only, LL Comb Run (LL)		5.049	-1.803
L Only, LL Comb Run (*L)		0.270	0.270
L Only, LL Comb Run (L*)		2.253	-0.751
L Only, LL Comb Run (LL)		2.523	-0.481
H Only			

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Project Title: 85-25 - E 20th St
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

(c) ENERCALC INC 1983-2023

DESCRIPTION: FB12

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

Analysis Method : Allowable Stress Design

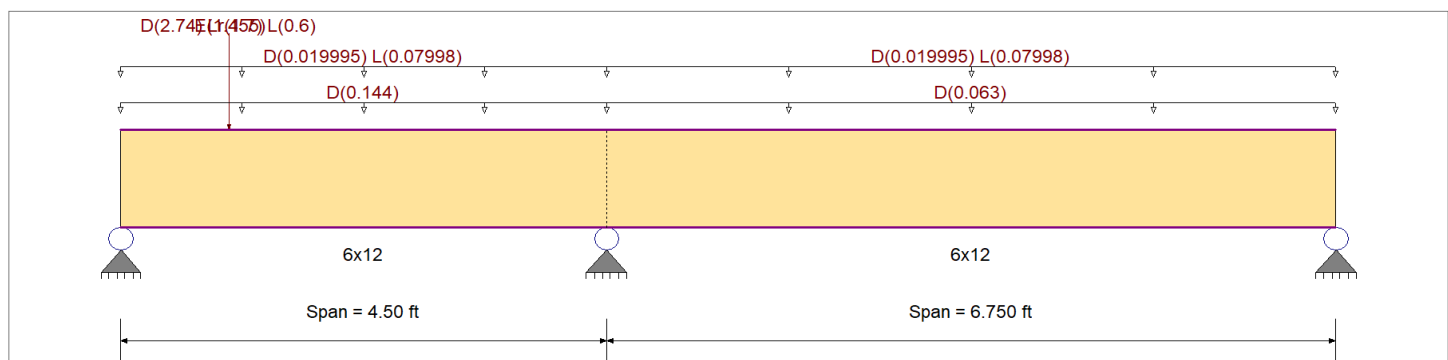
Load Combination : IBC 2021

Wood Species : Douglas Fir-Larch

Wood Grade : No.1

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

Fb +	1350 psi	E : Modulus of Elasticity	
Fb -	1350 psi	Ebend- xx	1600ksi
Fc - Prll	925 psi	Eminbend - xx	580ksi
Fc - Perp	625 psi		
Fv	170 psi		
Ft	675 psi	Density	31.21pcf



Applied Loads

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

Beam self weight calculated and added to loading

Load for Span Number 1

Uniform Load : D = 0.0180 ksf, Tributary Width = 8.0 ft, (WALL)

Uniform Load : D = 0.0150, L = 0.060 ksf, Tributary Width = 1.333 ft, (FJ)

Point Load : D = 2.740, Lr = 1.70, L = 0.60 k @ 1.0 ft, (FB9)

Point Load : E = 1.455 k @ 1.0 ft, (SW9)

Load for Span Number 2

Uniform Load : D = 0.0180 ksf, Tributary Width = 3.50 ft, (WALL)

Uniform Load : D = 0.0150, L = 0.060 ksf, Tributary Width = 1.333 ft, (FJ)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio			=	0.209	1	Maximum Shear Stress Ratio			=	0.387	1
Section used for this span				6x12		Section used for this span				6x12	
fb: Actual			=	353.14psi		fv: Actual			=	82.17 psi	
F'b			=	1,687.50psi		F'v			=	212.50 psi	
Load Combination +D+0.750Lr+0.750L+H, LL Comb Run (L*)						Load Combination +D+0.750Lr+0.750L+H, LL Comb Run (L*)					
Location of maximum on span			=	1.006ft		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			0.003 in	Ratio =	20663	Span: 2 : L Only, LL Comb Run (*L)					
Max Upward Transient Deflection			-0.001 in	Ratio =	54546	Span: 2 : Lr Only, LL Comb Run (L*)					
Max Downward Total Deflection			0.008 in	Ratio =	6972	Span: 2 : +D+L+H, LL Comb Run (*L)					
Max Upward Total Deflection			-0.003 in	Ratio =	29649	Span: 2 : +D+0.750Lr+0.750L+H, LL Comb Run (L*)					

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
+D+H																	
Length = 4.50 ft	1	0.180	0.330	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.21	218.7	1,215.0	2.13	50.6	153.0
Length = 6.750 ft	2	0.084	0.330	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.03	102.1	1,215.0	0.39	50.6	153.0
+D+L+H, LL Comb Run (*L)																	
					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0

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Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

(c) ENERCALC INC 1983-2023

DESCRIPTION: FB12

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F _b	V	fv	F _v
Length = 4.50 ft	1	0.158	0.289	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.15	212.7	1,350.0	2.07	49.1	170.0
Length = 6.750 ft	2	0.096	0.289	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.30	129.1	1,350.0	0.62	49.1	170.0
+D+L+H, LL Comb Run (L*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.203	0.371	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.77	274.4	1,350.0	2.66	63.1	170.0
Length = 6.750 ft	2	0.090	0.371	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.23	121.4	1,350.0	0.42	63.1	170.0
+D+L+H, LL Comb Run (LL)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.199	0.362	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.71	268.4	1,350.0	2.60	61.6	170.0
Length = 6.750 ft	2	0.110	0.362	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.50	148.4	1,350.0	0.65	61.6	170.0
+D+Lr+H, LL Comb Run (*L)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.130	0.238	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.21	218.7	1,687.5	2.13	50.6	212.5
Length = 6.750 ft	2	0.060	0.238	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.03	102.1	1,687.5	0.39	50.6	212.5
+D+Lr+H, LL Comb Run (L*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.203	0.377	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.46	342.2	1,687.5	3.38	80.2	212.5
Length = 6.750 ft	2	0.079	0.377	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.35	134.1	1,687.5	0.44	80.2	212.5
+D+Lr+H, LL Comb Run (LL)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.203	0.377	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.46	342.2	1,687.5	3.38	80.2	212.5
Length = 6.750 ft	2	0.079	0.377	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.35	134.1	1,687.5	0.44	80.2	212.5
+D+S+H					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.141	0.259	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.21	218.7	1,552.5	2.13	50.6	195.5
Length = 6.750 ft	2	0.066	0.259	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.03	102.1	1,552.5	0.39	50.6	195.5
+D+0.750Lr+0.750L+H, LL Cc					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.127	0.233	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.16	214.2	1,687.5	2.09	49.5	212.5
Length = 6.750 ft	2	0.073	0.233	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.24	122.4	1,687.5	0.56	49.5	212.5
+D+0.750Lr+0.750L+H, LL Cc					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.209	0.387	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.57	353.1	1,687.5	3.46	82.2	212.5
Length = 6.750 ft	2	0.083	0.387	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.42	140.6	1,687.5	0.45	82.2	212.5
+D+0.750Lr+0.750L+H, LL Cc					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.207	0.382	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.52	348.6	1,687.5	3.42	81.1	212.5
Length = 6.750 ft	2	0.095	0.382	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.62	160.9	1,687.5	0.62	81.1	212.5
+D+0.750L+0.750S+H, LL Co					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.138	0.253	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.16	214.2	1,552.5	2.09	49.5	195.5
Length = 6.750 ft	2	0.079	0.253	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.24	122.4	1,552.5	0.56	49.5	195.5
+D+0.750L+0.750S+H, LL Co					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.168	0.307	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.63	260.5	1,552.5	2.53	59.9	195.5
Length = 6.750 ft	2	0.075	0.307	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.18	116.6	1,552.5	0.41	59.9	195.5
+D+0.750L+0.750S+H, LL Co					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.165	0.301	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.59	256.0	1,552.5	2.48	58.9	195.5
Length = 6.750 ft	2	0.088	0.301	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.38	136.9	1,552.5	0.59	58.9	195.5
+D+0.60W+H					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.101	0.186	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.21	218.7	2,160.0	2.13	50.6	272.0
Length = 6.750 ft	2	0.047	0.186	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.03	102.1	2,160.0	0.39	50.6	272.0
+D+0.70E+H					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.136	0.251	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.96	292.7	2,160.0	2.88	68.3	272.0
Length = 6.750 ft	2	0.056	0.251	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.22	121.3	2,160.0	0.42	68.3	272.0
+D+0.750Lr+0.750L+0.450W+					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.099	0.182	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.16	214.2	2,160.0	2.09	49.5	272.0
Length = 6.750 ft	2	0.057	0.182	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.24	122.4	2,160.0	0.56	49.5	272.0

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Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

(c) ENERCALC INC 1983-2023

DESCRIPTION: FB12

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F ^b	V	fv	F ^v
+D+0.750Lr+0.750L+0.450W+					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.163	0.302	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.57	353.1	2,160.0	3.46	82.2	272.0
Length = 6.750 ft	2	0.065	0.302	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.42	140.6	2,160.0	0.45	82.2	272.0
+D+0.750Lr+0.750L+0.450W+					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.161	0.298	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.52	348.6	2,160.0	3.42	81.1	272.0
Length = 6.750 ft	2	0.074	0.298	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.62	160.9	2,160.0	0.62	81.1	272.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.099	0.182	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.16	214.2	2,160.0	2.09	49.5	272.0
Length = 6.750 ft	2	0.057	0.182	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.24	122.4	2,160.0	0.56	49.5	272.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.121	0.220	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.63	260.5	2,160.0	2.53	59.9	272.0
Length = 6.750 ft	2	0.054	0.220	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.18	116.6	2,160.0	0.41	59.9	272.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.118	0.216	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.59	256.0	2,160.0	2.48	58.9	272.0
Length = 6.750 ft	2	0.063	0.216	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.38	136.9	2,160.0	0.59	58.9	272.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.125	0.231	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.72	269.7	2,160.0	2.65	62.8	272.0
Length = 6.750 ft	2	0.063	0.231	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.38	136.8	2,160.0	0.59	62.8	272.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.146	0.269	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.19	316.0	2,160.0	3.09	73.3	272.0
Length = 6.750 ft	2	0.061	0.269	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.32	130.9	2,160.0	0.43	73.3	272.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.144	0.265	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.15	311.5	2,160.0	3.04	72.2	272.0
Length = 6.750 ft	2	0.070	0.265	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.53	151.2	2,160.0	0.61	72.2	272.0
+0.60D+0.60W+0.60H					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.061	0.112	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.33	131.2	2,160.0	1.28	30.3	272.0
Length = 6.750 ft	2	0.028	0.112	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.62	61.3	2,160.0	0.23	30.3	272.0
+0.60D+0.70E+0.60H					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.095	0.177	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.07	205.2	2,160.0	2.03	48.1	272.0
Length = 6.750 ft	2	0.037	0.177	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.81	80.4	2,160.0	0.26	48.1	272.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750Lr+0.750L+0.450W+H, I	1	0.0077	1.860		0.0000	0.000
L Only, LL Comb Run (*L)	2	0.0022	3.620	+D+0.750Lr+0.750L+0.450W+H, I	-0.0027	2.149

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	3.692	2.658	0.403
Max Upward from Load Combinations	3.692	2.658	0.403
Max Upward from Load Cases	2.302	1.717	0.229
Max Downward from all Load Conditio	-0.061		-0.048
Max Downward from Load Cases (Resis	-0.061		-0.048
+D+H	2.302	1.717	0.174
+D+L+H, LL Comb Run (*L)	2.241	2.088	0.403
+D+L+H, LL Comb Run (L*)	2.905	2.103	0.145
+D+L+H, LL Comb Run (LL)	2.844	2.474	0.374
+D+Lr+H, LL Comb Run (*L)	2.302	1.717	0.174
+D+Lr+H, LL Comb Run (L*)	3.552	2.215	0.126
+D+Lr+H, LL Comb Run (LL)	3.552	2.215	0.126
+D+S+H	2.302	1.717	0.174
+D+0.750Lr+0.750L+H, LL Comb Run (*)	2.256	1.995	0.346

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Project Title: 85-25 - E 20th St
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

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DESCRIPTION: FB12

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
+D+0.750Lr+0.750L+H, LL Comb Run (L	3.692	2.379	0.116
+D+0.750Lr+0.750L+H, LL Comb Run (L	3.646	2.658	0.288
+D+0.750L+0.750S+H, LL Comb Run (*L	2.256	1.995	0.346
+D+0.750L+0.750S+H, LL Comb Run (L*	2.754	2.006	0.152
+D+0.750L+0.750S+H, LL Comb Run (LL	2.709	2.285	0.324
+D+0.60W+H	2.302	1.717	0.174
+D+0.70E+H	3.051	2.015	0.145
+D+0.750Lr+0.750L+0.450W+H, LL Comb	2.256	1.995	0.346
+D+0.750Lr+0.750L+0.450W+H, LL Comb	3.692	2.379	0.116
+D+0.750Lr+0.750L+0.450W+H, LL Comb	3.646	2.658	0.288
+D+0.750L+0.750S+0.450W+H, LL Comb	2.256	1.995	0.346
+D+0.750L+0.750S+0.450W+H, LL Comb	2.754	2.006	0.152
+D+0.750L+0.750S+0.450W+H, LL Comb	2.709	2.285	0.324
+D+0.750L+0.750S+0.5250E+H, LL Comb	2.818	2.219	0.324
+D+0.750L+0.750S+0.5250E+H, LL Comb	3.316	2.230	0.130
+D+0.750L+0.750S+0.5250E+H, LL Comb	3.271	2.508	0.302
+0.60D+0.60W+0.60H	1.381	1.030	0.104
+0.60D+0.70E+0.60H	2.130	1.328	0.075
D Only	2.302	1.717	0.174
Lr Only, LL Comb Run (L*)	1.250	0.497	-0.048
Lr Only, LL Comb Run (LL)	1.250	0.497	-0.048
L Only, LL Comb Run (*L)	-0.061	0.371	0.229
L Only, LL Comb Run (L*)	0.603	0.386	-0.029
L Only, LL Comb Run (LL)	0.543	0.757	0.201
E Only	1.070	0.426	-0.041
H Only			

COLUMN AXIAL CAPACITY

WOOD PROPERTIES:

Use Douglas Fir-Larch	$C_F =$	1.1	$K_{ce} =$	0.30	Buckling about x-axis?	YES
Species: Stud	$C_m =$	1.0	$c =$	0.80	Check le/d reqmt.	OK!
$E = 1,400,000$ psi	$C_T =$	1.0	$K_f =$	1.00		
$F'_c = 850$ psi	$C_t =$	1.0	(for bolted or nailed members)			
$E' = 1,400,000$ psi	$C_i =$	1.0				
$F_{c*} = 935$ psi	$C_D =$	1.0				

COL	No. of Plies	Size	d_y	d_x	$le_{x,y}$	le/dy	le/dx	(le/d)	Fce	Fce/Fc*	$\frac{1+ Fce/Fc*}{2c}$	C_p	F'c	Column Capacity
C01	1 -	2x 4	1.5	3.50	8	64.0	27.4	27.4	558.27	0.597	1.00	0.498	465.79	2.45 kips
C02	2 -	2x 4	3.0	3.50	8	32.0	27.4	27.4	558.27	0.597	1.00	0.498	465.79	4.89 kips
C03	1 -	2x 6	1.5	5.50	8	64.0	17.5	17.5	1,378.58	1.474	1.55	0.806	753.41	6.22 kips
C04	2 -	2x 6	3.0	5.50	8	32.0	17.5	17.5	1,378.58	1.474	1.55	0.806	753.41	12.43 kips
C05	1 -	2x 8	1.5	7.50	8	64.0	12.8	12.8	2,563.48	2.742	2.34	0.910	850.54	9.57 kips
C06	2 -	2x 8	3.0	7.25	8	32.0	13.2	13.2	2,395.43	2.562	2.23	0.902	843.35	18.34 kips

WOOD PROPERTIES:

Use Douglas Fir-Larch	$C_F =$	1.1	$K_{ce} =$	0.30	Buckling about x-axis?	YES
Species: No.1	$C_m =$	1.0	$c =$	0.80	Check le/d reqmt.	OK!
$E = 1,700,000$ psi	$C_T =$	1.0	$K_f =$	1.00		
$F'_c = 1,500$ psi	$C_t =$	1.0	(for bolted or nailed members)			
$E' = 1,700,000$ psi	$C_i =$	1.0				
$F_{c*} = 1,650$ psi	$C_D =$	1.0				

COL	No. of Plies	Size	d_y	d_x	$le_{x,y}$	le/dy	le/dx	(le/d)	Fce	Fce/Fc*	$\frac{1+ Fce/Fc*}{2c}$	C_p	F'c	Column Capacity
C07	1 -	4x 4	3.5	3.50	8	27.4	27.4	27.4	558.27	0.597	1.00	0.498	465.79	5.71 kips
C08	1 -	4x 6	3.5	5.50	8	27.4	17.5	17.5	1,378.58	1.474	1.55	0.806	753.41	14.50 kips
C09	1 -	4x 8	3.5	7.50	8	27.4	12.8	12.8	2,563.48	2.742	2.34	0.910	850.54	22.33 kips
C10	1 -	6x 6	5.5	5.50	8	17.5	17.5	17.5	1,378.58	1.474	1.55	0.806	753.41	22.79 kips
C11	1 -	6x 8	5.5	7.25	8	17.5	13.2	13.2	2,395.43	2.562	2.23	0.902	843.35	33.63 kips

Note:

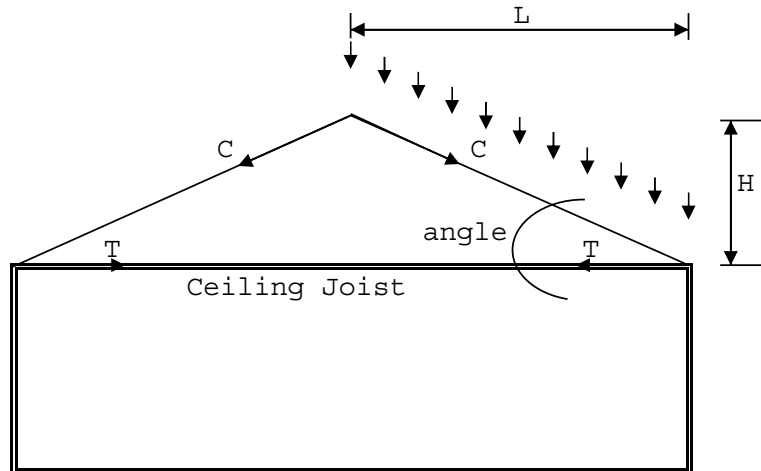
$K_{c_E} = 0.510 - 0.839$ (COV)	where COV =	0.25	- sawn lumber
0.3	- for visually graded lumber	0.15	- MEL
0.384	- for machine evaluated lumber (MEL)	0.11	- MSR (machine stress rated)
0.418	- for products w/ $COV \leq 0.11$	0.10	- glu-lam

FOUNDATION DESIGN

ALLOW BEARING CAPACITY : 1,500.00 psf
 CONCRETE STRENGTH, f'_c : 2,500.00 psi
 YEILD STRENGTH, F_y : 60,000.00 psi

		EXT. WALL FTG	INT. WALL FTG	F1	F1	F2
FOOTING DIMENSION						
WIDTH	in	18.0	18.0	18.0	30.0	42.0
LENGTH	in	12.0	12.0	18.0	30.0	42.0
DEPTH	in	24.0	24.0	12.0	12.0	12.0
COLUMN DIMENSION						
WIDTH	in	4.0	4.0	4.0	4.0	4.0
LENGTH	in	12.0	12.0	4.0	6.0	6.0
FOOTING LOAD						
ROOF TRIB. AREA	ft2	18.0	20.0			
CEILING TRIB. AREA	ft2	18.0	20.0			
FLOOR TRIB. AREA	ft2	-	-			
DECK TRIB. AREA	ft2	-	-			
WALL AREA	ft2	20.0	20.0			
WALL TYPE		EXT	INT			
DEAD LOAD	D lbs	702.0	580.1			
LIVE LOAD	L lbs	720.0	800.0			
SEISMIC LOAD	E lbs	-	-			
CHECK BEARING CAPACITY						
SERVICE LOAD, (D+L)	lbs	1,422.0	1,380.1			
SERVICE LOAD, (D+L+E)/1.33	lbs	1,069.2	1,037.6			
SOIL PRESSURE, $f_b = P/A_{FTG}$	psf	948.0	920.0			
CHECK $f_b < F_b$		OK	OK			
CHECK MINIMUM REINFORCEMENT						
REINFORCEMENT USE		2#4	2#4	2#4	3#4	4#4
AREA OF STEEL, A_s	in2	0.40	0.40			
$A_{s1 \text{ min}} = 0.0018 * \text{Area_Conc.}$	in2	0.68	0.68			
$A_{s2 \text{ min}} = 1.33 * A_{s \text{ Required}}$	in2	0.00	0.00			
$A_s \text{ min} = \text{Min}(A_{s1}, A_{s2})$	in2	0.00	0.00			
CHECK $A_s > A_s \text{ MIN}$		OK	OK			
CHECK BENDING STRESSES						
FACTORED, $1.4D+1.7L$	lbs	2,207	2,172			
FACTORED, $1.4(D+L+E)/1.33$	lbs	1,991	1,932			
MAXIMUM MOMENT, M_u	lb-in	3,004	2,956			
b	in	12.00	12.00			
$a = A_s * F_y / 0.85 * f'_c * b$	in	0.94	0.94			
$\phi M_n = \phi * A_s * F_y * (d - a/2)$	lb-in	443,435	443,435			
CHECK $M_u < \phi M_n$		OK	OK			
CHECK SHEAR STRESSES						
MAXIMUM SHEAR, V_u	lbs	-	-			
b	in	12.00	12.00			
$\phi V_c = 2 * \phi * b * w * d * \text{sqrt}(f'_c)$	lbs	21,420	21,420			
CHECK $V_u < \phi V_c$		OK	OK			

Nailing & Bolting Schedule for eaves tie connections



$W = L(\text{ft}) \times \text{rafter spacing (ft)} \times \text{roof design load (L.L. + D.L. in psf)}$

$$T = \frac{W \times L}{2H}$$

Given:

Roof Rafter

Roof Rafter Trib. Width	L	=	9.5	ft
Roof Dead Load	DL	=	13.003	psf
Roof Live Load	LL	=	20	psf
Roof Rafter Spacing	s	=	1.33	ft
Roof Pitch			5:12	

Calculated:

Tan a=	0.42
Angle a	22.62
H=	$L \times \tan a$
	3.96
W=	417
T=	500

Design criteria:

Nail Capcit: #/nail	Cd	#/nail
8d	98	1.25
16d	103	1.25
1/2" MB		400

Number of nails required: $T/\text{nail capacity}$

8d=	4 nails @ each connection
16d=	4 nails @ each connection
1/2" bolts=	1 bolts @ each connection

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 Engineer:
 Project ID:
 Project Descr:

2D Frame

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

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DESCRIPTION: MF1

Joints...

Joint Label	Joint Coordinates		X Translational Restraint	Y Translational Restraint	Z Rotational Restraint	Joint Temp deg F
	X ft	Y ft				
1	0.0	0.0	Fixed	Fixed	Fixed	0
2	0.0	10.0				0
3	25.0	10.0				0
4	25.0	0.0	Fixed	Fixed	Fixed	0

Members...

Member Label	Property Label	Endpoint Joints		Member Length ft	Releases Specify Connectivity of Member Ends to Joints					
		I Joint	J Joint		x	I End y	z (rotation)	x	J End y	z (rotation)
A	Column	1	2	10.000	Fixed	Fixed	Pinned	Fixed	Fixed	Fixed
B	Beam	2	3	25.000	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
C	Column	3	4	10.000	Fixed	Fixed	Fixed	Fixed	Fixed	Pinned

Member Stress Check Data...

Member Label	Unbraced Lengths		Slenderness Factors		AISC Bending & Stability Factors	
	Lu : z ft	Lu : y	K : z	K : y	Cm	Cb
A	10.000	10.000	1.00	1.00	Internal	Internal
B	25.000	25.000	1.00	1.00	Internal	Internal
C	10.000	10.000	1.00	1.00	Internal	Internal

Materials...

Member Label	Youngs ksi	Density kcf	Thermal in/deg	Yield ksi
Default	1.00	0.000	0.000000	1.00
Steel	29,000.00	0.490	0.000007	50.00
Wood	1,800.00	0.035	0.000000	0.00

Wood Material Data...

Wood, Not Defined, Density= 35.0pcf, FbT= 1000psi, FbC= 1000psi, Fv= 1000psi, Ft= 1000, Fc= 400psi, E Bend XX= 1800ksi, E BendMin XX= 1800ksi, E Beny YY= 1800ksi, E BendMin YY= 1800ksi, E Axial= 1800ksi, Species= , Grade= Any, Class=

Member Sections...

Prop Label	Group Tag	Material	Area	Depth	Width	Ixx	Iyy
Default	Group	Default	1.0 in^2	0.0 in	0.0 in	1.0 in^4	1.0 in^4
W10x45	Column	Steel	13.30 in^2	10.10 in	8.020 in	248.0 in^4	53.40 in^4
W10x45	Beam	Steel	13.30 in^2	10.10 in	8.020 in	248.0 in^4	53.40 in^4

Joint Loads....

Note: Loads labeled "Global Y" act downward (in "-Y" direction)

Joint Label	Load Direction	Load Magnitude					
		Dead	Roof Live	Live	Snow	Seismic	Wind
2	Global X				4.747		k

Member Distributed Loads...

Note: Loads labeled "Global Y" act downward (in "-Y" direction)

Member Label	Load Direction	Load Extents		Load Magnitude					
		Start ft	End ft	Dead	Roof Live	Live	Snow	Seismic	Wind
B	Global Y	0.0	25.0	Start Mag : 0.0150	0.020				k/ft
			End Mag : 0.0150	0.020					k/ft

Stress/Strength Load Combinations

IBC 2021

Load Combination Description	Cd	Load Combination Factors							
		Dead	0.2*Sds* Seismic	Roof Live	Live	Snow	Wind	Seismic	Rho
+D+H	0.9	1.0							1.0
+D+L+H	1	1.0			1.0				1.0
+D+Lr+H	1.25	1.0		1.0					1.0

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FA STRUCTURAL

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DESCRIPTION: MF1

Stress/Strength Load Combinations

IBC 2021

Load Combination Description	Cd	Load Combination Factors								
		Dead	0.2*Sds* Seismic	Roof Live	Live	Snow	Wind	Seismic	Rho	Earth
+D+S+H	1.15	1.0				1.0				1.0
+D+0.750Lr+0.750L+H	1.25	1.0		0.750	0.750					1.0
+D+0.750L+0.750S+H	1.15	1.0			0.750	0.750				1.0
+D+0.60W+H	1.6	1.0					0.60			1.0
+D+0.70E+H	0.9	1.0						0.70		1.0
+D+0.750Lr+0.750L+0.450W+H	1.6	1.0		0.750	0.750		0.450			1.0
+D+0.750L+0.750S+0.450W+H	1.6	1.0			0.750	0.750	0.450			1.0
+D+0.750L+0.750S+0.5250E+H	1.15	1.0			0.750	0.750		0.5250		1.0
+0.60D+0.60W+0.60H	1.6	0.60					0.60			0.60
+0.60D+0.70E+0.60H	0.9	0.60						0.70		0.60

Reaction Load Combinations

IBC 2021

Load Combination Description	Load Combination Factors							
	Dead	Roof Live	Live	Snow	Wind	Seismic	Earth	
+D+H	1.0						1.0	
+D+L+H	1.0		1.0				1.0	
+D+Lr+H	1.0	1.0					1.0	
+D+S+H	1.0			1.0			1.0	
+D+0.750Lr+0.750L+H	1.0	0.750	0.750				1.0	
+D+0.750L+0.750S+H	1.0		0.750	0.750			1.0	
+D+0.60W+H	1.0				0.60		1.0	
+D+0.70E+H	1.0					0.70	1.0	
+D+0.750Lr+0.750L+0.450W+H	1.0	0.750	0.750		0.450		1.0	
+D+0.750L+0.750S+0.450W+H	1.0		0.750	0.750	0.450		1.0	
+D+0.750L+0.750S+0.5250E+H	1.0		0.750	0.750		0.5250	1.0	
+0.60D+0.60W+0.60H	0.60				0.60		0.60	
+0.60D+0.70E+0.60H	0.60					0.70	0.60	
D Only	1.0							
Lr Only		1.0						
L Only			1.0					
S Only				1.0				
W Only					1.0			
E Only						1.0		
H Only							1.0	

Deflection Load Combinations

IBC 2021

Load Combination Description	Load Combination Factors						
	Dead	Roof Live	Live	Snow	Wind	Seismic	Earth
+D+H	1.0						1.0
+D+L+H	1.0		1.0				1.0
+D+Lr+H	1.0	1.0					1.0
+D+S+H	1.0			1.0			1.0
+D+0.750Lr+0.750L+H	1.0	0.750	0.750				1.0
+D+0.750L+0.750S+H	1.0		0.750	0.750			1.0
+D+0.60W+H	1.0				0.60		1.0
+D+0.70E+H	1.0					0.70	1.0
+D+0.750Lr+0.750L+0.450W+H	1.0	0.750	0.750		0.450		1.0
+D+0.750L+0.750S+0.450W+H	1.0		0.750	0.750	0.450		1.0
+D+0.750L+0.750S+0.5250E+H	1.0		0.750	0.750		0.5250	1.0
+0.60D+0.60W+0.60H	0.60				0.60		0.60
+0.60D+0.70E+0.60H	0.60					0.70	0.60
D Only	1.0						
Lr Only		1.0					
L Only			1.0				
S Only				1.0			
W Only					1.0		
E Only						1.0	
H Only							1.0

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2D Frame

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FA STRUCTURAL

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DESCRIPTION: MF1

Extreme Joint Displacements

Only Load Combinations giving maximum values are listed

Joint Label	Joint Displacements		
	X in	Y in	Z Radians
1 Max	0.0 E Only	0.0 E Only	0.0
1 Min	0.0 +D+Lr	0.0 +D+Lr	0.0
2 Max	0.4291 E Only	0.000591 E Only	-0.000250 +0.60D
2 Min	0.000140 +0.60D	-0.000136 +D+Lr	-0.001990 E Only
3 Max	0.4273 E Only	-0.000350 +0.60D	0.000960 +D+Lr
3 Min	-0.000560 +D+Lr	-0.000591 E Only	-0.001978 E Only
4 Max	0.0 E Only	0.0 +0.60D	0.0
4 Min	0.0 +0.60D	0.0 E Only	0.0

Extreme Joint Reactions

Only Load Combinations giving maximum values are listed

Joint Label	Joint Reactions		
	X k	Y k	Z k-ft
1 Max	0.1438 +D+Lr	0.4375 +D+Lr	
1 Min	-2.376 E Only	-1.899 E Only	
2 Max			
2 Min			
3 Max			
3 Min			
4 Max	-0.03697 +0.60D	1.899 E Only	
4 Min	-2.371 E Only	0.1125 +0.60D	

Extreme Member End Forces

Only Load Combinations giving maximum values are listed

Member Label	Member " I " End Forces			Member " J " End Forces		
	Axial k	Shear k	Moment k-ft	Axial k	Shear k	Moment k-ft
A Max	0.4375 +D+Lr	1.626 +0.60D+0.70E	0.0 D Only	1.217 +0.60D+0.70E	0.1438 +D+Lr	16.262 +0.60D+0.70E
A Min	-1.217 +0.60D+0.70E	-0.1438 +D+Lr	0.0 D Only	-0.4375 +D+Lr	-1.626 +0.60D+0.70E	-1.438 +D+Lr
B Max	1.721 +D+0.70E	0.4375 +D+Lr	1.438 +D+Lr	-0.03697 +0.60D	1.517 +D+0.70E	-0.3697 +0.60D
B Min	0.03697 +0.60D	-1.217 +0.60D+0.70E	-16.262 +0.60D+0.70E	-1.721 +D+0.70E	0.1125 +0.60D	-17.214 +D+0.70E
C Max	1.517 +D+0.70E	1.721 +D+0.70E	17.214 +D+0.70E	-0.1125 +0.60D	-0.03697 +0.60D	0.0 D Only
C Min	0.1125 +0.60D	0.03697 +0.60D	0.3697 +0.60D	-1.517 +D+0.70E	-1.721 +D+0.70E	0.0 D Only

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2D Frame

Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

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DESCRIPTION: MF1

Extreme Member Forces

Only Load Combinations giving maximum values are listed

Mmbr Label	Axial	Dist from "I" Joint	Moment	Dist from "I" Joint	Shear	Dist from "I" Joint
A	1.217k	0.0 ft	1.438 k-ft	10.0 ft	1.626 k	0.0 ft
Max	+0.60D+0.70E		+D+Lr		+0.60D+0.70E	
A	-0.4375k	0.0 ft	-16.262 k-ft	10.0 ft	-0.1438 k	0.0 ft
Min	+D+Lr		+0.60D+0.70E		+D+Lr	
B	-0.03697k	0.0 ft	17.214 k-ft	25.0 ft	0.4375 k	0.0 ft
Max	+0.60D		+D+0.70E		+D+Lr	
B	-1.721k	0.0 ft	-16.262 k-ft	0.0 ft	-1.517 k	25.0 ft
Min	+D+0.70E		+0.60D+0.70E		+D+0.70E	
C	-0.1125k	0.0 ft	17.214 k-ft	0.0 ft	1.721 k	0.0 ft
Max	+0.60D		+D+0.70E		+D+0.70E	
C	-1.517k	0.0 ft	0.007545 k-ft	9.796 ft	0.03697 k	0.0 ft
Min	+D+0.70E		+0.60D		+0.60D	

Member Stress Checks...

Stress Checks per AISC 360-16 & NDS 2018

Member Label	Section Label	Material	Max. Axial + Bending Stress Ratios				Max. Shear Stress Ratios			
			Load Combination	Ratio	Status	Dist (ft)	Load Combination	Ratio	Status	Dist (ft)
A	Column	Steel	+0.60D+0.70E	0.127	PASS	10.00	+0.60D+0.70E	0.023	PASS	0.00
B	Beam	Steel	+D+0.70E	0.199	PASS	25.00	+D+0.70E	0.021	PASS	25.00
C	Column	Steel	+D+0.70E	0.135	PASS	0.00	+D+0.70E	0.024	PASS	0.00

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Project Title: 85-25 - E 20th St
Engineer:
Project ID:
Project Descr:

2D Frame

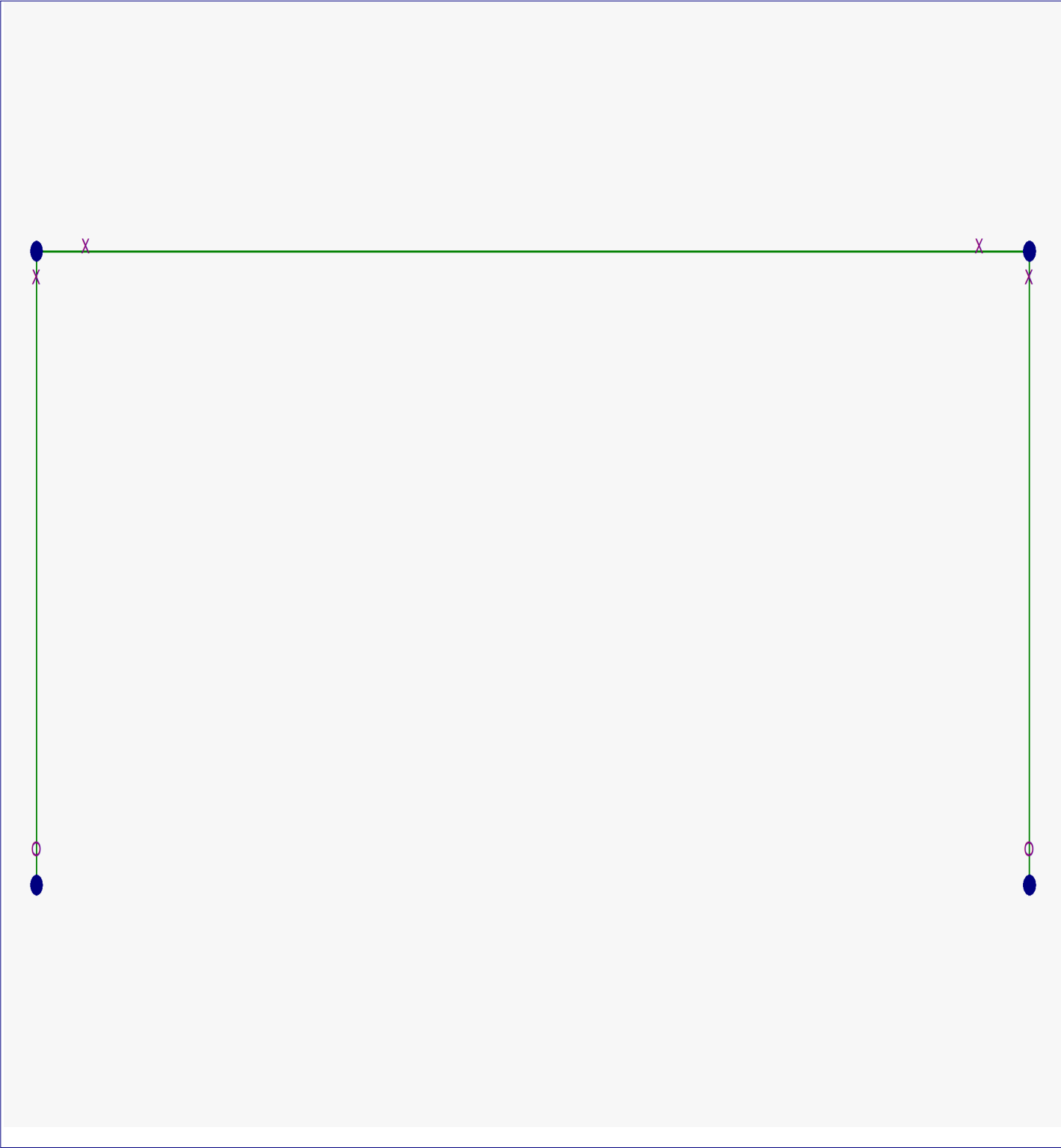
Project File: 85-25_Storage.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

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DESCRIPTION: MF1



DRIFT CALCULATION

ALLOWABLE STORY DRIFT , $\Delta a = 0.025 h_{sx}$ ASCE7- 16 - TABLE 12.12-1

DEFLECTION AMPLIFICATION FACTOR, $C_d = 4.5$ IMF ASCE7-16 - TABLE 12.2-

IMPORTANCE FACTOR, $I = 1$

STORY HEIGHT = 12 FT

ALLOWABLE STORY DRIFT, $\Delta a = 0.025 \times 12 \text{ ft} \times 12 = 3.6 \text{ ''}$

AMPLIFIED DISPLACEMENT, $\delta x = C_d \delta_{ex}/I$ ASCE7-10 - (12.8-15)

CHECK DRIFT

$\delta_e = 0.429 \text{ ''}$

$\delta_e = C_d \times \delta_e / I = 4.5 \times 0.427'' / 1 = 2.3595 < 3.6$

**E 20th St,
Upland, CA 91784**

STRUCTURAL CALCULATIONS
Trash Enclosure

Date : 09/12/25

Rev. : 0

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 : CA 92648
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Project Title: TRASH ENCLOSURE
 Engineer:
 Project ID: 85-25 Water Structure
 Project Descr:

Steel Beam

Project File: 85-25 TRASH ENCLOSURE.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

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DESCRIPTION: RB1

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

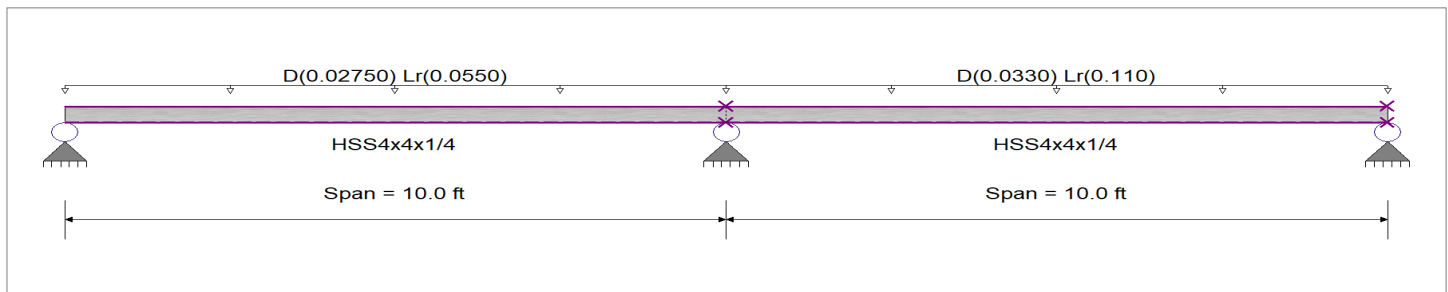
Analysis Method : Allowable Strength Design

Fy : Steel Yield : 50.0 ksi

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

E: Modulus : 29,000.0 ksi

Bending Axis : Major Axis Bending



Applied Loads

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

Beam self weight calculated and added to loading

Load for Span Number 1

Uniform Load : D = 0.0050, Lr = 0.010 ksf, Tributary Width = 5.50 ft, (RR)

Load for Span Number 2

Uniform Load : D = 0.0060, Lr = 0.020 ksf, Tributary Width = 5.50 ft, (RR)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.133 : 1	Maximum Shear Stress Ratio =		0.034 : 1
Section used for this span		HSS4x4x1/4	Section used for this span		HSS4x4x1/4
Ma : Applied		1.562 k-ft	Va : Applied		0.9321 k
Mn / Omega : Allowable		11.702 k-ft	Vn/Omega : Allowable		27.634 k
Load Combination		+D+Lr+H, LL Comb Run (LL)	Load Combination		+D+Lr+H, LL Comb Run (LL)
			Location of maximum on span		10.000 ft
Span # where maximum occurs		Span # 1	Span # where maximum occurs		Span # 1
Maximum Deflection					
Max Downward Transient Deflection		0.077 in Ratio = 1,553	>=360.	Span: 2 : Lr Only, LL Comb Run (*L)	
Max Upward Transient Deflection		-0.034 in Ratio = 3,543	>=360.	Span: 2 : Lr Only, LL Comb Run (*L)	
Max Downward Total Deflection		0.097 in Ratio = 1233	>=240.	Span: 2 : +D+Lr+H, LL Comb Run (*L)	
Max Upward Total Deflection		-0.023 in Ratio = 5145	>=240.	Span: 2 : +D+Lr+H, LL Comb Run (*L)	

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values						Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx/Vnx/Omega
+D+H												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70	1.00	1.00	0.28	46.15
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70	1.00	1.00	0.28	46.15
+D+L+H, LL Comb Run (*L)												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70	1.00	1.00	0.28	46.15
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70	1.00	1.00	0.28	46.15
+D+L+H, LL Comb Run (L*)												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70	1.00	1.00	0.28	46.15
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70	1.00	1.00	0.28	46.15
+D+L+H, LL Comb Run (LL)												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70	1.00	1.00	0.28	46.15
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70	1.00	1.00	0.28	46.15
+D+Lr+H, LL Comb Run (*L)												
Dsgn. L = 10.00 ft	1	0.104	0.032	0.07	-1.22	1.22	19.54	11.70	1.00	1.00	0.90	46.15
Dsgn. L = 10.00 ft	2	0.118	0.032	1.38	-1.22	1.38	19.54	11.70	1.00	1.00	0.90	46.15
+D+Lr+H, LL Comb Run (L*)												
Dsgn. L = 10.00 ft	1	0.075	0.020	0.79	-0.87	0.87	19.54	11.70	1.00	1.00	0.56	46.15
Dsgn. L = 10.00 ft	2	0.075	0.011	0.21	-0.87	0.87	19.54	11.70	1.00	1.00	0.31	46.15

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Project Title: TRASH ENCLOSURE
 Engineer:
 Project ID: 85-25 Water Structure
 Project Descr:

Steel Beam

Project File: 85-25 TRASH ENCLOSURE.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

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DESCRIPTION: RB1

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values						Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega Cb	Rm	Va Max	Vnx/Vnx/Omega	
+D+Lr+H, LL Comb Run (LL)												
Dsgn. L = 10.00 ft	1	0.133	0.034	0.53	-1.56	1.56	19.54	11.70 1.00 1.00		0.93	46.15	27.63
Dsgn. L = 10.00 ft	2	0.133	0.034	1.24	-1.56	1.56	19.54	11.70 1.00 1.00		0.93	46.15	27.63
+D+S+H												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
+D+0.750Lr+0.750L+H, LL Comb												
Dsgn. L = 10.00 ft	1	0.089	0.027	0.11	-1.05	1.05	19.54	11.70 1.00 1.00		0.74	46.15	27.63
Dsgn. L = 10.00 ft	2	0.095	0.027	1.12	-1.05	1.12	19.54	11.70 1.00 1.00		0.74	46.15	27.63
+D+0.750Lr+0.750L+H, LL Comb												
Dsgn. L = 10.00 ft	1	0.067	0.017	0.66	-0.79	0.79	19.54	11.70 1.00 1.00		0.48	46.15	27.63
Dsgn. L = 10.00 ft	2	0.067	0.011	0.24	-0.79	0.79	19.54	11.70 1.00 1.00		0.30	46.15	27.63
+D+0.750Lr+0.750L+H, LL Comb												
Dsgn. L = 10.00 ft	1	0.111	0.028	0.46	-1.30	1.30	19.54	11.70 1.00 1.00		0.77	46.15	27.63
Dsgn. L = 10.00 ft	2	0.111	0.028	1.01	-1.30	1.30	19.54	11.70 1.00 1.00		0.77	46.15	27.63
+D+0.750L+0.750S+H, LL Comb I												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
+D+0.750L+0.750S+H, LL Comb I												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
+D+0.750L+0.750S+H, LL Comb I												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
+D+0.60W+H												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
+D+0.70E+H												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
+D+0.750Lr+0.750L+0.450W+H, I												
Dsgn. L = 10.00 ft	1	0.089	0.027	0.11	-1.05	1.05	19.54	11.70 1.00 1.00		0.74	46.15	27.63
Dsgn. L = 10.00 ft	2	0.095	0.027	1.12	-1.05	1.12	19.54	11.70 1.00 1.00		0.74	46.15	27.63
+D+0.750Lr+0.750L+0.450W+H, I												
Dsgn. L = 10.00 ft	1	0.067	0.017	0.66	-0.79	0.79	19.54	11.70 1.00 1.00		0.48	46.15	27.63
Dsgn. L = 10.00 ft	2	0.067	0.011	0.24	-0.79	0.79	19.54	11.70 1.00 1.00		0.30	46.15	27.63
+D+0.750Lr+0.750L+0.450W+H, I												
Dsgn. L = 10.00 ft	1	0.111	0.028	0.46	-1.30	1.30	19.54	11.70 1.00 1.00		0.77	46.15	27.63
Dsgn. L = 10.00 ft	2	0.111	0.028	1.01	-1.30	1.30	19.54	11.70 1.00 1.00		0.77	46.15	27.63
+D+0.750L+0.750S+0.450W+H, L												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
+D+0.750L+0.750S+0.450W+H, L												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
+D+0.750L+0.750S+0.450W+H, L												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
+D+0.750L+0.750S+0.5250E+H, I												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
+D+0.750L+0.750S+0.5250E+H, I												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
+D+0.750L+0.750S+0.5250E+H, I												
Dsgn. L = 10.00 ft	1	0.045	0.010	0.27	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
Dsgn. L = 10.00 ft	2	0.045	0.010	0.33	-0.53	0.53	19.54	11.70 1.00 1.00		0.28	46.15	27.63
+0.60D+0.60W+0.60H												
Dsgn. L = 10.00 ft	1	0.027	0.006	0.16	-0.32	0.32	19.54	11.70 1.00 1.00		0.17	46.15	27.63
Dsgn. L = 10.00 ft	2	0.027	0.006	0.20	-0.32	0.32	19.54	11.70 1.00 1.00		0.17	46.15	27.63
+0.60D+0.70E+0.60H												
Dsgn. L = 10.00 ft	1	0.027	0.006	0.16	-0.32	0.32	19.54	11.70 1.00 1.00		0.17	46.15	27.63
Dsgn. L = 10.00 ft	2	0.027	0.006	0.20	-0.32	0.32	19.54	11.70 1.00 1.00		0.17	46.15	27.63

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Project Title: TRASH ENCLOSURE
 Engineer:
 Project ID: 85-25 Water Structure
 Project Descr:

Steel Beam

Project File: 85-25 TRASH ENCLOSURE.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

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DESCRIPTION: RB1

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr+H	1	0.0535	4.560	Lr Only	-0.0268	7.840
+D+Lr+H	2	0.0973	5.400		0.0000	7.840

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	0.386	1.562	0.654
Max Upward from Load Combinations	0.386	1.562	0.654
Max Upward from Load Cases	0.241	1.031	0.481
Max Downward from all Load Conditions (Resis	-0.069		-0.034
Max Downward from Load Cases (Resisting Up	-0.069		-0.034
+D+H	0.145	0.530	0.173
+D+L+H, LL Comb Run (*L)	0.145	0.530	0.173
+D+L+H, LL Comb Run (L*)	0.145	0.530	0.173
+D+L+H, LL Comb Run (LL)	0.145	0.530	0.173
+D+Lr+H, LL Comb Run (*L)	0.077	1.218	0.654
+D+Lr+H, LL Comb Run (L*)	0.386	0.874	0.138
+D+Lr+H, LL Comb Run (LL)	0.317	1.562	0.620
+D+S+H	0.145	0.530	0.173
+D+0.750Lr+0.750L+H, LL Comb Run (*L)	0.094	1.046	0.534
+D+0.750Lr+0.750L+H, LL Comb Run (L*)	0.326	0.788	0.147
+D+0.750Lr+0.750L+H, LL Comb Run (LL)	0.274	1.304	0.508
+D+0.750L+0.750S+H, LL Comb Run (*L)	0.145	0.530	0.173
+D+0.750L+0.750S+H, LL Comb Run (L*)	0.145	0.530	0.173
+D+0.750L+0.750S+H, LL Comb Run (LL)	0.145	0.530	0.173
+D+0.60W+H	0.145	0.530	0.173
+D+0.70E+H	0.145	0.530	0.173
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run	0.094	1.046	0.534
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run	0.326	0.788	0.147
+D+0.750Lr+0.750L+0.450W+H, LL Comb Run	0.274	1.304	0.508
+D+0.750L+0.750S+0.450W+H, LL Comb Run	0.145	0.530	0.173
+D+0.750L+0.750S+0.450W+H, LL Comb Run	0.145	0.530	0.173
+D+0.750L+0.750S+0.450W+H, LL Comb Run	0.145	0.530	0.173
+D+0.750L+0.750S+0.5250E+H, LL Comb Rur	0.145	0.530	0.173
+D+0.750L+0.750S+0.5250E+H, LL Comb Rur	0.145	0.530	0.173
+D+0.750L+0.750S+0.5250E+H, LL Comb Rur	0.145	0.530	0.173
+0.60D+0.60W+0.60H	0.087	0.318	0.104
+0.60D+0.70E+0.60H	0.087	0.318	0.104
D Only	0.145	0.530	0.173
Lr Only, LL Comb Run (*L)	-0.069	0.687	0.481
Lr Only, LL Comb Run (L*)	0.241	0.344	-0.034
Lr Only, LL Comb Run (LL)	0.172	1.031	0.447
H Only			

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Project Title: **TRASH ENCLOSURE**
 Engineer:
 Project ID: **85-25 Water Structure**
 Project Descr:

Steel Beam

Project File: 85-25 TRASH ENCLOSURE.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

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DESCRIPTION: **RB2**

CODE REFERENCES

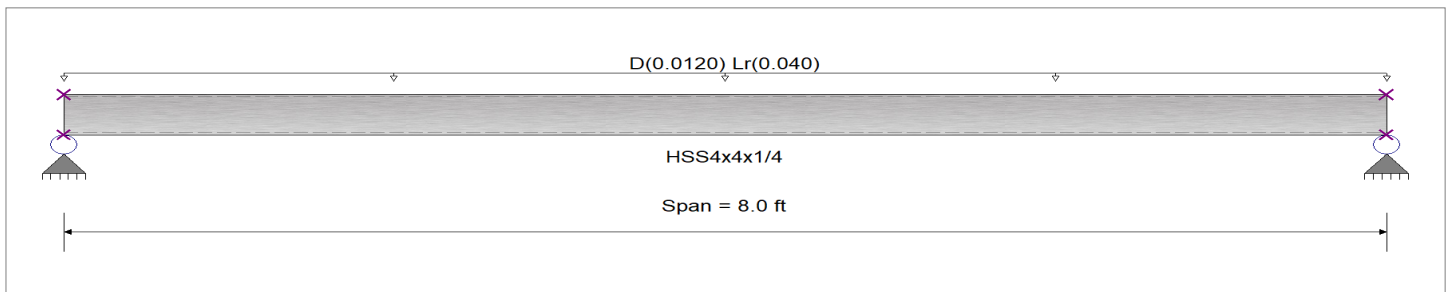
Calculations per AISC 360-16, IBC 2021, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

Analysis Method : Allowable Strength Design
 Beam Bracing : Completely Unbraced
 Bending Axis : Major Axis Bending

Fy : Steel Yield : 50.0 ksi
 E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.0060, Lr = 0.020 ksf, Tributary Width = 2.0 ft, (RR)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.044 : 1	Maximum Shear Stress Ratio =		0.009 : 1
Section used for this span		HSS4x4x1/4	Section used for this span		HSS4x4x1/4
Ma : Applied		0.513 k-ft	Va : Applied		0.2567 k
Mn / Omega : Allowable		11.702 k-ft	Vn/Omega : Allowable		27.634 k
Load Combination		+D+Lr	Load Combination		+D+Lr
Span # where maximum occurs		Span # 1	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		0.016 in	Ratio =	5,863	>=360. Span: 1 : Lr Only
Max Upward Transient Deflection		0 in	Ratio =	0	<360.0 n/a
Max Downward Total Deflection		0.026 in	Ratio =	3655	>=240. Span: 1 : +D+Lr
Max Upward Total Deflection		0 in	Ratio =	0	<240.0 n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 8.00 ft	1	0.017	0.004	0.19		0.19	19.54	11.70	1.14	1.00	0.10	46.15	27.63
+D+Lr													
Dsgn. L = 8.00 ft	1	0.044	0.009	0.51		0.51	19.54	11.70	1.14	1.00	0.26	46.15	27.63
+D+0.750Lr													
Dsgn. L = 8.00 ft	1	0.037	0.008	0.43		0.43	19.54	11.70	1.14	1.00	0.22	46.15	27.63
+0.60D													
Dsgn. L = 8.00 ft	1	0.010	0.002	0.12		0.12	19.54	11.70	1.14	1.00	0.06	46.15	27.63

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0263	4.023		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.257	0.257
Max Upward from Load Combinations	0.257	0.257
Max Upward from Load Cases	0.160	0.160
D Only	0.097	0.097
+D+Lr	0.257	0.257
+D+0.750Lr	0.217	0.217
+0.60D	0.058	0.058

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Engineer:
Project ID: 85-25 Water Structure
Project Descr:

Steel Beam

Project File: 85-25 TRASH ENCLOSURE.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

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DESCRIPTION: RB2

Vertical Reactions		Support notation : Far left is #1		Values in KIPS
Load Combination		Support 1	Support 2	
Lr Only		0.160	0.160	

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 Project Descr:

Steel Column

Project File: 85-25 TRASH ENCLOSURE.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

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DESCRIPTION: Steel Column

Code References

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
 Load Combinations Used : IBC 2021

General Information

Steel Section Name : HSS4x4x1/4	Overall Column Height	8.0 ft
Analysis Method : Allowable Strength	Top & Bottom Fixity	Top Free, Bottom Fixed
Steel Stress Grade	Brace condition :	
Fy : Steel Yield 36.0 ksi	Unbraced Length for buckling ABOUT X-X Axis = 8.0 ft, K = 2.4	
E : Elastic Bending Modulus 29,000.0 ksi	Unbraced Length for buckling ABOUT Y-Y Axis = 8.0 ft, K = 2.4	

Applied Loads

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

Column self weight included : 97.446 lbs * Dead Load Factor

AXIAL LOADS . . .

Roof Load: Axial Load at 8.0 ft, D = 0.530, LR = 1.030 k

BENDING LOADS . . .

Moment acting about X-X axis at 8.0 ft, D = 2.0 k-ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.2750 : 1	Maximum Load Reactions . .	
Load Combination	+D+Lr	Top along X-X	0.0 k
Location of max.above base	0.0 ft	Bottom along X-X	0.0 k
At maximum location values are . . .		Top along Y-Y	0.0 k
Pa : Axial	1.657 k	Bottom along Y-Y	0.0 k
Pn / Omega : Allowable	22.046 k	Maximum Load Deflections . . .	
Ma-x : Applied	2.0 k-ft	Along Y-Y -0.4856 in at	8.0ft above base
Mn-x / Omega : Allowable	8.425 k-ft	for load combination : D Only	
Ma-y : Applied	0.0 k-ft	Along X-X 0.0 in at	0.0ft above base
Mn-y / Omega : Allowable	8.425 k-ft	for load combination :	
PASS Maximum Shear Stress Ratio	0.0 : 1		
Load Combination	0.0		
Location of max.above base	0.0 ft		
At maximum location values are . . .			
Va : Applied	0.0 k		
Vn / Omega : Allowable	0.0 k		

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios								Maximum Shear Ratios			
	Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx		Stress Ratio	Status	Location	
D Only	0.252	PASS	0.00 ft	1.00	1.00	151.58	151.58		0.000	PASS	0.00 ft	
+D+Lr	0.275	PASS	0.00 ft	1.00	1.00	151.58	151.58		0.000	PASS	0.00 ft	
+D+0.750Lr	0.269	PASS	0.00 ft	1.00	1.00	151.58	151.58		0.000	PASS	0.00 ft	
+0.60D	0.151	PASS	0.00 ft	1.00	1.00	151.58	151.58		0.000	PASS	0.00 ft	

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction		X-X Axis Reaction		k	Y-Y Axis Reaction		Mx - End Moments		k-ft	My - End Moments	
	@ Base		@ Base	@ Top		@ Base	@ Top	@ Base	@ Top		@ Base	@ Top
D Only	0.627							2.000				
+D+Lr	1.657							2.000				
+D+0.750Lr	1.400							2.000				
+0.60D	0.376							1.200				
Lr Only	1.030											

Extreme Reactions

Item	Axial Reaction		X-X Axis Reaction		k	Y-Y Axis Reaction		Mx - End Moments		k-ft	My - End Moments	
	Extreme Value	@ Base	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top		@ Base	@ Top
Axial @ Base	Maximum	1.657						2.000				

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 Engineer:
 Project ID: 85-25 Water Structure
 Project Descr:

Steel Column

Project File: 85-25 TRASH ENCLOSURE.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

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DESCRIPTION: Steel Column

Extreme Reactions

Item	Extreme Value	Axial Reaction	X-X Axis Reaction		k	Y-Y Axis Reaction		Mx - End Moments		k-ft	My - End Moments	
		@ Base	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top		@ Base	@ Top
"	Minimum	0.376						1.200				
Reaction, X-X Axis Base	Maximum	0.627						2.000				
"	Minimum	0.627						2.000				
Reaction, Y-Y Axis Base	Maximum	0.627						2.000				
"	Minimum	0.627									2.000	
Reaction, X-X Axis Top	Maximum	0.627						2.000				
"	Minimum	0.627						2.000				
Reaction, Y-Y Axis Top	Maximum	0.627						2.000				
"	Minimum	0.627						2.000				
Moment, X-X Axis Base	Maximum	0.627	2.000					2.000				
"	Minimum	1.030										
Moment, Y-Y Axis Base	Maximum	0.627						2.000				
"	Minimum	0.627						2.000				
Moment, X-X Axis Top	Maximum	0.627						2.000				
"	Minimum	0.627						2.000				
Moment, Y-Y Axis Top	Maximum	0.627						2.000				
"	Minimum	0.627						2.000				

Maximum Deflections for Load Combinations

Load Combination	Max. Deflection in X dir	Distance	Max. Deflection in Y dir	Distance
D Only	0.0000 in	0.000 ft	-0.486 in	8.000 ft
+D+Lr	0.0000 in	0.000 ft	-0.486 in	8.000 ft
+D+0.750Lr	0.0000 in	0.000 ft	-0.486 in	8.000 ft
+0.60D	0.0000 in	0.000 ft	-0.291 in	8.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Steel Section Properties : HSS4x4x1/4

Depth	=	4.000 in	I xx	=	7.80 in^4	J	=	12.800 in^4
Design Thick	=	0.233 in	S xx	=	3.90 in^3			
Width	=	4.000 in	R xx	=	1.520 in			
Wall Thick	=	0.250 in	Zx	=	4.690 in^3			
Area	=	3.370 in^2	I yy	=	7.800 in^4	C	=	6.560 in^3
Weight	=	12.181 plf	S yy	=	3.900 in^3			
			R yy	=	1.520 in			
Ycg	=	0.000 in						

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Steel Column

Project File: 85-25 TRASH ENCLOSURE.ec6

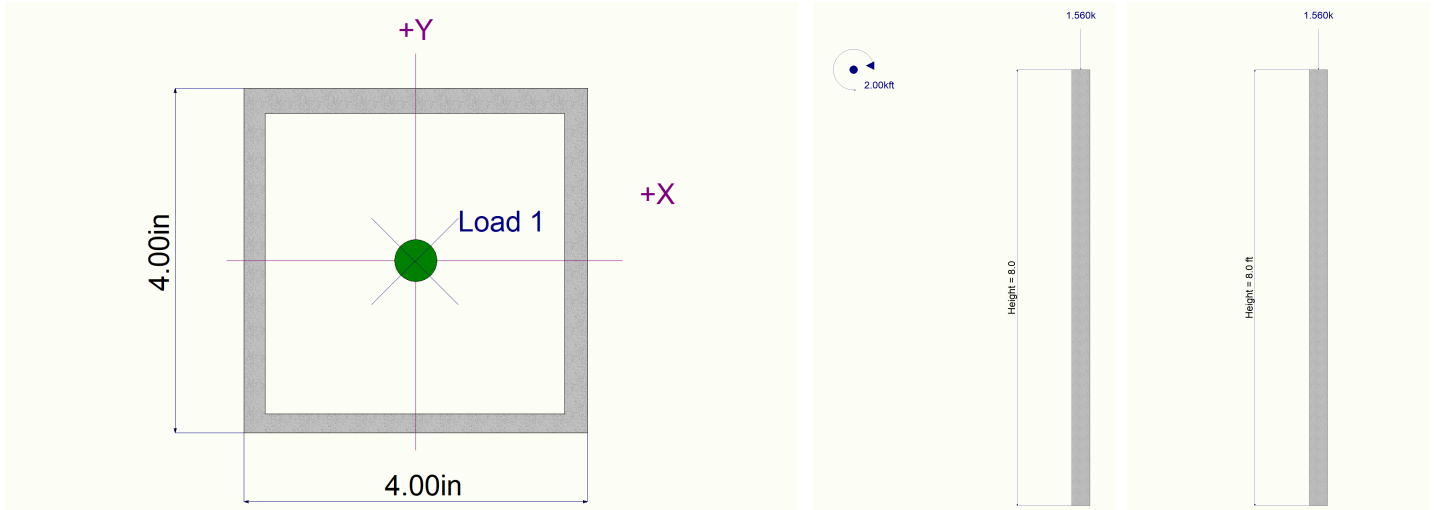
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DESCRIPTION: Steel Column

Sketches



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 Project Descr:

Cantilevered Retaining Wall

Project File: 85-25 TRASH ENCLOSURE.ec6

LIC# : KW-06018875, Build:20.23.08.30

FA STRUCTURAL

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DESCRIPTION: Wall

Code Reference:

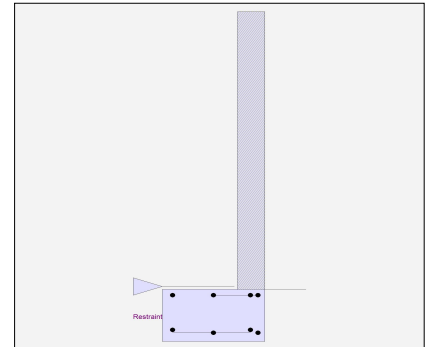
Calculations per IBC 2021 1807.3, ASCE 7-16

Criteria

Retained Height	=	0.00 ft
Wall height above soil	=	8.00 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	0.00 in
Water table above bottom of footing	=	0.0 ft

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	32.0 psf/ft
	=	
Passive Pressure	=	100.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
NOT Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
NOT Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	7.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Strength Level)
Wind on Exposed Stem	=	10.0 psf (Strength Level)

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type	=	Spread Footing
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

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FA STRUCTURAL

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DESCRIPTION: Wall

Design Summary

Wall Stability Ratios

Overturning	=	2.63	OK
Slab Resists All Sliding !			
Global Stability	=	1.82	
Total Bearing Load	=	578 lbs	
...resultant ecc.	=	5.13 in	
Eccentricity outside middle third			
Soil Pressure @ Toe	=	456 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	1,500 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	639 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	1.4 psi	OK
Footing Shear @ Heel	=	0.1 psi	OK
Allowable	=	75.0 psi	

Sliding Calcs

Lateral Sliding Force	=	84.0 lbs
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Vertical component of active lateral soil pressure IS considered in the calculation of soil bearing pressures.

Load Factors

Building Code	
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

Design Height Above Ftg

Wall Material Above "Ht"	=	Masonry
Design Method	=	ASD
Thickness	=	8.00
Rebar Size	=	# 4
Rebar Spacing	=	16.00
Rebar Placed at	=	Center

Design Data

fb/FB + fa/Fa	=	0.191
---------------	---	-------

Total Force @ Section

Service Level	lbs =	48.0
Strength Level	lbs =	

Moment.....Actual

Service Level	ft-# =	192.0
Strength Level	ft-# =	

Moment.....Allowable	=	1,004.1
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Shear.....Actual

Service Level	psi =	0.5
Strength Level	psi =	

Shear.....Allowable	psi =	43.6
---------------------	-------	------

Anet (Masonry)	in2 =	91.50
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Wall Weight	psf =	0.0
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Rebar Depth 'd'	in =	3.81
-----------------	------	------

Masonry Data

f'm	psi =	1,500
Fs	psi =	32,000
Solid Grouting	=	Yes
Modular Ratio 'n'	=	21.48
Equiv. Solid Thick.	in =	7.63
Masonry Block Type	=	
Masonry Design Method	=	ASD

Concrete Data

f'c	psi =	
Fy	psi =	

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DESCRIPTION: Wall

Footing Data

Toe Width	=	1.83 ft
Heel Width	=	0.67
Total Footing Width	=	2.50
Footing Thickness	=	18.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c =	2,500 psi	Fy = 60,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top	2.00	@ Btm.= 3.00 in

Footing Design Results

	Toe	Heel
Factored Pressure	= 639	0 psf
Mu' : Upward	= 805	0 ft-#
Mu' : Downward	= 534	0 ft-#
Mu: Design	= 270 OK	0 ft-# OK
phiMn	= 29,589	OK - Flush
Actual 1-Way Shear	= 1.38	0.14 psi
Allow 1-Way Shear	= 75.00	75.00 psi
Toe Reinforcing	= # 5 @ 8.00 in	
Heel Reinforcing	=	Flush heel condition. No reinforcing required.
Key Reinforcing	=	None Spec'd
Footing Torsion, Tu	=	0.00 ft-lbs
Footing Allow. Torsion, phi Tu	=	0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: #4@ 6.17 in, #5@ 9.56 in, #6@ 13.58 in, #7@ 18.51 in, #8@ 24.38 in, #9@ 30.86 in, #10@ 39.19 in

Heel: Flush heel condition. No reinforcing required.

Key: No key defined

Min footing T&S reinf Area	0.97	in2
Min footing T&S reinf Area per foot	0.39	in2 /ft
<u>If one layer of horizontal bars:</u>		<u>If two layers of horizontal bars:</u>
#4@ 6.17 in		#4@ 12.35 in
#5@ 9.57 in		#5@ 19.14 in
#6@ 13.58 in		#6@ 27.16 in

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FA STRUCTURAL

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DESCRIPTION: Wall

Rebar Lap & Embedment Lengths Information

Stem Design Segment: Bottom

Stem Design Height: 0.00 ft above top of footing

Calculated Rebar Stress, f_s = 4495.54 psi

Lap Splice length for #4 bar specified in this stem design segment (25.4.2.4a) = 20.00 in

Development length for #4 bar specified in this stem design segment = 12.00 in

Hooked embedment length into footing for #4 bar specified in this stem design segment = 0.00 in

As Provided = 0.1500 in²/ft

As Required = 0.0210 in²/ft

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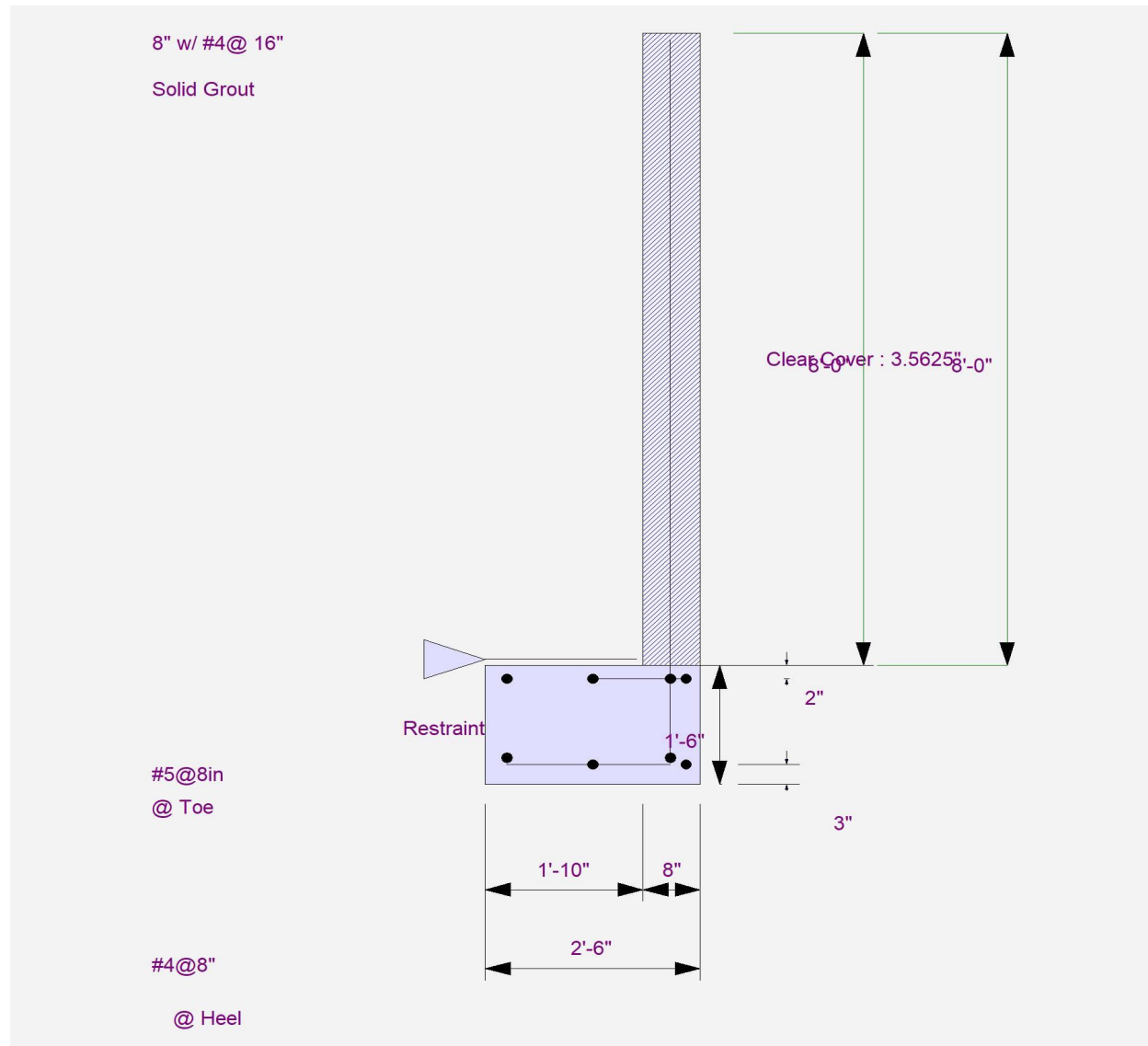
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