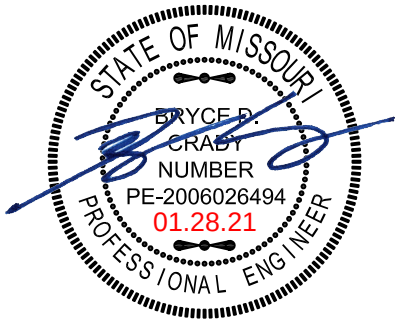
APEX
ENGINEERS, INC.

Structural Design Calculations

For

Summit View Farms Pool house
2919 SW Arthur Dr.
Lee's Summit, Missouri



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2003004673

January 28, 2021

Elswood Smith Carlson Architects

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Design Loads

Project:	Summit View Farms Pool House
Client:	Elswood Smith Carlson Architects, PA
Engineer:	Garrett Pieper, EIT
Date:	1/15/2021
Description:	Project Information Page

PROJECT INFORMATION PAGE

Client

Name	Elswood Smith Carlson Architects, PA
Address	7133 w 95th Street
	Overland Park, KS 66212
Phone	913-649-7557
Email	sue@escarchitects.com

Project Name and Address

Name	Summit View Farms Pool House
Address	2919 SW Arthur Dr
	Lee's Summit, MO 64082

General Information

Building Code	2018 International Building Code
Bldg. Dept.	Commercial
Notes	

Project:	Summit View Farms Pool House
Client:	Elswood Smith Carlson Architects, PA
Engineer:	Garrett Pieper, EIT
Date:	1/15/2021
Description:	Project Soils Information

Project Soils Information

Foundation Overview

The foundation design provided is based off a minimum allowable presumptive load-bearing value as indicated by 2018 International Building Code in lieu of a site geotechnical evaluation.

Type	Shallow Foundations
Sulfate Levels	Negligible Sulfate
Frost Depth	36 in
Site Class	D

Foundation Design Information

Shallow Foundation

Isolated Footing Pressure:	1500 psf
Minimum Pad Dimension:	24 in
Continuous Footing Pressure:	1500 psf
Minimum Wall Footing Width:	12 in
Minimum Bearing Pressure:	0 psf

Horizontal Soil Design

Active Pressure:	0 psf/ft
At-Rest Pressure:	0 psf/ft
Passive Pressure:	0 psf/ft
Lateral Sliding Resistance Means:	Coefficient of friction
Lateral Sliding Resistance μ :	0.35

Project:	Summit View Farms Pool House
Client:	Elswood Smith Carlson Architects, PA
Engineer:	Garrett Pieper, EIT
Date:	1/15/2021
Description:	Dead Load Development

Roof Dead Load Calculations

Description	**Typical Roof**		
Roofing Materials	Composition Shingle	3.40 psf	
Sheathing	1/2" Plywood	1.50 psf	
Framing	Wood Trusses	3.80 psf	<-- Equivalent to (2) 2x4 @ 24" o.c.
Insulation	Blow-in Insulation	1.50 psf	
Ceiling Finish	1/2" Gyp Board	2.20 psf	
Sprinklers	No	0.00 psf	
Mechanical	Yes	4.00 psf	
Collateral	No	0.00 psf	
Miscellaneous		0.00 psf	
Total DL:		16.40 psf	
Roof Pitch:	6.00 : 12		
Adjusted for Roof Slope:		19 psf	

Wall Dead Load Calculations

Description	**Typ Exterior**		
Exterior Finish	1" Plaster/Stucco	8.00 psf	
Wall Sheathing	1/2" Plywood	1.50 psf	
Wall Framing	Wood Framing	1.70 psf	<-- Equivalent to 2x6 @ 16" o.c.
Insulation	R-11 Batt Insulation (4")	0.50 psf	
Interior Finish	5/8" Gyp. Board	2.80 psf	
Miscellaneous		0.00 psf	
Total DL:		15 psf	

APEX ENGINEERS INC	Project:	Summit View Farms Pool House
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	Date:	1/15/2021
	Description:	Live Load Tables

Occupancy or Use	Uniform, L_o (psf)	Live Load Reduction Permitted?	Multiple-Story Live Load Reduction Permitted?	Concentrated (lb)
Residential - Multi-Family Dwellings				
Private rooms and corridors serving them	40	Yes	Yes	
Public rooms	100	No	No	
Corridors serving public rooms	100	Yes	Yes	
Stairs and exit ways	100	Yes	Yes	300
Fire escapes	100	Yes	Yes	
Roofs				
Roof: Ordinary flat, pitched, and curved	20	Yes	—	
Roof areas used for occupants	Same as occupancy served	Yes	—	
Roof areas used for assembly purposes	100	Yes		
Vegetative and landscaped roofs				
Roof areas not intended for occupancy	20	Yes	—	
Roof areas used for assembly purposes	100	Yes	—	
Roof areas used for other occupancies	Same as occupancy served	Yes	—	
Roof fabric awnings and canopies supported by a skeleton structure	5	No	—	
Roof screen enclosure support frame	5 based on	No	—	200
Roof: All other construction	20	Yes	—	
Primary roof members, exposed to a work floor				
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages				2,000
All other primary roof members		—	—	300
All roof surfaces subject to maintenance workers		—	—	300

APEX ENGINEERS INC	Project:	Summit View Farms Pool House
	Client:	Elswood Smith Carlson Architects, PA
	Engineer:	Garrett Pieper, EIT
	Date:	1/15/2021
	Description:	Snow Load Development - Main Building

SNOW DESIGN LOADS - ASCE 7-16, CHAPTER 7
Building 1

General Information

Risk Category =	II	*(Table 1.5-1)*
Importance Factor, I_s =	1.00	*(Table 1.5-2)*
Ground Snow Load, p_g =	20 psf	*(Section 7.2)*
Exposure Classification =	C	*(Section 26.7)*
Snow Density, $\gamma = 0.13p_g + 14$ =	17 pcf	*(Eq. 7.7-1)*

Building Information

Exposure to Roof =	Fully Exposed	*(Table 7.3-1)*
Thermal Condition =	Unheated/Open-Air Structures	*(Section 7.4)*
Roof Slope =	6.00 : 12	
Roof Shape =	Hip	
Roof Surface =	Composition Shingle	
Surface Classification =	All Other Surfaces	*(Section 7.4)*
Roof Obstructions=	No	*(Section 7.4)*

Snow Loads

Exposure Factor, C_e =	0.90	*(Table 7.3-1)*
Thermal Factor, C_t =	1.20	*(Table 7.3-2)*
Flat Roof Snow Load, $p_f = 0.7*C_e*C_t*I_s*p_g$ =	15.1 psf	*(Eq. 7.3-1)*
Roof Angle =	26.57 °	
Slope Factor, C_s =	1.00	*(Figure 7.4-1)*
Sloped Surface Roof Snow Load, $p_s = C_s*p_f$	15.1 psf	*(Eq. 7.4-1)*

Minimum Snow For Low-Slope Roofs

Min. Snow For Low-Slope Roofs, p_m =	N/A	*(Section 7.3.4)*

Rain on Snow Surcharge

For locations where pg ≤ 20 psf, all roofs with slopes (in degrees) less than W/50 with W in ft shall include 5 psf rain-on-snow surcharge load.

Ridge to Eave Width, W =	13.0 ft	
Roof Angle for Surcharge Check, W/50 =	0.26 °	
Rain-on-Snow Surcharge Load =	N/A	*(Section 7.10)*

APEX ENGINEERS INC	Project:	Summit View Farms Pool House
	Client:	Elswood Smith Carlson Architects, PA
	Engineer:	Garrett Pieper, EIT
	Date:	1/15/2021
	Description:	Snow Load Development - Main Building

Design Case: Unbalanced Snow Loads - Drift Load

Height of Balanced Snow, h_b = 0.91 ft *(Section 7.7)*

Number of Snow Drifts: 1

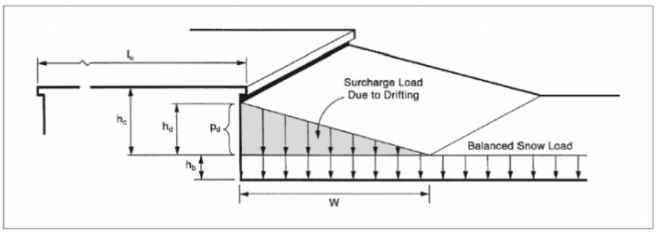


FIGURE 7-8 Configuration of Snow Drifts on Lower Roofs.

Note: Snow drifts are not required when:

- hc/hb < 0.2 (I.E. h < 1.2hb) [Section 7.7.1]
- Roof Projection ≤ 15' [Section 7.8]

$h_{d,\ Leeward} = 0.43 l_{u,lw}^{1/3}(p_g + 10)^{1/4} - 1.5$

$h_{d,\ windward} = 0.75*(0.43 l_{u,ww}^{1/3}(p_g + 10)^{1/4} - 1.5)$

Description	Condition	Upper Fetch	Lower Fetch	Projected Height	Hd	Drift Width	Drift Load	Total Load
Type 1	Low Roof	38.8 ft	22.1 ft	3.0 ft	1.91 ft	7.63 ft	31.7 psf	46.8 psf

Project:	Summit View Farms Pool House
Client:	Elswood Smith Carlson Architects, PA
Engineer:	Garrett Pieper, EIT
Date:	1/15/2021
Description:	Seismic Mass Development - Main Building

SEISMIC MASS
Building 1

Floor Dead Load Contributions

Level	Level Area	Int. Wall Density	Level Type	Dead Load	Snow (1)	Total Weight
Base	1537 ft²	None	Typical Floor	13 psf	0 psf	20 k
Roof	1537 ft²	None	Typical Roof	19 psf	0 psf	29 k

(1) 20% of Flat Roof Snow Loads above 30 PSF

Storage/Equipment Load Contributions

Level	Storage	Storage Load	Storage Area	Equipment	Total Weight
Base	None	31.3 psf	0 ft²		0.00 k
Roof	None	31.3 psf	0 ft²		0.00 k

Wall Load Contributions

Level	Wall Trib	Wall Type	Wall Length	Wall Weight	Total Weight
Base	6.54 ft	Typ Exterior	202 ft	15.0 psf	20 k
Roof	6.54 ft	Typ Exterior	202 ft	15.0 psf	20 k

Total Level Loads

Level	Floor Weight	Storage Weight	Wall Weight	Total Weight
Roof	29.2 k	0.0 k	19.8 k	49.0 k

Project:	Summit View Farms Pool House
Client:	Elswood Smith Carlson Architects, PA
Engineer:	Garrett Pieper, EIT
Date:	1/15/2021
Description:	Sesimic Load Development - Cover Page

SEISMIC DESIGN LOADS - ASCE 7-16, CHAPTER 12
EQUIVALENT LATERAL FORCE PROCEDURE

Site Classifciation

Site Classifciation =	D	*(Section 11.4.3)*

Spectral Response Acceleration Parameter

Short Period, S_S =	0.100 g	*(Section 11.4.2)*
Coefficient at 0.2s, F_a =	1.60	*(Table 11.4-1)*
1s Period, S_1 =	0.068 g	*(Section 11.4.2)*
Coefficient at 1s, F_v =	2.40	*(Table 11.4-2)*

Maximum Spectral Response Accelerations

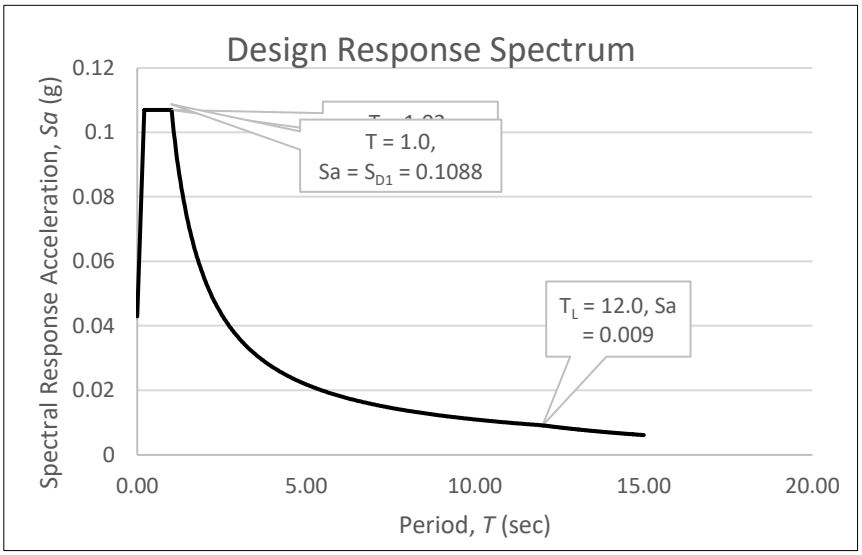
Short Period, S_{MS} = Fa * SS =	0.160 g	*(Eq. 11.4-1)*
1s Period, S_{M1} = Fv * S1 =	0.163 g	*(Eq. 11.4-2)*

Design Spectral Response Accelerations

Short Period, S_{DS} = (2/3) * SMS =	0.107 g	*(Eq. 11.4-3)*
1s Period, S_{D1} = (2/3) * SM1 =	0.109 g	*(Eq. 11.4-4)*

Long-Period Transition Period

Long-Period Transition Period, T_L=	12	*(Section 11.4.6)*



APEX ENGINEERS INC	Project:	Summit View Farms Pool House
	Client:	Elswood Smith Carlson Architects, PA
	Engineer:	Garrett Pieper, EIT
	Date:	1/15/2021
	Description:	Sesimic Load Development - Main Building

SEISMIC DESIGN INFORMATION
Building 1

Seismic Design Category

Risk Category =	II	*(Table 1.5-1)*
Imprtance Factor, I_e =	1.00	*(Table 1.5-2)*
Seismic Design Category =	B	*(Section 11.6)*

Building Parameters

Buiding Height, H =	15.08 ft	
Structure Type =	All other structural systems	*(Table 12.8-2)*
Approx. Fundamental Period, $T_a = C_t h_n^x$	0.153	*(Eq. 12.8-7)*
Ct =	0.02	*(Table 12.8-2)*
x =	0.75	*(Table 12.8-2)*
Vertical Distribution Exponent, k =	1.00	*(Section 12.8.3)*

Lateral Resisting System

A. BEARING WALL SYSTEMS

15. LIGHT-FRAME (WOOD) WALLS SHEATHED WITH WOOD STRUCTURAL PANELS RATED FOR SHEAR RESISTANCE OR STEEL SHEETS

Response Modification Coefficient, R =	6.5	*(Table 12.2-1)*
Deflection Amplification Factor, C_d =	4	*(Table 12.2-1)*
Overstrength Factor, Ω_o =	3	*(Table 12.2-1)*

Seismic Loads

Cs, Min =	0.0100	*(Section 12.8.1.1)*
Cs, Max =	0.1093	*(Section 12.8.1.1)*
Seismic Response Coefficient, C_s =	0.016	*(Section 12.8.1.1)*
Effective Seismic Weight, W =	49 k	*(Section 12.7.2)*
N/S Seismic Base Shear, $V_{N/S} = C_s W$ =	0.80 k	*(Eq. 12.8-1)*

		Vertical Distribution of Seismic Load, Fx				Diaphragm Load, Fp,x		
Level	**Weight (k)**	**Height (ft)**	**wxhx^k**	**Cv,x**	**Fx**	**Σw**	**ΣF**	**Fp,x**
Roof	49.0 k	13.1 ft	641.4	1.00	0.8 k	49.0 k	0.8 k	1.0 k
Sum	49.0 k		641.4					

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	Description:	Sesimic Load Development - Main Building

East/West Lateral Resisting System

A. BEARING WALL SYSTEMS

15. LIGHT-FRAME (WOOD) WALLS SHEATHED WITH WOOD STRUCTURAL PANELS RATED FOR SHEAR RESISTANCE OR STEEL SHEETS

Response Modification Coefficient, R =	6.5	*(Table 12.2-1)*
Deflection Amplification Factor, Cd =	4	*(Table 12.2-1)*
Overstrength Factor, Ωo =	3	*(Table 12.2-1)*

East/West Seismic Loads

Cs, Min =	0.0100	*(Section 12.8.1.1)*
Cs, Max =	0.1093	*(Section 12.8.1.1)*
Seismic Response Coefficient, Cs =	0.0164	*(Section 12.8.1.1)*
Effective Seismic Weight, W =	49 k	*(Section 12.7.2)*
E/W Seismic Base Shear, VE/W = CsW =	0.80 k	*(Eq. 12.8-1)*

			Vertical Distribution of Seismic Load, Fx			Diaphragm Load, Fp,x		
Level	**Weight (k)**	**Height (ft)**	**wxhx^k**	**Cv,x**	**Fx**	**Σw**	**ΣF**	**Fp,x**
Roof	49.0 k	13.1 ft	641.4	1.00	0.8 k	49.0 k	0.8 k	1.0 k
Sum	49.0 k		641.4					

APEX ENGINEERS INC	Project:	Summit View Farms Pool House
	Client:	Elswood Smith Carlson Architects, PA
	Engineer:	Garrett Pieper, EIT
	Date:	1/15/2021
	Description:	Wind Load Development - Directional Procedure - Main Building

WIND DESIGN LOADS - ASCE 7-16, CHAPTER 27 (MWFRS)
DIRECTIONAL PROCEDURE - PART 1, ALL HEIGHTS METHOD
Building 1

Wind Load Site Parameters

Risk Category =	II	
Exposure Category =	C	*(Section 26.7.3)*
Topographic Factor, K_{zt} =	1.00	*(Section 26.8.2)*
Elevation Reduction Factor, K_e =	1.00	*(Table 26.9-1)*

Wind Load Building Parameters

Structure Type =	Buildings	
Directionality factor, K_d =	0.85	*(Table 26.6-1)*
Gust Factor, G =	0.85	
Enclosure Classification =	Enclosed Buildings	
Internal Pressure Coefficient, GC_{pi} =	± 0.18	*(Table 26.13-1)*
Ch. 27, Minimum Wall Pressure, Pmin. =	16 psf	*(Section 27.1.5)*
Basic Wind Speed, V =	110 mph	*(Section 26.5.1)*

Region Parameters	**Region 1**	
Region Length, L =	53.00 ft	
Region Width, W =	29.00 ft	
Mean Roof Height, H =	15.08 ft	
Roof Angle =	26.57 °	
Edge Strip, a =	3.00 ft	*(Figure 28.3-1)*
Velocity Pressure Exposure Coefficient, K_z =	0.85	
Velocity Pressure, $q_h = 0.00256\ K_e K_z K_{zt} K_d V^2$ =	22 psf	
Building Length / Building Width, L/B =	1.83	
Building Height/Length, h/L =	0.28	
Building Height/Width, h/W =	0.52	

APEX ENGINEERS INC	Project:	Summit View Farms Pool House
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	Date:	1/15/2021
	Description:	Wind Load Development - Directional Procedure - Main Building

Wind Loads: Main Wind Force Resisting System

Pressure, p=qGCp ± qi(GCpi)

Region 1

Walls

Surface	Cp	qGCp	± qi(GCpi)
Windward	0.80	Ref Table	4 psf
Leeward L/W	-0.33	-6.4 psf	4 psf
Leeward W/L	-0.50	-9.5 psf	4 psf
Side	-0.70	-13.3 psf	4 psf

Hip 26.57 °

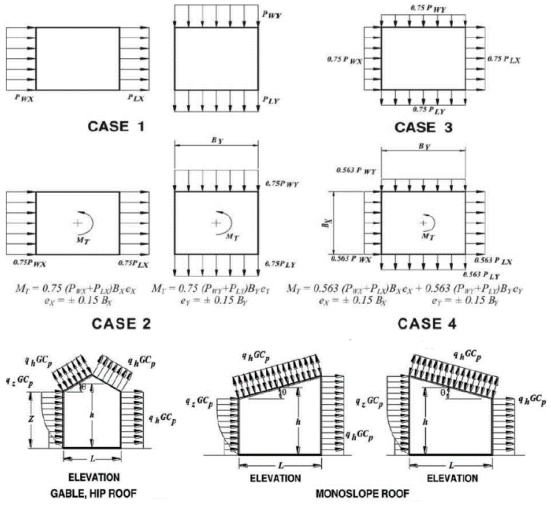
Roof Pressures along Length

Surface	Cp	Pext
Windward Case 1	-0.21	-8.0 psf
Windward Case 2	0.29	1.4 psf
Leeward	-0.60	-15.4 psf

Roof Pressures along Width

Surface	Cp Case 1	Cp Case 2	Pext Case 1	Pext Case 2
0 to h/2	-0.92	-0.18	-21.5 psf	0.6 psf
h/2 to h	-0.89	-0.18	-21.0 psf	0.6 psf
h to 2h	-0.51	-0.18	-13.7 psf	0.6 psf
>2h	-0.32	-0.18	-10.0 psf	0.6 psf

Region 1 Load Application Cases



Region 1

Building Parameters			Windward Wall Pressures			Leeward Wall	Combined Wall	
Level	**Height**	**Wall Trib**	**Kz**	**qz**	**Pext**	**Pext**	**Pressure**	**Uniform Load**
Base	0.00 ft	6.54 ft	0.85	22.4 psf	17.9 psf	-9.5 psf	27.4 psf	180 plf
Roof	13.08 ft	6.54 ft	0.85	22.4 psf	17.9 psf	-9.5 psf	27.4 psf	180 plf
Roof Proj.	17.08 ft	4.00 ft	0.87	23.0 psf	0.6 psf	-6.9 psf	7.5 psf	31 plf

TOTAL ROOF DIAPHRAGM LOAD = 211 plf

APEX ENGINEERS INC	Project:	Summit View Farms Pool House
	Client:	Elswood Smith Carlson Architects, PA
	Engineer:	Garrett Pieper, EIT
	Date:	1/25/2021
	Description:	Wind Load Development - Componenets and Cladding - Main Building

WIND DESIGN LOADS - ASCE 7-16, CHAPTER 30
COMPONENTS AND CLADDING - PART 1, H ≤ 60 FT
Building 1

Site Parameters

Risk Category =	II	
Exposure Category =	C	*(Section 26.7.3)*
Topographic Factor, K_{zt} =	1.00	*(Section 26.8.2)*
Elevation Reduction Factor, K_e =	1.00	*(Table 26.9-1)*

Building Parameters

Structure Type =	Buildings	
Directionality factor, K_d =	0.85	*(Table 26.6-1)*
Enclosure Classification =	Enclosed Buildings	
Internal Pressure Coefficient, GC_{pi} =	0.18	*(Table 26.13-1)*
Ch. 28, Minimum Wall Pressure, Pmin. =	16 psf	*(Section 28.3.4)*
Basic Wind Speed, V =	110 mph	*(Section 26.5.1)*

Region Parameters

Region Length, L =	53.00 ft	
Region Width, W =	29.00 ft	
Mean Roof Height, H =	15.08 ft	
Roof Angle =	26.57 °	
Edge Strip, a =	3.00 ft	*(Figure 28.3-1)*
Velocity Pressure Exposure Coefficient, K_z =	0.85	
Velocity Pressure, $q_h = 0.00256 K_e K_z K_{zt} K_d V^2$ =	22 psf	

APEX ENGINEERS INC	Project:	Summit View Farms Pool House
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	Date:	1/25/2021
	Description:	Wind Load Development - Componenets and Cladding - Main Building

Wall Design Pressures for Components and Cladding

$p = q_h[(GC_p) - (GC_{pi})]$

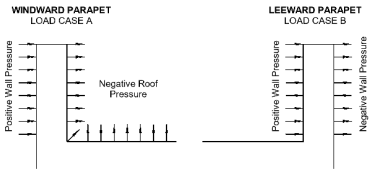
Zones	10 ft²	20 ft²	50 ft²	100 ft²	200 ft²
4 & 5	26 psf	25 psf	24 psf	22 psf	21 psf
4	-29 psf	-27 psf	-26 psf	-25 psf	-24 psf
5	-35 psf	-33 psf	-30 psf	-27 psf	-25 psf

Parapet Design Pressures for Components and Cladding - Part 6

Velocity Pressure Exposure Coefficient, K_z = 1.00

Velocity Pressure, $q_h = 0.00256\ K_e K_z K_{zt} K_d V^2$ = 22 psf

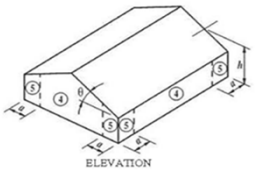
Zones	10 ft²	20 ft²	50 ft²	100 ft²	200 ft²
4 PAR - A	74 psf	69 psf	63 psf	58 psf	53 psf
5 PAR - A	94 psf	86 psf	75 psf	66 psf	58 psf
4 PAR - B	47 psf	45 psf	41 psf	39 psf	37 psf
5 PAR - B	54 psf	50 psf	45 psf	42 psf	38 psf



Load Case A shall consist of applying the applicable positive wall pressure to the front surface of the parapet while applying the applicable negative edge or corner zone roof pressure to the back surface.

Load Case B shall consist of applying the applicable positive wall pressure to the back of the parapet surface and applying the applicable negative wall pressure to the front surface.

Diagram



Notation

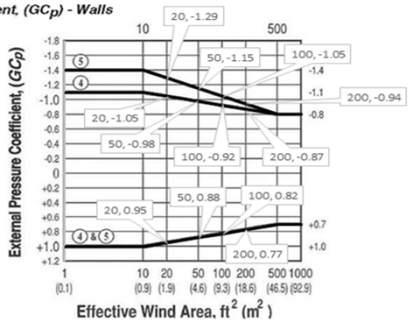
a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).

Exception: For buildings with $\theta = 0°$ to 7° and a least horizontal dimension greater than 300 ft (90 m), dimension a shall be limited to a maximum of $0.8h$.

h = Mean roof height, in ft (m), except that eave height shall be used for $\theta \le 10°$.

θ = Angle of plane of roof from horizontal, in degrees.

External Pressure Coefficient, (GC_p) - Walls



Notes

1. Vertical scale denotes (GCp) to be used with q_h.
2. Horizontal scale denotes effective wind area, in ft² (m²).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of (GCp) for walls shall be reduced by 10% when $\theta \le 10°$.

FIGURE 30.3-1 Components and Cladding [$h \le 60$ ft ($h \le 18.3$ m)]: External Pressure Coefficients, (GC_p), for Enclosed and Partially Enclosed Buildings—Walls

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	Description:	Wind Load Development - Componenets and Cladding - Main Building

Roof Design Pressures for Components and Cladding

$p = q_h[(GC_p) - (GC_{pi})]$

Bldg 1 Region 1 - Roof	Hip
Bldg 1 Region 1 - Angle	26.57 °

Region 1 Roof Pressures, p

Zone	10 ft²	20 ft²	50 ft²	100 ft²	200 ft²
All Zones	20 psf	17 psf	13 psf	11 psf	11 psf
1	-35 psf	-31 psf	-26 psf	-22 psf	-22 psf
2e, 2r & 3	-49 psf	-44 psf	-37 psf	-32 psf	-26 psf
1 OH	-43 psf	-42 psf	-41 psf	-41 psf	-40 psf
2e & 2r OH	-55 psf	-53 psf	-50 psf	-47 psf	-45 psf
3 OH	-66 psf	-58 psf	-48 psf	-41 psf	-34 psf

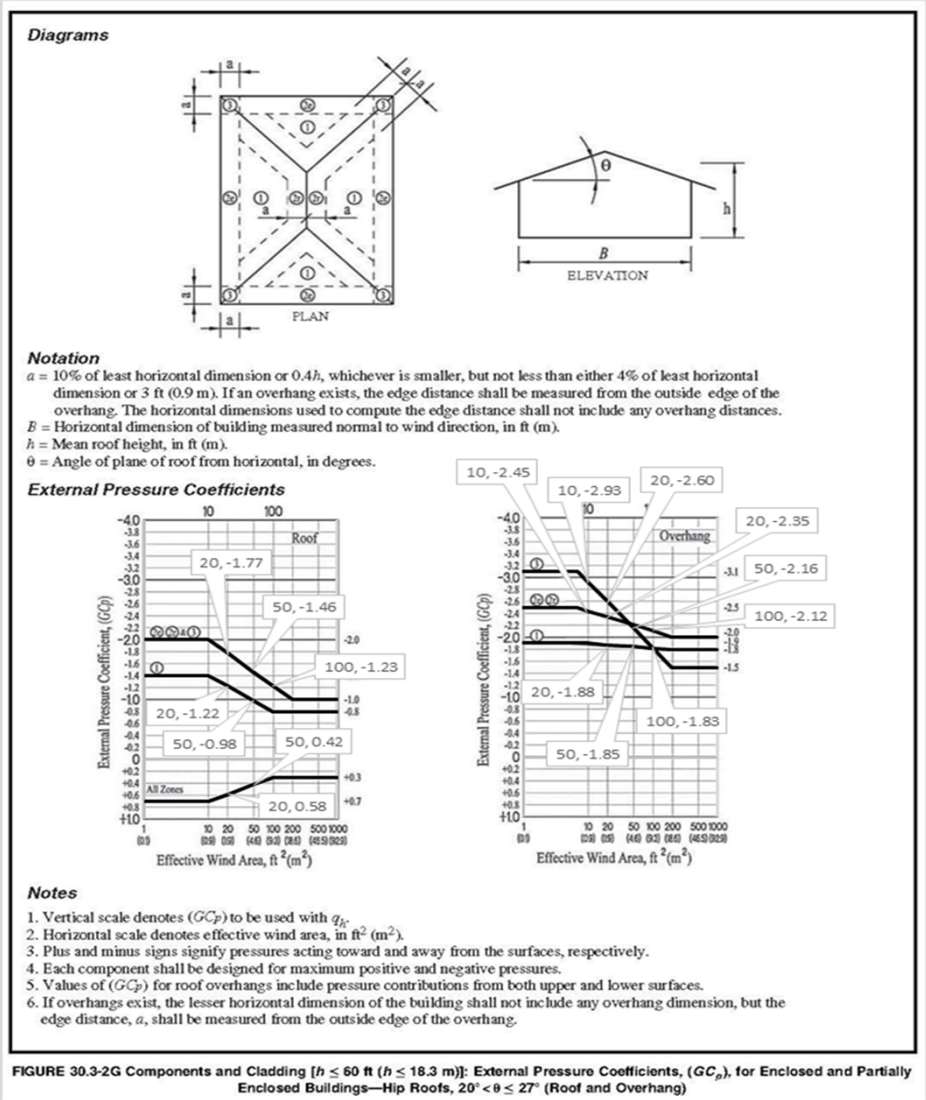

FIGURE 30.3-2G Components and Cladding [$h \le 60$ ft ($h \le 18.3$ m)]: External Pressure Coefficients, (GC_p), for Enclosed and Partially Enclosed Buildings—Hip Roofs, $20° < \theta \le 27°$ (Roof and Overhang)

Hazards by Location

Search Information

Coordinates: 38.869035, -94.428782

Elevation: 961 ft

Timestamp: 2021-01-27T13:54:08.397Z

Hazard Type: Snow

ASCE 7-16			ASCE 7-10			ASCE 7-05		
Ground Snow Load	20	lb/sqft	Ground Snow Load	20	lb/sqft	Ground Snow Load	20	lb/sqft

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer.

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Hazards by Location

Search Information

Coordinates: 38.869035, -94.428782

Elevation: 961 ft

Timestamp: 2021-01-27T13:56:20.677Z

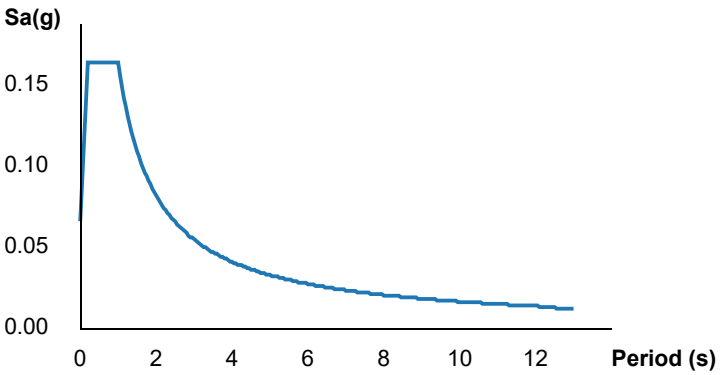
Hazard Type: Seismic

Reference Document: ASCE7-16

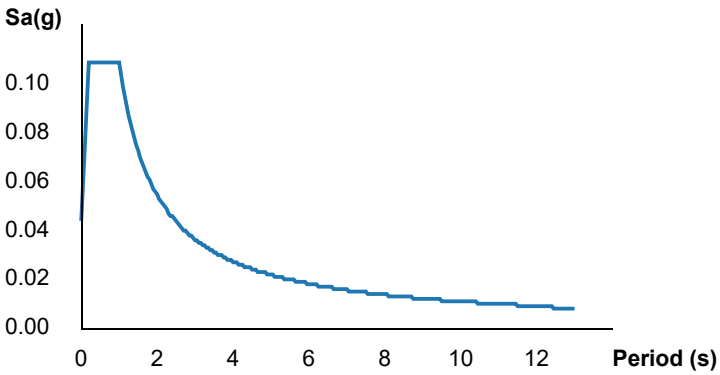
Risk Category: II

Site Class: D-default

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.1	MCE_R ground motion (period=0.2s)
S_1	0.068	MCE_R ground motion (period=1.0s)
S_{MS}	0.159	Site-modified spectral acceleration value
S_{M1}	0.164	Site-modified spectral acceleration value
S_{DS}	0.106	Numeric seismic design value at 0.2s SA
S_{D1}	0.109	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	B	Seismic design category
F_a	1.6	Site amplification factor at 0.2s
F_v	2.4	Site amplification factor at 1.0s
CR_S	0.927	Coefficient of risk (0.2s)

CR_1	0.876	Coefficient of risk (1.0s)
PGA	0.047	MCE_G peak ground acceleration
F_{PGA}	1.6	Site amplification factor at PGA
PGA_M	0.075	Site modified peak ground acceleration
T_L	12	Long-period transition period (s)
SsRT	0.1	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.108	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.068	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.078	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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Hazards by Location

Search Information

Coordinates: 38.869035, -94.428782

Elevation: 961 ft

Timestamp: 2021-01-27T13:55:57.932Z

Hazard Type: Wind

ASCE 7-16

MRI 10-Year	76 mph
MRI 25-Year	83 mph
MRI 50-Year	88 mph
MRI 100-Year	94 mph
Risk Category I	103 mph
Risk Category II	109 mph
Risk Category III	117 mph
Risk Category IV	122 mph

ASCE 7-10

MRI 10-Year	76 mph
MRI 25-Year	84 mph
MRI 50-Year	90 mph
MRI 100-Year	96 mph
Risk Category I	105 mph
Risk Category II	115 mph
Risk Category III-IV	120 mph

ASCE 7-05

ASCE 7-05 Wind Speed	90 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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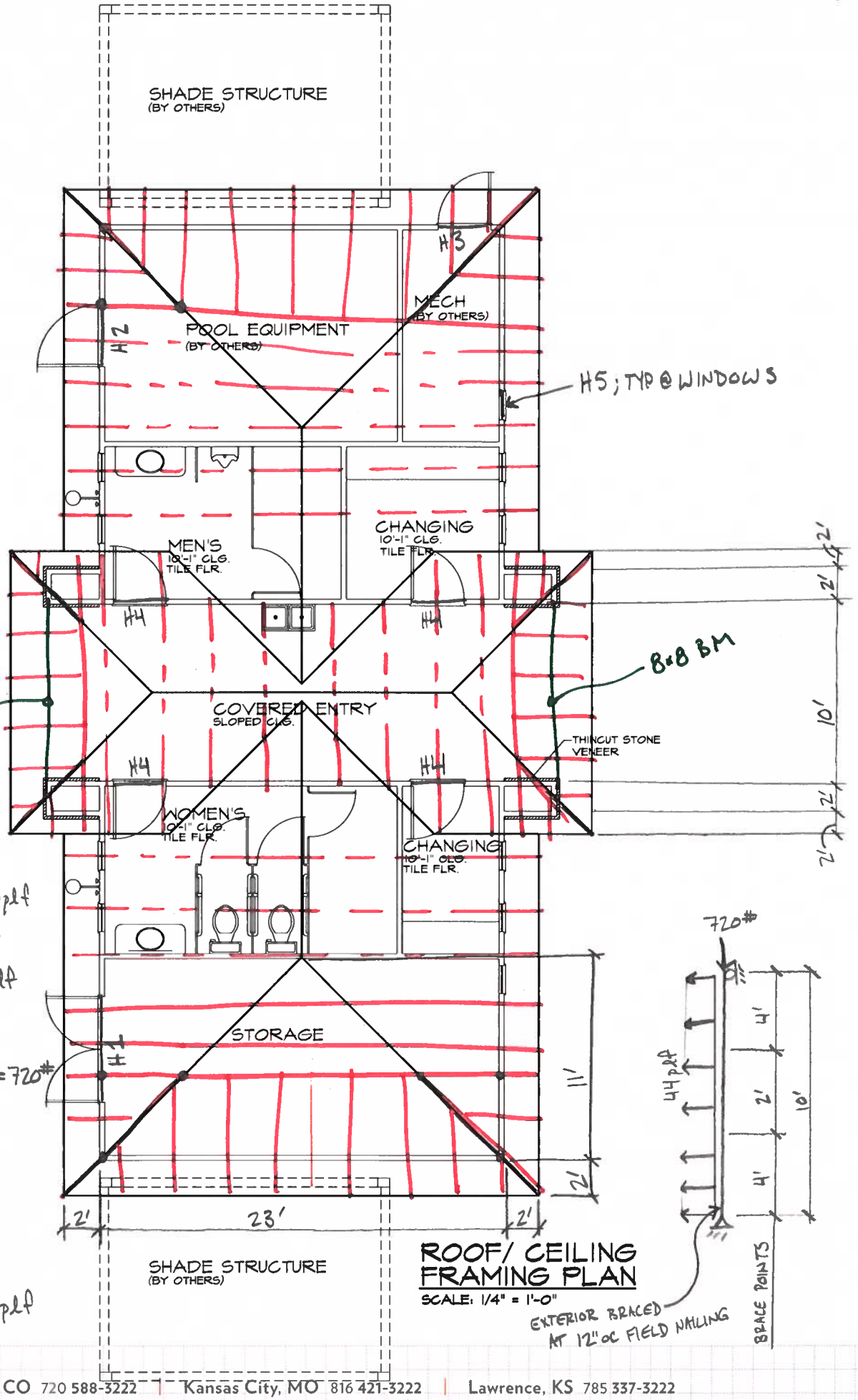
this website does not imply approval by the governing building code bodies responsible for building code approval and interpretation for the building site described by latitude/longitude location in the report.

Gravity Design

APEX ENGINEERS, INC



EAST & WEST WALL

TRIB. WIDTH = $2' + \frac{23'}{2} = 13.5'$

ROOF DL = 20psf x 13.5' = 270plf

L_r = 20 psf x 13.5' = 270 plf

S = 15 psf x 13.5' = 203plf

D+L_r GOVERNS

$P_{D+L_r} = (2)(270plf)(16"/12"/ft) = 720\#$

E&C WIND LOAD:

$A_e = \begin{cases} 10' \times \frac{10'}{3} = 33\ \text{ft}^2 \leftarrow \text{USE} \\ 10' \times 1.33' = 13.3\ \text{ft}^2 \end{cases}$

ZONE 5

P = -33 psf, +25psf

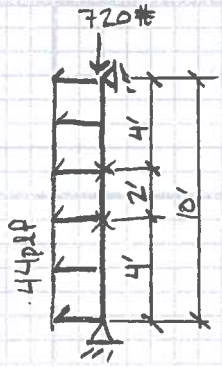
$N_{WL} = (-33psf)(16"/12"/ft) = -44plf$

EXTERIOR BRACED AT 12" OC FIELD NAILING

ROOF/ CEILING FRAMING PLAN

SCALE: 1/4" = 1'-0"

LOW ROOF WALL STUD LOAD



FROM PREVIOUS PAGE

- BLOCKING @ 4' oc MAX SPACING
- EXTERIOR SIDE BRACED @12"oc BY PANEL NAILING

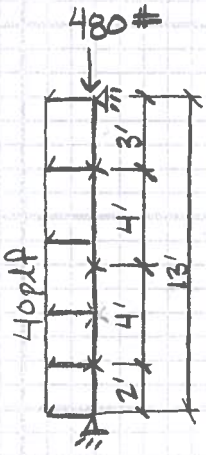
$P_{DL} = 360\#$

$P_{Lr} = 360\#$

$W_{c\&c} = -44 plf$

Can use (1) 2x6 at 16" O.C.

HIGH ROOF WALL STUD LOAD



- BLOCKING @ 4' oc MAX SPACING
- BREEZEWAY FACE BRACED @12"oc BY WALL PANEL FIELD NAILING

TRIBUTARY WIDTH = $4' + \frac{10'}{2} = 9'$

		STUD SPACING		
ROOF DL = 20psf x 9'	= 180 plf	x 16"/12"/ft	=	240#
L_r = 20psf x 9'	= 180 plf	x 16"/12"/ft	=	240#
S = 15 psf x 9'	= 135 plf	x 16"/12"/ft	=	180#

$D + L_r = 480\#$

Use (1) 2x6 at 16" O.C.

WIND C&C: WALL ZONES

$$A_e = \begin{cases} 13' \times 13'/3 = 56\# \leftarrow \text{USE} \\ 13' \times 16"/12"/ft = 17\# \end{cases}$$

$p = -30psf, +24psf$

$W_{c\&c} = (-30psf)(16"/12"/ft) = -40 plf$

LOW ROOF TOP CHORD LOAD

W_{DL} = 270 plf

W_{Lr} = 270 plf

W_S = 203 plf

use (2) 2 x 6

WORST CASE AXIAL FORCE FROM WIND, T=C = 805# (FROM pg-1)

HIGH ROOF TOP CHORD LOAD

W_{DL} = 180 plf

W_{Lr} = 180 plf

W_S = 135 plf

use (2) 2x6

WORST CASE AXIAL FORCE FROM WIND, T=C = 649# (FROM pg-1)

HEADERS:

H1

· CONSERVATIVELY ASSUME GIRDER TRUSS POINT LOAD IS AT MID-SPAN

GIRDER TRUSS: TRIB. WIDTH = $\frac{2' + 5.5'}{2}$ = 3.75'

W_{DL} = 20 psf x 3.75' = 75 plf

W_{Lr} = 20 psf x 3.75' = 75 plf → REACTIONS

W_S = 15 psf x 3.75' = 57 plf

P_{DL} = 75 plf x $\frac{27'}{2}$ = 1,013 #

P_{Lr} = 1,013 #

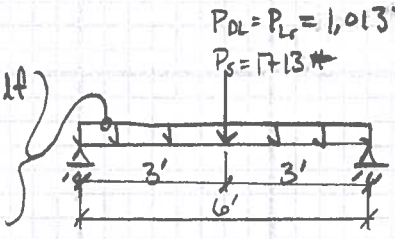
P_S = 57 plf x $\frac{27'}{2}$ = 763 #

WALL HEIGHT ABOVE = 3.25'

W_{DL} = 270 plf + (3.25')(15 psf) = 320 plf

W_{Lr} = 270 plf

W_S = 203 plf



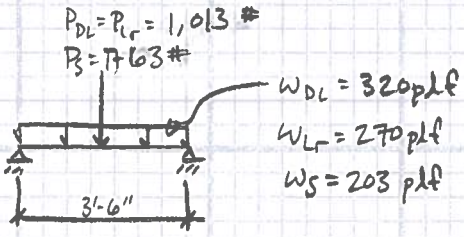
H1

use (2) 2x12

H2

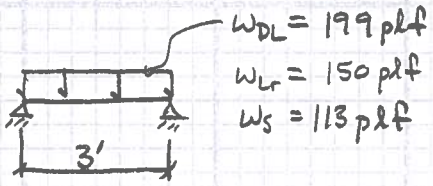
SPAN = 3'-6"

SAME LOADS AS H1



use (2) 2x6

H3



use (2) 2x4

TRIB. WIDTH = $2' + \frac{11'}{2} = 7.5'$

DL = $L_r = (20\text{psf})(7.5') = 150$ plf

WALL = $(3.25')(15\text{ psf}) = 49$ plf

S = $(15\text{psf})(7.5') = 113$ plf

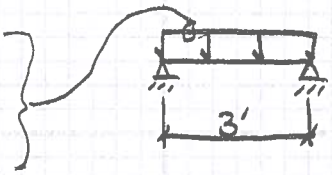
H4

WALL = $(6.25')(15\text{psf}) = 94$ plf

$w_{DL} = 180\text{plf} + 94\text{plf} = 274$ plf

$w_{Lr} = 180$ plf

$w_S = 135$ plf



use (2) 2x4

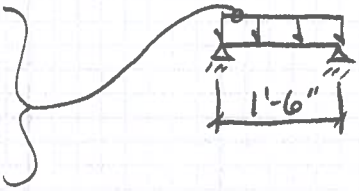
H5

WALL = $(1.5')(15\text{psf}) = 23$ plf

$w_{DL} = 270\text{ plf} + 23\text{plf} = 293$ plf

$w_{Lr} = 270$ plf

$w_S = 203$ plf



use (2) 2x4

by inspection H4

OFFICE STANDARD IS NOT TO USE ANYTHING SMALLER THAN (2) 2x10 (ADEQUATE BY INSPECTION FOR HEADERS H2-H5)

Wood Beam

DESCRIPTION: Header H1

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

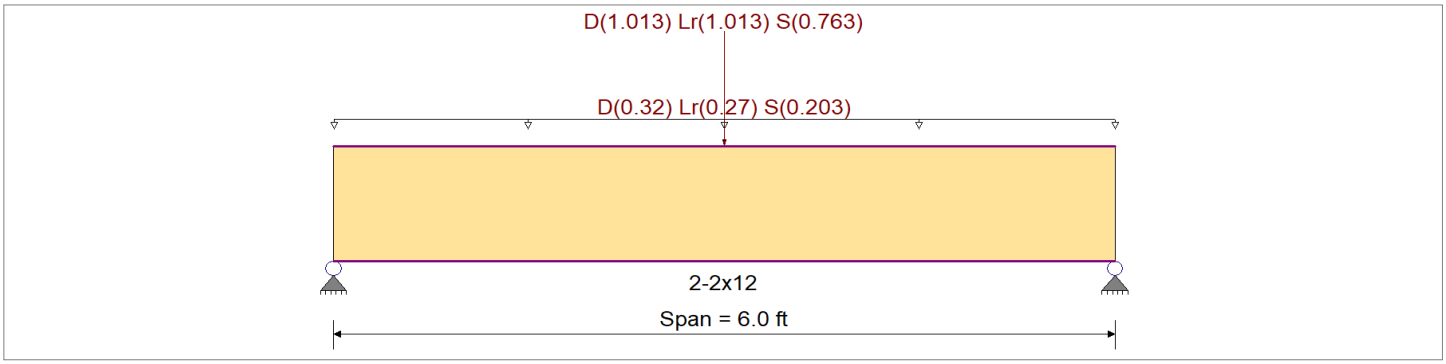
Wood Species : Spruce-Pine-Fir
Wood Grade : No. 1/No. 2

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

Fb +	875.0 psi
Fb -	875.0 psi
Fc - Prll	1,150.0 psi
Fc - Perp	425.0 psi
Fv	135.0 psi
Ft	450.0 psi

E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	26.220 pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations

Uniform Load : D = 0.320, Lr = 0.270, S = 0.2030 , Tributary Width = 1.0 ft
Point Load : D = 1.013, Lr = 1.013, S = 0.7630 k @ 3.0 ft, (Girder truss reaction)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.987 : 1	Maximum Shear Stress Ratio	=	0.590 : 1
Section used for this span		2-2x12	Section used for this span		2-2x12
fb: Actual	=	1,079.75 psi	fv: Actual	=	99.57 psi
Fb: Allowable	=	1,093.75 psi	Fv: Allowable	=	168.75 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	3.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.032 in	Ratio =	2265 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.067 in	Ratio =	1082 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.713	0.429	0.90	1.000	1.00	1.00	1.00	1.00	1.00	2.96	561.21	787.50	1.17	52.10	121.50
+D+Lr					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.987	0.590	1.25	1.000	1.00	1.00	1.00	1.00	1.00	5.69	1,079.75	1093.75	2.24	99.57	168.75
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.946	0.566	1.15	1.000	1.00	1.00	1.00	1.00	1.00	5.02	951.47	1006.25	1.98	87.82	155.25
+D+0.750Lr					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.869	0.520	1.25	1.000	1.00	1.00	1.00	1.00	1.00	5.01	950.12	1093.75	1.97	87.70	168.75
+D+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.849	0.508	1.15	1.000	1.00	1.00	1.00	1.00	1.00	4.50	853.90	1006.25	1.78	78.89	155.25
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.241	0.145	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.78	336.73	1400.00	0.70	31.26	216.00

Wood Beam

DESCRIPTION: Header H1

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0665	3.022		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.783	2.783
Overall MINimum	0.991	0.991
D Only	1.467	1.467
+D+Lr	2.783	2.783
+D+S	2.457	2.457
+D+0.750Lr	2.454	2.454
+D+0.750S	2.209	2.209
+0.60D	0.880	0.880
Lr Only	1.317	1.317
S Only	0.991	0.991

Wood Beam

DESCRIPTION: H2

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

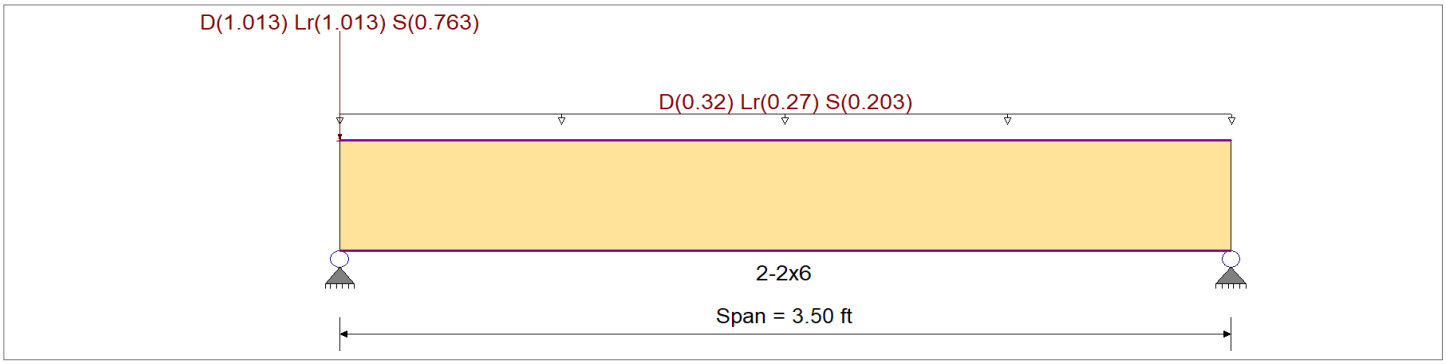
Wood Species : Spruce-Pine-Fir
Wood Grade : No. 1/No. 2

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

Fb +	875.0 psi
Fb -	875.0 psi
Fc - Prll	1,150.0 psi
Fc - Perp	425.0 psi
Fv	135.0 psi
Ft	450.0 psi

E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	26.220 pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations

Beam self weight calculated and added to loads
Uniform Load : D = 0.320, Lr = 0.270, S = 0.2030 , Tributary Width = 1.0 ft
Point Load : D = 1.013, Lr = 1.013, S = 0.7630 k @ 0.0 ft, (Girder)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.507 : 1	Maximum Shear Stress Ratio	=	0.416 : 1
Section used for this span		2-2x6	Section used for this span		2-2x6
fb: Actual	=	720.43 psi	fv: Actual	=	70.24 psi
Fb: Allowable	=	1,421.88 psi	Fv: Allowable	=	168.75 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	1.750 ft	Location of maximum on span	=	3.053 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.016 in	Ratio =	2667 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.035 in	Ratio =	1214 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L		M	fb	F'b	V	fv	F'v
D Only														0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.383	0.315	0.90	1.300	1.00	1.00	1.00	1.00	1.00		0.49	392.41	1023.75	0.42	38.26	121.50
+D+Lr					1.300	1.00	1.00	1.00	1.00	1.00				0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.507	0.416	1.25	1.300	1.00	1.00	1.00	1.00	1.00		0.91	720.43	1421.88	0.77	70.24	168.75
+D+S					1.300	1.00	1.00	1.00	1.00	1.00				0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.489	0.401	1.15	1.300	1.00	1.00	1.00	1.00	1.00		0.81	639.03	1308.13	0.69	62.30	155.25
+D+0.750Lr					1.300	1.00	1.00	1.00	1.00	1.00				0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.449	0.369	1.25	1.300	1.00	1.00	1.00	1.00	1.00		0.80	638.42	1421.88	0.68	62.24	168.75
+D+0.750S					1.300	1.00	1.00	1.00	1.00	1.00				0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.441	0.363	1.15	1.300	1.00	1.00	1.00	1.00	1.00		0.73	577.38	1308.13	0.62	56.29	155.25
+0.60D					1.300	1.00	1.00	1.00	1.00	1.00				0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.129	0.106	1.60	1.300	1.00	1.00	1.00	1.00	1.00		0.30	235.45	1820.00	0.25	22.96	216.00

Wood Beam

DESCRIPTION: H2

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0346	1.763		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.064	1.038
Overall MINimum	1.118	0.355
D Only	1.578	0.565
+D+Lr	3.064	1.038
+D+S	2.697	0.921
+D+0.750Lr	2.692	0.920
+D+0.750S	2.417	0.832
+0.60D	0.947	0.339
Lr Only	1.486	0.473
S Only	1.118	0.355

Wood Beam

DESCRIPTION: H3

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

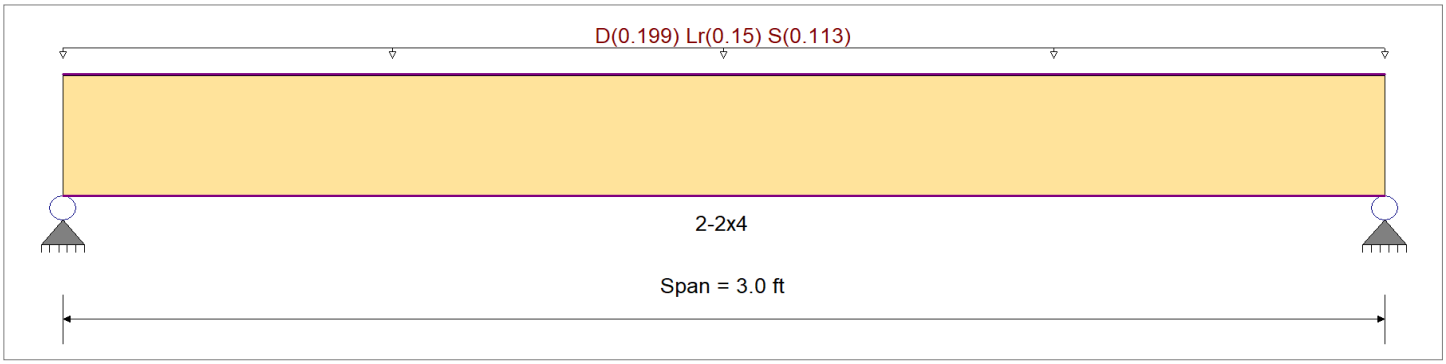
Wood Species :
Wood Grade :

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,000.0 psi
Fc - Perp	1,000.0 psi
Fv	65.0 psi
Ft	65.0 psi

E : Modulus of Elasticity

Ebend- xx	1,300.0 ksi
Eminbend - xx	1,300.0 ksi
Density	34.0 pcf

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 loads
Uniform Load : D = 0.1990, Lr = 0.150, S = 0.1130 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.620 : 1	Maximum Shear Stress Ratio	=	0.751 : 1
Section used for this span		2-2x4	Section used for this span		2-2x4
fb: Actual	=	774.69 psi	fv: Actual	=	61.02 psi
Fb: Allowable	=	1,250.00 psi	Fv: Allowable	=	81.25 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	1.500 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.020 in	Ratio =	1824 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.046 in	Ratio =	778 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.493	0.598	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.23	444.08	900.00	0.24	34.98	58.50
+D+Lr					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.620	0.751	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.40	774.69	1250.00	0.43	61.02	81.25
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.603	0.730	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.35	693.14	1150.00	0.38	54.60	74.75
+D+0.750Lr					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.554	0.671	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.35	692.04	1250.00	0.38	54.51	81.25
+D+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.549	0.665	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.32	630.87	1150.00	0.35	49.69	74.75
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.167	0.202	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.14	266.45	1600.00	0.15	20.99	104.00

Wood Beam

DESCRIPTION: H3

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0462	1.511		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.527	0.527
Overall MINimum	0.170	0.170
D Only	0.302	0.302
+D+Lr	0.527	0.527
+D+S	0.472	0.472
+D+0.750Lr	0.471	0.471
+D+0.750S	0.429	0.429
+0.60D	0.181	0.181
Lr Only	0.225	0.225
S Only	0.170	0.170

Wood Beam

DESCRIPTION: H4

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

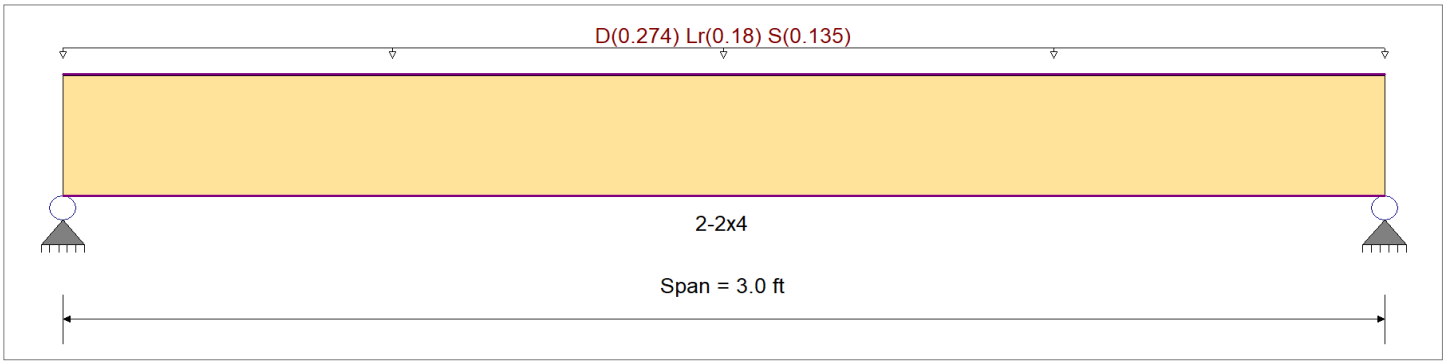
Wood Species :
Wood Grade :

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

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,000.0 psi
Fc - Perp	1,000.0 psi
Fv	65.0 psi
Ft	65.0 psi

E : Modulus of Elasticity

Ebend- xx	1,300.0 ksi
Eminbend - xx	1,300.0 ksi
Density	34.0 pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations

Uniform Load : D = 0.2740, Lr = 0.180, S = 0.1350 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.801 : 1	Maximum Shear Stress Ratio	=	0.970 : 1
Section used for this span		2-2x4	Section used for this span		2-2x4
fb: Actual	=	1,000.65 psi	fv: Actual	=	78.82 psi
Fb: Allowable	=	1,250.00 psi	Fv: Allowable	=	81.25 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	1.500 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.024 in	Ratio =	1520 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.060 in	Ratio =	602 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.671	0.813	0.90							0.31	603.92	900.00	0.33	47.57	58.50
+D+Lr					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.801	0.970	1.25							0.51	1,000.65	1250.00	0.55	78.82	81.25
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.784	0.950	1.15							0.46	901.47	1150.00	0.50	71.01	74.75
+D+0.750Lr					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.721	0.874	1.25							0.46	901.47	1250.00	0.50	71.01	81.25
+D+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.719	0.872	1.15							0.42	827.08	1150.00	0.46	65.15	74.75
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.226	0.274	1.60							0.18	362.35	1600.00	0.20	28.54	104.00

Wood Beam

DESCRIPTION: H4

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0597	1.511		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.681	0.681
Overall MINimum	0.203	0.203
D Only	0.411	0.411
+D+Lr	0.681	0.681
+D+S	0.614	0.614
+D+0.750Lr	0.614	0.614
+D+0.750S	0.563	0.563
+0.60D	0.247	0.247
Lr Only	0.270	0.270
S Only	0.203	0.203

Wood Beam

Lic. # : KW-06005244 APEX ENGINEERS INC

DESCRIPTION: Breezeway Beam

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

Wood Species : Spruce-Pine-Fir
Wood Grade : No. 1/No. 2

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

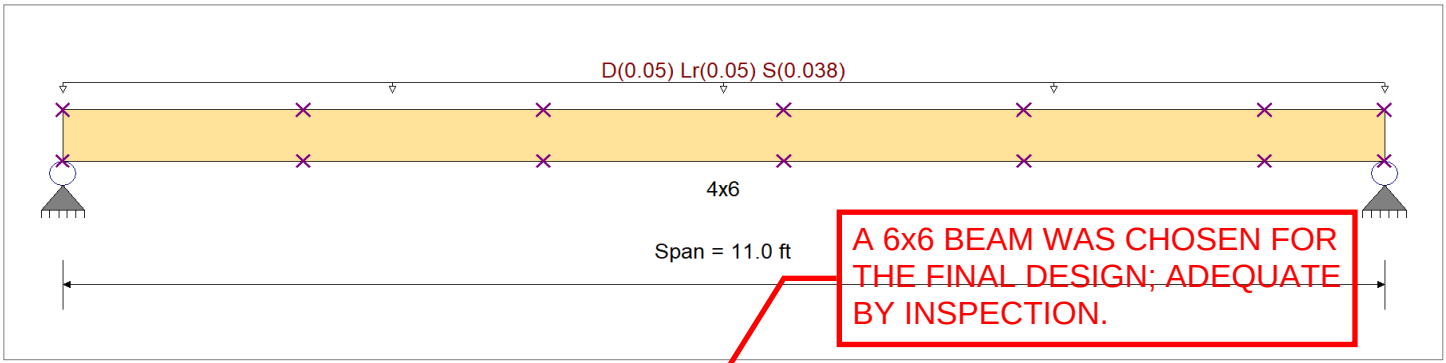
Fb +	875.0 psi
Fb -	875.0 psi
Fc - Prll	1,150.0 psi
Fc - Perp	425.0 psi
Fv	135.0 psi
Ft	450.0 psi

E : Modulus of Elasticity

Ebend- xx	1,400.0 ksi
Eminbend - xx	510.0 ksi
Density	26.220 pcf

Unbraced Lengths

First Brace starts at ft from Left-Most support
Regular spacing of lateral supports on length of beam = 2.0 ft



Applied Loads

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

Uniform Load : D = 0.050, Lr = 0.050, S = 0.0380 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.725 : 1	Maximum Shear Stress Ratio	=	0.234 : 1
Section used for this span		4x6	Section used for this span		4x6
fb: Actual	=	1,028.57 psi	fv: Actual	=	39.42 psi
Fb: Allowable	=	1,418.02 psi	Fv: Allowable	=	168.75 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	5.500 ft	Location of maximum on span	=	10.558 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.244 in	Ratio =	541 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.488 in	Ratio =	270 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
D Only													0.00	0.00	0.00	0.00
Length = 1.967 ft	1	0.296	0.162	0.90	1.300	1.00	1.00	1.00	1.00	1.00	0.44	302.09	1021.82	0.25	19.71	121.50
Length = 2.007 ft	1	0.465	0.162	0.90	1.300	1.00	1.00	1.00	1.00	1.00	0.70	474.72	1021.78	0.18	19.71	121.50
Length = 2.007 ft	1	0.503	0.162	0.90	1.300	1.00	1.00	1.00	1.00	1.00	0.76	514.29	1021.78	0.08	19.71	121.50
Length = 2.007 ft	1	0.499	0.162	0.90	1.300	1.00	1.00	1.00	1.00	1.00	0.75	510.34	1021.78	0.12	19.71	121.50
Length = 2.007 ft	1	0.400	0.162	0.90	1.300	1.00	1.00	1.00	1.00	1.00	0.60	408.96	1021.78	0.22	19.71	121.50
Length = 1.004 ft	1	0.167	0.162	0.90	1.300	1.00	1.00	1.00	1.00	1.00	0.25	170.57	1022.78	0.25	19.71	121.50
+D+Lr					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.967 ft	1	0.426	0.234	1.25	1.300	1.00	1.00	1.00	1.00	1.00	0.89	604.19	1418.10	0.51	39.42	168.75
Length = 2.007 ft	1	0.670	0.234	1.25	1.300	1.00	1.00	1.00	1.00	1.00	1.40	949.44	1418.02	0.35	39.42	168.75
Length = 2.007 ft	1	0.725	0.234	1.25	1.300	1.00	1.00	1.00	1.00	1.00	1.51	1,028.57	1418.02	0.15	39.42	168.75

Wood Beam

DESCRIPTION: Breezeway Beam

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
Length = 2.007 ft	1	0.720	0.234	1.25	1.300	1.00	1.00	1.00	1.00	1.00	1.50	1,020.68	1418.02	0.25	39.42	168.75
Length = 2.007 ft	1	0.577	0.234	1.25	1.300	1.00	1.00	1.00	1.00	1.00	1.20	817.91	1418.02	0.45	39.42	168.75
Length = 1.004 ft	1	0.240	0.234	1.25	1.300	1.00	1.00	1.00	1.00	1.00	0.50	341.14	1419.99	0.51	39.42	168.75
+D+S					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.967 ft	1	0.407	0.223	1.15	1.300	1.00	1.00	1.00	1.00	1.00	0.78	531.69	1304.94	0.45	34.69	155.25
Length = 2.007 ft	1	0.640	0.223	1.15	1.300	1.00	1.00	1.00	1.00	1.00	1.23	835.51	1304.87	0.31	34.69	155.25
Length = 2.007 ft	1	0.694	0.223	1.15	1.300	1.00	1.00	1.00	1.00	1.00	1.33	905.14	1304.87	0.13	34.69	155.25
Length = 2.007 ft	1	0.688	0.223	1.15	1.300	1.00	1.00	1.00	1.00	1.00	1.32	898.20	1304.87	0.22	34.69	155.25
Length = 2.007 ft	1	0.552	0.223	1.15	1.300	1.00	1.00	1.00	1.00	1.00	1.06	719.76	1304.87	0.40	34.69	155.25
Length = 1.004 ft	1	0.230	0.223	1.15	1.300	1.00	1.00	1.00	1.00	1.00	0.44	300.20	1306.53	0.45	34.69	155.25
+D+0.750Lr					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.967 ft	1	0.373	0.204	1.25	1.300	1.00	1.00	1.00	1.00	1.00	0.78	528.66	1418.10	0.44	34.49	168.75
Length = 2.007 ft	1	0.586	0.204	1.25	1.300	1.00	1.00	1.00	1.00	1.00	1.22	830.76	1418.02	0.31	34.49	168.75
Length = 2.007 ft	1	0.635	0.204	1.25	1.300	1.00	1.00	1.00	1.00	1.00	1.32	900.00	1418.02	0.13	34.49	168.75
Length = 2.007 ft	1	0.630	0.204	1.25	1.300	1.00	1.00	1.00	1.00	1.00	1.31	893.09	1418.02	0.22	34.49	168.75
Length = 2.007 ft	1	0.505	0.204	1.25	1.300	1.00	1.00	1.00	1.00	1.00	1.05	715.67	1418.02	0.39	34.49	168.75
Length = 1.004 ft	1	0.210	0.204	1.25	1.300	1.00	1.00	1.00	1.00	1.00	0.44	298.50	1419.99	0.44	34.49	168.75
+D+0.750S					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.967 ft	1	0.363	0.199	1.15	1.300	1.00	1.00	1.00	1.00	1.00	0.70	474.29	1304.94	0.40	30.94	155.25
Length = 2.007 ft	1	0.571	0.199	1.15	1.300	1.00	1.00	1.00	1.00	1.00	1.10	745.31	1304.87	0.28	30.94	155.25
Length = 2.007 ft	1	0.619	0.199	1.15	1.300	1.00	1.00	1.00	1.00	1.00	1.19	807.43	1304.87	0.12	30.94	155.25
Length = 2.007 ft	1	0.614	0.199	1.15	1.300	1.00	1.00	1.00	1.00	1.00	1.18	801.23	1304.87	0.20	30.94	155.25
Length = 2.007 ft	1	0.492	0.199	1.15	1.300	1.00	1.00	1.00	1.00	1.00	0.94	642.06	1304.87	0.35	30.94	155.25
Length = 1.004 ft	1	0.205	0.199	1.15	1.300	1.00	1.00	1.00	1.00	1.00	0.39	267.79	1306.53	0.40	30.94	155.25
+0.60D					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.967 ft	1	0.100	0.055	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.27	181.26	1813.73	0.15	11.82	216.00
Length = 2.007 ft	1	0.157	0.055	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.42	284.83	1813.59	0.11	11.82	216.00
Length = 2.007 ft	1	0.170	0.055	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.45	308.57	1813.59	0.05	11.82	216.00
Length = 2.007 ft	1	0.169	0.055	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.45	306.20	1813.59	0.07	11.82	216.00
Length = 2.007 ft	1	0.135	0.055	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.36	245.37	1813.59	0.13	11.82	216.00
Length = 1.004 ft	1	0.056	0.055	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.15	102.34	1816.89	0.15	11.82	216.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.4877	5.540		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.550	0.550
Overall MINimum	0.209	0.209
D Only	0.275	0.275
+D+Lr	0.550	0.550
+D+S	0.484	0.484
+D+0.750Lr	0.481	0.481
+D+0.750S	0.432	0.432
+0.60D	0.165	0.165
Lr Only	0.275	0.275
S Only	0.209	0.209

Wood Column

DESCRIPTION: Breezeway column

Code References

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-16

General Information

Analysis Method : Allowable Stress Design
End Fixities : Top & Bottom Pinned
Overall Column Height : 13 ft
(Used for non-slender calculations)

Wood Species : Spruce-Pine-Fir
Wood Grade : No. 1/No. 2

Fb +	875 psi	Fv	135 psi
Fb -	875 psi	Ft	450 psi
Fc - Prll	1150 psi	Density	26.22 pcf
Fc - Perp	425 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1400	1400	1400 ksi
Minimum	510	510	

Wood Section Name : **2-2x4**
Wood Grading/Manuf. : Graded Lumber
Wood Member Type : Sawn

Exact Width	**3.0**	in
Exact Depth	**3.50**	in
Area	10.50	in^2
Ix	10.719	in^4
Iy	7.875	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.50
Cf or Cv for Compression	1.150
Cf or Cv for Tension	1.50
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling ABOUT Y-Y Axis
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 13 ft, K = 1.0

Applied Loads

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

Column self weight included : 24.854 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 13.0 ft, D = 0.2750, Lr = 0.2750 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.2668** : 1
Load Combination : +D+Lr
Governing NDS Forumla : Comp Only, fc/Fc'
Location of max.above base : 0.0 ft
At maximum location values are . . .
Applied Axial : 0.5749 k
Applied Mx : 0.0 k-ft
Applied My : 0.0 k-ft
Fc : Allowable : 205.206 psi

PASS Maximum Shear Stress Ratio = **0.0** : 1
Load Combination : +0.60D
Location of max.above base : 13.0 ft
Applied Design Shear : 0.0 psi
Allowable Shear : 216.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y : 0.0 in at 0.0 ft above base
for load combination : n/a
Along X-X : 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.170	0.1409	PASS	0.0 ft	0.0	PASS	13.0 ft
+D+Lr	1.250	0.124	0.2668	PASS	0.0 ft	0.0	PASS	13.0 ft
+D+0.750Lr	1.250	0.124	0.2349	PASS	0.0 ft	0.0	PASS	13.0 ft
+0.60D	1.600	0.098	0.08295	PASS	0.0 ft	0.0	PASS	13.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction k @ Base	@ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only					0.300				
+D+Lr					0.575				
+D+0.750Lr					0.506				

Wood Column

Lic. # : KW-06005244 APEX ENGINEERS INC

DESCRIPTION: Breezeway column

Maximum Reactions

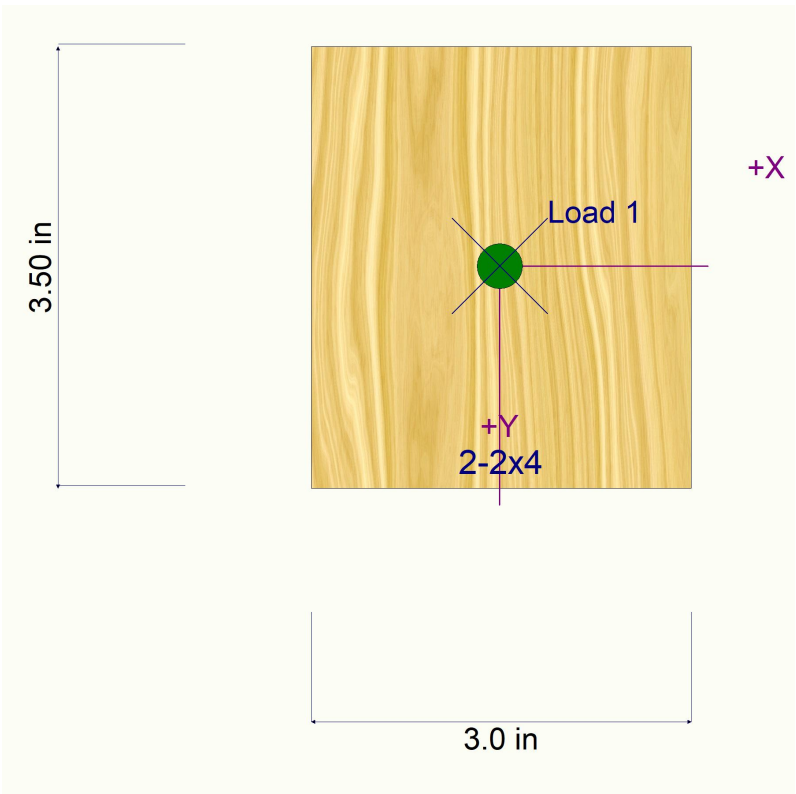
Note: Only non-zero reactions are listed.

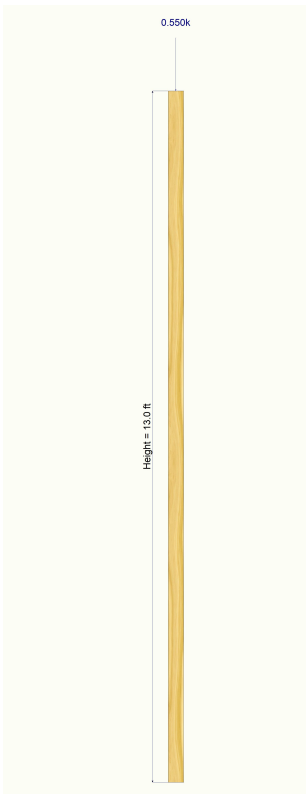
Load Combination	X-X Axis Reaction @ Base	@ Top	k	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments @ Base	@ Top	k-ft	Mx - End Moments @ Base	@ Top
+0.60D						0.180					
Lr Only						0.275					

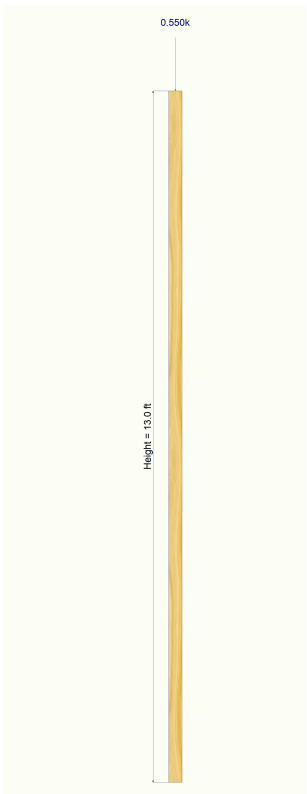
Maximum Deflections for Load Combinations

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

Sketches







GIRDER TRUSS SUPPORTS

$P_{DL} = 1,013\#$

$P_{LR} = 1,013\#$

$P_S = 763\#$

BLOCKED @ 4'oc MAX

(1) 2x6 ADEQUATE TO SUPPORT (SEE FOLLOWING PAGES)

Wood Column

DESCRIPTION: Girder Truss Built Up Column

Code References

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-16

General Information

Analysis Method : Allowable Stress Design
End Fixities : Top & Bottom Pinned
Overall Column Height : 10 ft
(Used for non-slender calculations)

Wood Species
Wood Grade

Fb +	1,500.0 psi	Fv	150.0 psi
Fb -	1,500.0 psi	Ft	1,000.0 psi
Fc - Prll	1,000.0 psi	Density	33.0 pcf
Fc - Perp	1,000.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,400.0	1,400.0	1,400.0 ksi
Minimum	1,400.0	1,400.0	

Wood Section Name: **2x6**
Wood Grading/Manuf.: Graded Lumber
Wood Member Type: Sawn

Exact Width	**1.50**	in
Exact Depth	**5.50**	in
Area	8.250	in^2
Ix	20.797	in^4
Iy	1.547	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0
Cf or Cv for Compression	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Lu for buckling ABOUT Y-Y Axis : 4 ft, 6 ft, K = 1.0
Y-Y (depth) axis : Fully braced against buckling ABOUT X-X Axis

Applied Loads

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

Column self weight included : 18.906 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft, D = 1.013, Lr = 1.013, S = 1.713 k
BENDING LOADS . . .
C&C: Lat. Uniform Load creating Mx-x, W = 0.0440 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.4236** : 1
Load Combination +D+S
Governing NDS Forumla Comp Only, fc/Fc'
Location of max.above base 0.0 ft
At maximum location values are . . .
Applied Axial 2.745 k
Applied Mx 0.0 k-ft
Applied My 0.0 k-ft
Fc : Allowable 785.42 psi

PASS Maximum Shear Stress Ratio = **0.10** : 1
Load Combination +D+0.60W
Location of max.above base 10.0 ft
Applied Design Shear 24.0 psi
Allowable Shear 240.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.220 k	Bottom along Y-Y	0.220 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.3437 in at 5.034 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.762	0.1825	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+Lr	1.250	0.653	0.3036	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+S	1.150	0.683	0.4236	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+0.750Lr	1.250	0.653	0.2660	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+0.750S	1.150	0.683	0.3575	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+0.60W	1.600	0.560	0.2344	PASS	4.027 ft	0.10	PASS	10.0 ft
+D+0.750Lr+0.450W	1.600	0.560	0.2451	PASS	4.027 ft	0.0750	PASS	10.0 ft
+D+0.750S+0.450W	1.600	0.560	0.3162	PASS	4.027 ft	0.0750	PASS	10.0 ft
+0.60D+0.60W	1.600	0.912	0.2231	PASS	5.034 ft	0.10	PASS	10.0 ft

Wood Column

DESCRIPTION: Girder Truss Built Up Column

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D	1.600	0.560	0.08377	PASS	0.0 ft	0.0	PASS	10.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
D Only						1.032					
+D+Lr						2.045					
+D+S						2.745					
+D+0.750Lr						1.792					
+D+0.750S						2.317					
+D+0.60W				0.132	0.132	1.032					
+D+0.750Lr+0.450W				0.099	0.099	1.792					
+D+0.750S+0.450W				0.099	0.099	2.317					
+0.60D+0.60W				0.132	0.132	0.619					
+0.60D						0.619					
Lr Only						1.013					
S Only						1.713					
W Only				0.220	0.220						

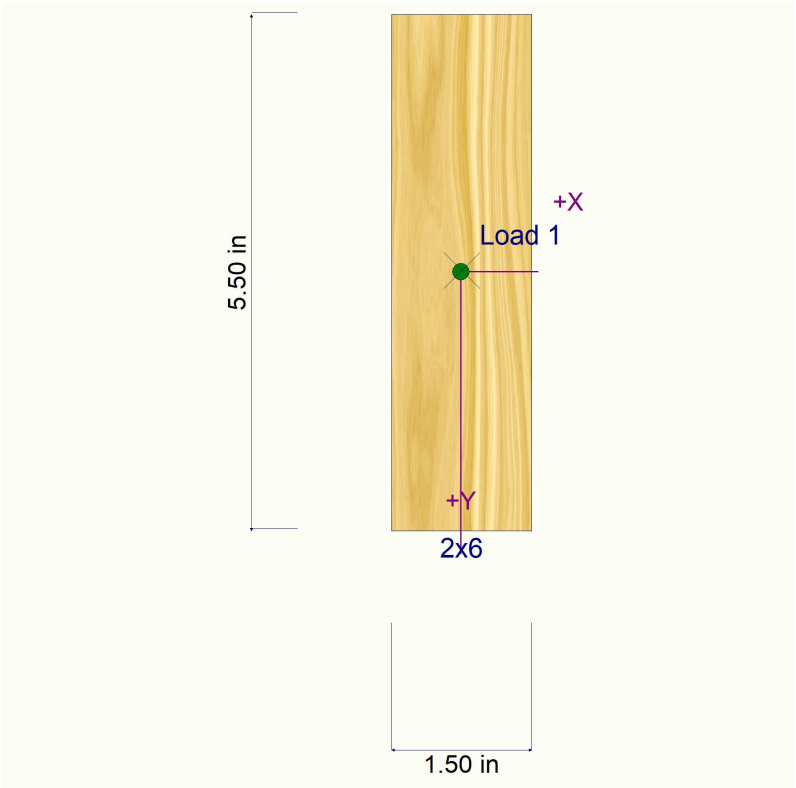
Maximum Deflections for Load Combinations

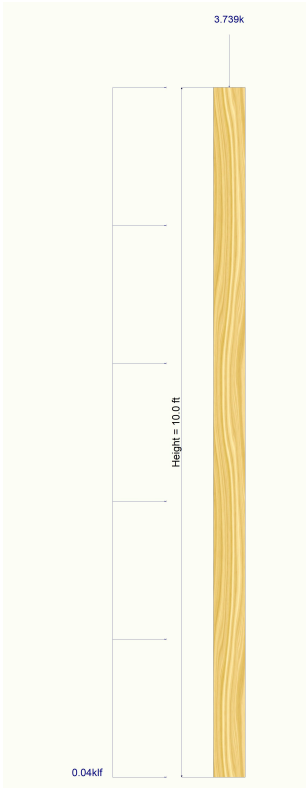
Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.750Lr	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	0.2062 in	5.034 ft
+D+0.750Lr+0.450W	0.0000 in	0.000 ft	0.1547 in	5.034 ft
+D+0.750S+0.450W	0.0000 in	0.000 ft	0.1547 in	5.034 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.2062 in	5.034 ft
+0.60D	0.0000 in	0.000 ft	0.0000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.3437 in	5.034 ft

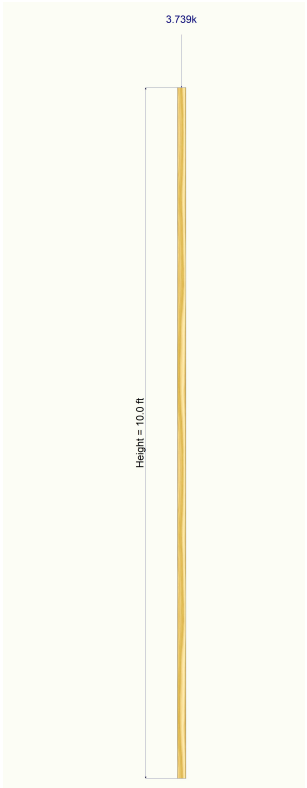
Wood Column

DESCRIPTION: Girder Truss Built Up Column

Sketches







Wood Beam

DESCRIPTION: High Roof Chord

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

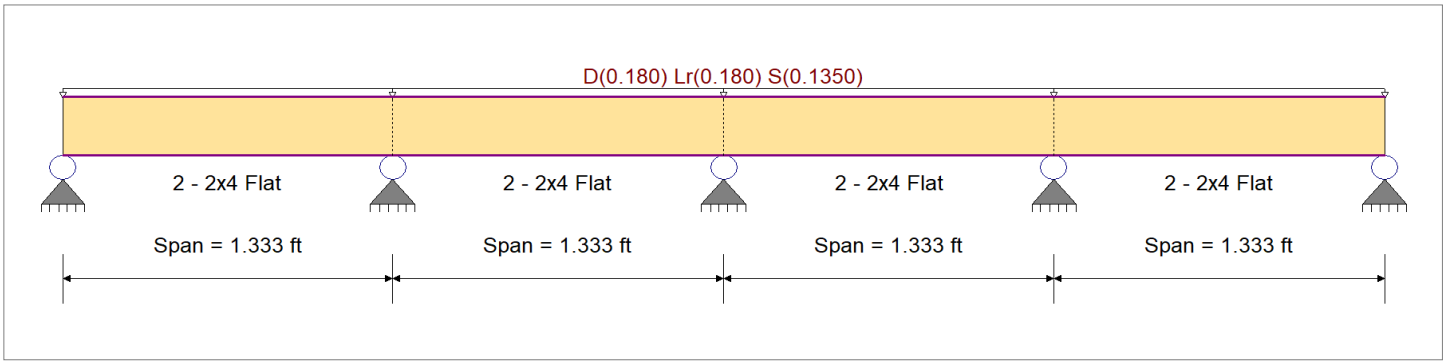
Wood Species :
Wood Grade :

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

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,000.0 psi
Fc - Perp	1,000.0 psi
Fv	65.0 psi
Ft	65.0 psi

E : Modulus of Elasticity

Ebend- xx	1,300.0 ksi
Eminbend - xx	1,300.0 ksi
Density	34.0 pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations

Beam self weight calculated and added to loads
Loads on all spans...
Uniform Load on ALL spans : D = 0.020, Lr = 0.020, S = 0.0150 ksf, Tributary Width = 9.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.126 : 1	Maximum Shear Stress Ratio	=	0.365 : 1
Section used for this span		2 - 2x4 Flat	Section used for this span		2 - 2x4 Flat
fb: Actual	=	157.81 psi	fv: Actual	=	29.68 psi
Fb: Allowable	=	1,250.00 psi	Fv: Allowable	=	81.25 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	1.333 ft	Location of maximum on span	=	1.097 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 <360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.001 in	Ratio =	12514 >=180
Max Upward Total Deflection	-0.000 in	Ratio =	370506 >=180

(2) 2x6 FLATWISE CHOSEN FOR FINAL TOP CHORD DESIGN; ADEQUATE BY INSPECTION

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 1.333 ft	1	0.088	0.255	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.03	79.45	900.00	0.10	14.94	58.50
Length = 1.333 ft	2	0.088	0.255	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.03	79.45	900.00	0.09	14.94	58.50
Length = 1.333 ft	3	0.088	0.255	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.03	79.40	900.00	0.10	14.94	58.50
Length = 1.333 ft	4	0.088	0.255	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.03	79.40	900.00	0.10	14.94	58.50
+D+Lr					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.333 ft	1	0.126	0.365	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.07	157.81	1250.00	0.21	29.68	81.25
Length = 1.333 ft	2	0.126	0.365	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.07	157.81	1250.00	0.17	29.68	81.25
Length = 1.333 ft	3	0.126	0.365	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.07	157.73	1250.00	0.21	29.68	81.25
Length = 1.333 ft	4	0.126	0.365	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.07	157.73	1250.00	0.21	29.68	81.25
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.333 ft	1	0.120	0.348	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.06	138.22	1150.00	0.18	26.00	74.75

Wood Beam

DESCRIPTION: High Roof Chord

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 1.333 ft	2	0.120	0.348	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.06	138.22	1150.00	0.15	26.00	74.75
Length = 1.333 ft	3	0.120	0.348	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.06	138.15	1150.00	0.18	26.00	74.75
Length = 1.333 ft	4	0.120	0.348	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.06	138.15	1150.00	0.18	26.00	74.75
+D+0.750Lr					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.333 ft	1	0.111	0.320	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.06	138.22	1250.00	0.18	26.00	81.25
Length = 1.333 ft	2	0.111	0.320	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.06	138.22	1250.00	0.15	26.00	81.25
Length = 1.333 ft	3	0.111	0.320	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.06	138.15	1250.00	0.18	26.00	81.25
Length = 1.333 ft	4	0.111	0.320	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.06	138.15	1250.00	0.18	26.00	81.25
+D+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.333 ft	1	0.107	0.311	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.05	123.53	1150.00	0.16	23.23	74.75
Length = 1.333 ft	2	0.107	0.311	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.05	123.53	1150.00	0.14	23.23	74.75
Length = 1.333 ft	3	0.107	0.311	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.05	123.46	1150.00	0.16	23.23	74.75
Length = 1.333 ft	4	0.107	0.311	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.05	123.46	1150.00	0.16	23.23	74.75
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.333 ft	1	0.030	0.086	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.02	47.67	1600.00	0.06	8.97	104.00
Length = 1.333 ft	2	0.030	0.086	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.02	47.67	1600.00	0.05	8.97	104.00
Length = 1.333 ft	3	0.030	0.086	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.02	47.64	1600.00	0.06	8.97	104.00
Length = 1.333 ft	4	0.030	0.086	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.02	47.64	1600.00	0.06	8.97	104.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0013	0.591		0.0000	0.000
+D+Lr	2	0.0004	0.726	+D+Lr	-0.0000	0.101
+D+Lr	3	0.0004	0.624	+D+Lr	-0.0000	1.249
+D+Lr	4	0.0013	0.759		0.0000	1.249

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
Overall MAXimum	0.190	0.552	0.449	0.552	0.190
Overall MINimum	0.071	0.206	0.167	0.206	0.071
D Only	0.096	0.278	0.226	0.278	0.096
+D+Lr	0.190	0.552	0.449	0.552	0.190
+D+S	0.166	0.484	0.393	0.484	0.166
+D+0.750Lr	0.166	0.484	0.393	0.484	0.166
+D+0.750S	0.149	0.432	0.351	0.432	0.149
+0.60D	0.057	0.167	0.136	0.167	0.057
Lr Only	0.094	0.274	0.223	0.274	0.094
S Only	0.071	0.206	0.167	0.206	0.071

Wood Column

Lic. # : KW-06005244 APEX ENGINEERS INC

DESCRIPTION: High roof wall design

Code References

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-16

General Information

Analysis Method : Allowable Stress Design
End Fixities : Top & Bottom Pinned
Overall Column Height : 13 ft
(Used for non-slender calculations)

Wood Species : Spruce-Pine-Fir
Wood Grade : No. 1/No. 2

Fb +	875.0 psi	Fv	135.0 psi
Fb -	875.0 psi	Ft	450.0 psi
Fc - Prll	1,150.0 psi	Density	26.220 pcf
Fc - Perp	425.0 psi		

E : Modulus of Elasticity . . .

	x-x Bending	y-y Bending	Axial
Basic	1,400.0	1,400.0	1,400.0 ksi
Minimum	510.0	510.0	

Wood Section Name : **2x6**
Wood Grading/Manuf. : Graded Lumber
Wood Member Type : Sawn

Exact Width	**1.50** in	Area	8.250 in^2
Exact Depth	**5.50** in	Ix	20.797 in^4
		Iy	1.547 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Lu for buckling ABOUT Y-Y Axis : 2 ft, 6 ft, 10 ft, K = 1.0
Y-Y (depth) axis : Fully braced against buckling ABOUT X-X Axis

Applied Loads

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

Column self weight included : 19.528 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 13.0 ft, D = 0.240, Lr = 0.240, S = 0.180 k
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, W = 0.040 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.4868** : 1
Load Combination +D+0.60W
Governing NDS Forumla 1Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base 6.544 ft
At maximum location values are . . .
Applied Axial 0.2595 k
Applied Mx 0.5070 k-ft
Applied My 0.0 k-ft
Fc : Allowable 390.702 psi

PASS Maximum Shear Stress Ratio = **0.1313** : 1
Load Combination +D+0.60W
Location of max.above base 0.0 ft
Applied Design Shear 28.364 psi
Allowable Shear 216.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.260 k	Bottom along Y-Y	0.260 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.3748 in at 6.544 ft above base
for load combination : +D+0.420W
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.328	0.08433	PASS	2.094 ft	0.0	PASS	13.0 ft
+D+Lr	1.250	0.243	0.1574	PASS	2.094 ft	0.0	PASS	13.0 ft
+D+S	1.150	0.263	0.1394	PASS	2.094 ft	0.0	PASS	13.0 ft
+D+0.750Lr	1.250	0.243	0.1385	PASS	2.094 ft	0.0	PASS	13.0 ft
+D+0.750S	1.150	0.263	0.1251	PASS	2.094 ft	0.0	PASS	13.0 ft
+D+0.60W	1.600	0.193	0.4868	PASS	6.544 ft	0.1313	PASS	0.0 ft
+D+0.750Lr+0.450W	1.600	0.193	0.3788	PASS	6.456 ft	0.09849	PASS	13.0 ft
+D+0.750S+0.450W	1.600	0.193	0.3752	PASS	6.456 ft	0.09849	PASS	13.0 ft
+0.60D+0.60W	1.600	0.193	0.4826	PASS	6.544 ft	0.1313	PASS	0.0 ft

Wood Column

DESCRIPTION: High roof wall design

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D	1.600	0.193	0.04831	PASS	2.094 ft	0.0	PASS	13.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base k-ft	My - End Moments @ Top k-ft	Mx - End Moments @ Base k-ft	Mx - End Moments @ Top k-ft
D Only						0.260				
+D+Lr						0.500				
+D+S						0.440				
+D+0.750Lr						0.440				
+D+0.750S						0.395				
+D+0.60W				0.156	0.156	0.260				
+D+0.750Lr+0.450W				0.117	0.117	0.440				
+D+0.750S+0.450W				0.117	0.117	0.395				
+0.60D+0.60W				0.156	0.156	0.156				
+0.60D						0.156				
Lr Only						0.240				
S Only						0.180				
W Only				0.260	0.260					

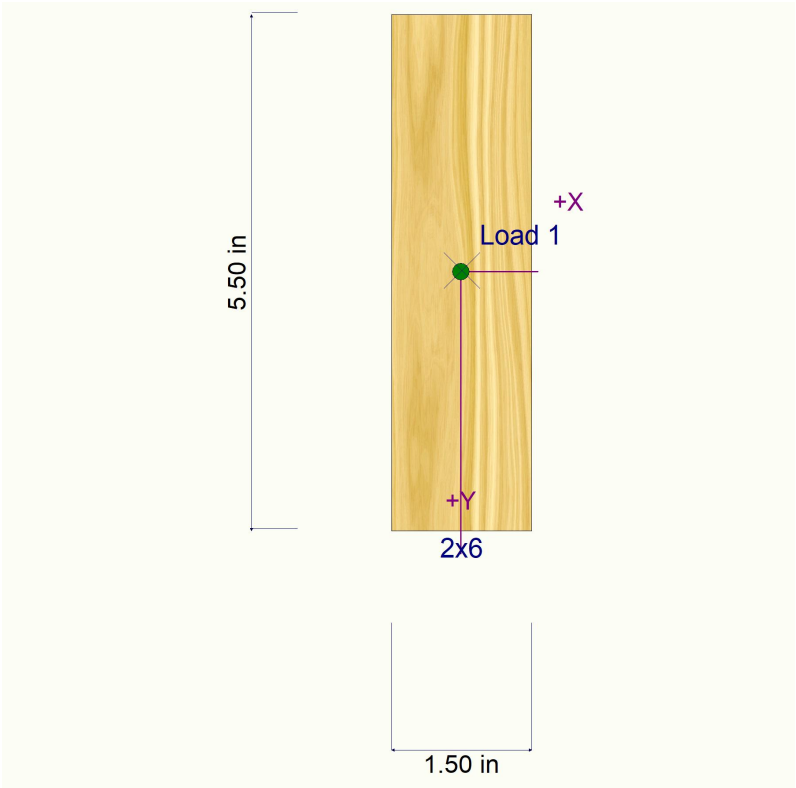
Maximum Deflections for Load Combinations

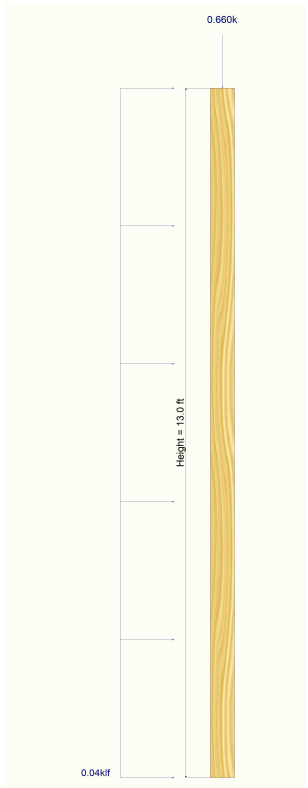
Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.750Lr	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.420W	0.0000 in	0.000 ft	0.3748 in	6.544 ft
+D+0.750Lr+0.320W	0.0000 in	0.000 ft	0.2856 in	6.544 ft
+D+0.750S+0.320W	0.0000 in	0.000 ft	0.2856 in	6.544 ft
+0.60D+0.420W	0.0000 in	0.000 ft	0.3748 in	6.544 ft
+0.60D	0.0000 in	0.000 ft	0.0000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+0.420W	0.0000 in	0.000 ft	0.3748 in	6.544 ft

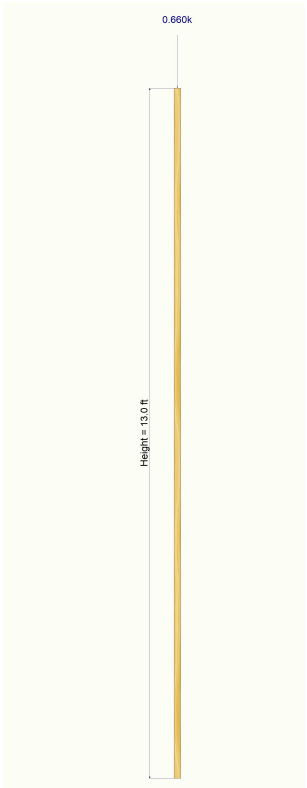
Wood Column

DESCRIPTION: High roof wall design

Sketches







Wood Beam

DESCRIPTION: Low roof top chord

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

Wood Species : Spruce-Pine-Fir
Wood Grade : No. 1/No. 2

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

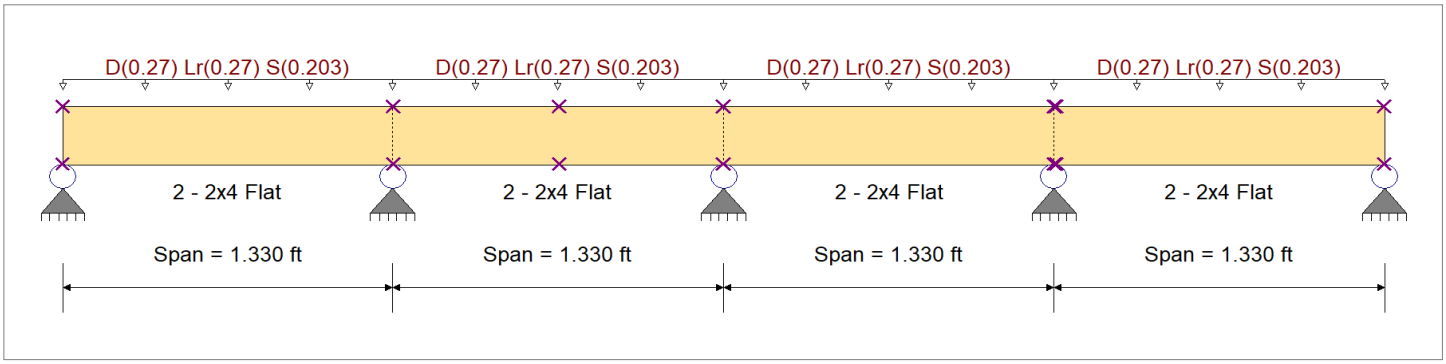
Fb +	875 psi
Fb -	875 psi
Fc - Prll	1150 psi
Fc - Perp	425 psi
Fv	135 psi
Ft	450 psi

E : Modulus of Elasticity

Ebend- xx	1400 ksi
Eminbend - xx	510 ksi
Density	26.22 pcf

Unbraced Lengths

First Brace starts at 0.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 2.0 ft



Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.270, Lr = 0.270, S = 0.2030 , Tributary Width = 1.0 ft
Load for Span Number 2
Uniform Load : D = 0.270, Lr = 0.270, S = 0.2030 , Tributary Width = 1.0 ft
Load for Span Number 3
Uniform Load : D = 0.270, Lr = 0.270, S = 0.2030 , Tributary Width = 1.0 ft
Load for Span Number 4
Uniform Load : D = 0.270, Lr = 0.270, S = 0.2030 , Tributary Width = 1.0 ft

(2) 2x6 FLATWISE CHOSEN FOR FINAL TOP CHORD DESIGN; ADEQUATE BY INSPECTION

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.215 : 1	Maximum Shear Stress Ratio	=	0.262 : 1
Section used for this span		2 - 2x4 Flat	Section used for this span		2 - 2x4 Flat
fb: Actual	=	234.76 psi	fv: Actual	=	44.27 psi
Fb: Allowable	=	1,093.75 psi	Fv: Allowable	=	168.75 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	1.330 ft	Location of maximum on span	=	1.094 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 <360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.002 in	Ratio =	9081 >=180
Max Upward Total Deflection	-0.000 in	Ratio =	268750 >=180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
D Only													0.00	0.00	0.00	0.00
Length = 1.330 ft	1	0.150	0.183	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.05	117.79	787.50	0.16	22.21	121.50

Wood Beam

DESCRIPTION: Low roof top chord

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
Length = 0.6566 ft	2	0.150	0.183	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.05	117.79	787.50	0.13	22.21	121.50
Length = 0.6734 ft	2	0.100	0.183	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.03	78.53	787.50	0.10	22.21	121.50
Length = 1.330 ft	3	0.150	0.183	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.05	117.79	787.50	0.16	22.21	121.50
Length = 1.330 ft	4	0.150	0.183	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.05	117.79	787.50	0.16	22.21	121.50
+D+Lr					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.330 ft	1	0.215	0.262	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.10	234.76	1093.75	0.31	44.27	168.75
Length = 0.6566 ft	2	0.215	0.262	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.10	234.76	1093.75	0.26	44.27	168.75
Length = 0.6734 ft	2	0.143	0.262	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.07	156.50	1093.75	0.21	44.27	168.75
Length = 1.330 ft	3	0.215	0.262	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.10	234.76	1093.75	0.31	44.27	168.75
Length = 1.330 ft	4	0.215	0.262	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.10	234.76	1093.75	0.31	44.27	168.75
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.330 ft	1	0.204	0.250	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.09	205.73	1006.25	0.27	38.79	155.25
Length = 0.6566 ft	2	0.204	0.250	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.09	205.73	1006.25	0.23	38.79	155.25
Length = 0.6734 ft	2	0.136	0.250	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.06	137.15	1006.25	0.18	38.79	155.25
Length = 1.330 ft	3	0.204	0.250	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.09	205.73	1006.25	0.27	38.79	155.25
Length = 1.330 ft	4	0.204	0.250	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.09	205.73	1006.25	0.27	38.79	155.25
+D+0.750Lr					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.330 ft	1	0.188	0.230	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.09	205.52	1093.75	0.27	38.75	168.75
Length = 0.6566 ft	2	0.188	0.230	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.09	205.52	1093.75	0.23	38.75	168.75
Length = 0.6734 ft	2	0.125	0.230	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.06	137.01	1093.75	0.18	38.75	168.75
Length = 1.330 ft	3	0.188	0.230	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.09	205.52	1093.75	0.27	38.75	168.75
Length = 1.330 ft	4	0.188	0.230	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.09	205.52	1093.75	0.27	38.75	168.75
+D+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.330 ft	1	0.183	0.223	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.08	183.75	1006.25	0.24	34.65	155.25
Length = 0.6566 ft	2	0.183	0.223	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.08	183.75	1006.25	0.20	34.65	155.25
Length = 0.6734 ft	2	0.122	0.223	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.05	122.50	1006.25	0.16	34.65	155.25
Length = 1.330 ft	3	0.183	0.223	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.08	183.75	1006.25	0.24	34.65	155.25
Length = 1.330 ft	4	0.183	0.223	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.08	183.75	1006.25	0.24	34.65	155.25
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 1.330 ft	1	0.050	0.062	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.03	70.68	1400.00	0.09	13.33	216.00
Length = 0.6566 ft	2	0.050	0.062	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.03	70.68	1400.00	0.08	13.33	216.00
Length = 0.6734 ft	2	0.034	0.062	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.02	47.12	1400.00	0.06	13.33	216.00
Length = 1.330 ft	3	0.050	0.062	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.03	70.68	1400.00	0.09	13.33	216.00
Length = 1.330 ft	4	0.050	0.062	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.03	70.68	1400.00	0.09	13.33	216.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0018	0.589		0.0000	0.000
+D+Lr	2	0.0005	0.724	+D+Lr	-0.0001	0.101
+D+Lr	3	0.0005	0.623	+D+Lr	-0.0000	1.246
+D+Lr	4	0.0017	0.758		0.0000	1.246

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
Overall MAXimum	0.283	0.824	0.669	0.824	0.283
Overall MINimum	0.106	0.309	0.251	0.309	0.106
D Only	0.142	0.413	0.336	0.413	0.142
+D+Lr	0.283	0.824	0.669	0.824	0.283
+D+S	0.248	0.722	0.587	0.722	0.248
+D+0.750Lr	0.248	0.721	0.586	0.721	0.248
+D+0.750S	0.222	0.645	0.524	0.645	0.222
+0.60D	0.085	0.248	0.201	0.248	0.085
Lr Only	0.141	0.410	0.333	0.410	0.141
S Only	0.106	0.309	0.251	0.309	0.106

Wood Column

DESCRIPTION: Low roof wall design

Code References

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-16

General Information

Analysis Method :	Allowable Stress Design		
End Fixities	Top & Bottom Pinned		
Overall Column Height	10 ft		
(Used for non-slender calculations)			
Wood Species	Spruce-Pine-Fir		
Wood Grade	No. 1/No. 2		
Fb +	875.0 psi	Fv	135.0 psi
Fb -	875.0 psi	Ft	450.0 psi
Fc - Prll	1,150.0 psi	Density	26.220 pcf
Fc - Perp	425.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,400.0	1,400.0	1,400.0 ksi
Minimum	510.0	510.0	

Wood Section Name	**2x6**		
Wood Grading/Manuf.	Graded Lumber		
Wood Member Type	Sawn		
Exact Width	**1.50** in	Allow Stress Modification Factors	
Exact Depth	**5.50** in	Cf or Cv for Bending	1.30
Area	8.250 in^2	Cf or Cv for Compression	1.10
Ix	20.797 in^4	Cf or Cv for Tension	1.30
Iy	1.547 in^4	Cm : Wet Use Factor	1.0
		Ct : Temperature Factor	1.0
		Cfu : Flat Use Factor	1.0
		Kf : Built-up columns	1.0 *NDS 15.3.2*
		Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Lu for buckling ABOUT Y-Y Axis : 4 ft, 6 ft, K = 1.0
Y-Y (depth) axis : Fully braced against buckling ABOUT X-X Axis

Applied Loads

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

Column self weight included : 15.022 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft, D = 0.360, Lr = 0.360, S = 0.270 k
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, W = 0.0440 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.3143** : 1
Load Combination +D+0.60W
Governing NDS Forumla 1Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base 4.027 ft
At maximum location values are . . .
Applied Axial 0.3750 k
Applied Mx 0.3175 k-ft
Applied My 0.0 k-ft
Fc : Allowable 390.702 psi

PASS Maximum Shear Stress Ratio = **0.1111** : 1
Load Combination +D+0.60W
Location of max.above base 10.0 ft
Applied Design Shear 24.0 psi
Allowable Shear 216.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.220 k	Bottom along Y-Y	0.220 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.1444 in at 5.034 ft above base
for load combination : +D+0.420W
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.328	0.1219	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+Lr	1.250	0.243	0.2316	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+S	1.150	0.263	0.2046	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+0.750Lr	1.250	0.243	0.2033	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+0.750S	1.150	0.263	0.1832	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+0.60W	1.600	0.193	0.3143	PASS	4.027 ft	0.1111	PASS	10.0 ft
+D+0.750Lr+0.450W	1.600	0.193	0.2656	PASS	4.027 ft	0.08333	PASS	10.0 ft
+D+0.750S+0.450W	1.600	0.193	0.2577	PASS	4.027 ft	0.08333	PASS	10.0 ft
+0.60D+0.60W	1.600	0.193	0.3056	PASS	4.027 ft	0.1111	PASS	10.0 ft

Wood Column

DESCRIPTION: Low roof wall design

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D	1.600	0.193	0.06981	PASS	0.0 ft	0.0	PASS	10.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
D Only						0.375					
+D+Lr						0.735					
+D+S						0.645					
+D+0.750Lr						0.645					
+D+0.750S						0.578					
+D+0.60W				0.132	0.132	0.375					
+D+0.750Lr+0.450W				0.099	0.099	0.645					
+D+0.750S+0.450W				0.099	0.099	0.578					
+0.60D+0.60W				0.132	0.132	0.225					
+0.60D						0.225					
Lr Only						0.360					
S Only						0.270					
W Only				0.220	0.220						

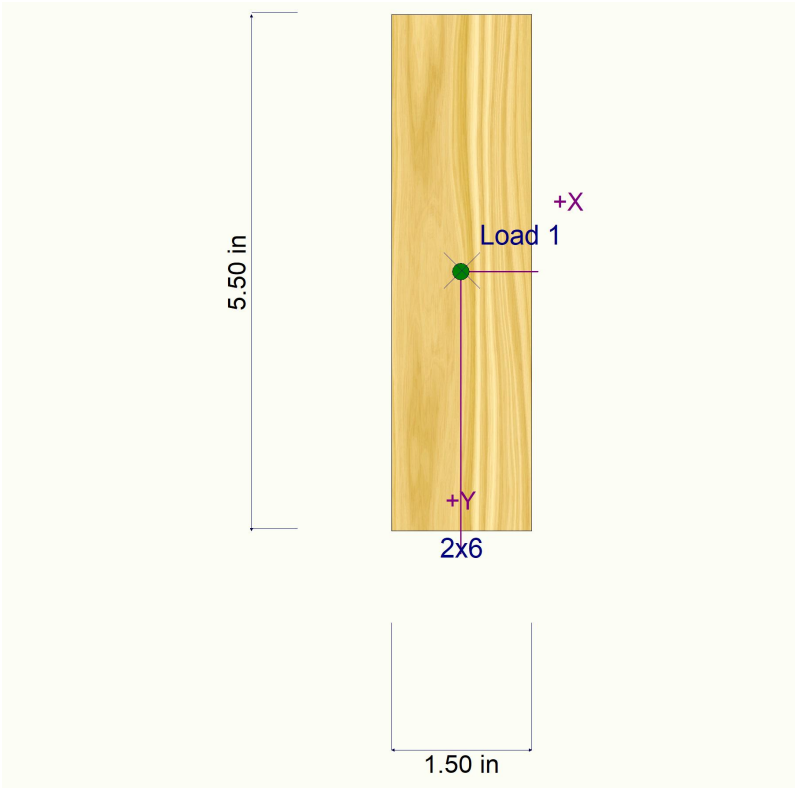
Maximum Deflections for Load Combinations

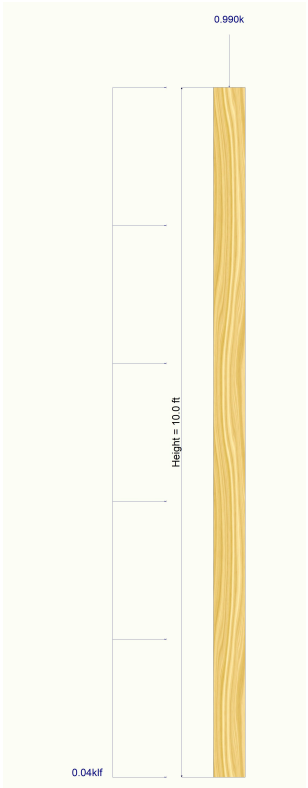
Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.750Lr	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+D+0.420W	0.0000 in	0.000 ft	0.1444 in	5.034 ft
+D+0.750Lr+0.320W	0.0000 in	0.000 ft	0.1100 in	5.034 ft
+D+0.750S+0.320W	0.0000 in	0.000 ft	0.1100 in	5.034 ft
+0.60D+0.420W	0.0000 in	0.000 ft	0.1444 in	5.034 ft
+0.60D	0.0000 in	0.000 ft	0.0000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.0000 in	0.000 ft
+0.420W	0.0000 in	0.000 ft	0.1444 in	5.034 ft

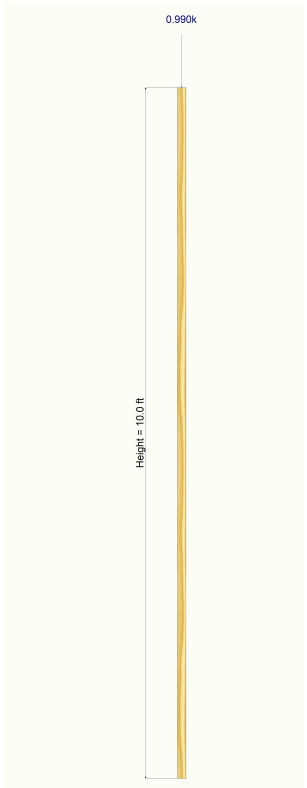
Wood Column

DESCRIPTION: Low roof wall design

Sketches







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FOUNDATION DESIGN

ASSUMED SOIL BEARING CAPACITY = 1500 psf

DETERMING TRENCH FOOTING WIDTH FOR GRAVITY

LOW ROOF WALL LOAD:

WALL = (15psf)(10') = 150 plf
ROOF DL = 270 plf
} TOTAL DL = 420plf

ROOF LIVE = 270plf
SNOW = 203 plf

$D+L_r = 420\ plf + 270\ plf = 690\ plf$

$q = \frac{P}{A}$

$1500\ psf = \frac{690\#}{(b)(1')}$

$b = 0.46' \longrightarrow$ 24" WIDE FTG. IS ADEQUATE

HIGH ROOF WALL LOAD:

WALL = (15psf)(13') = 195 plf
ROOF DL = 180plf
} 375plf total DL

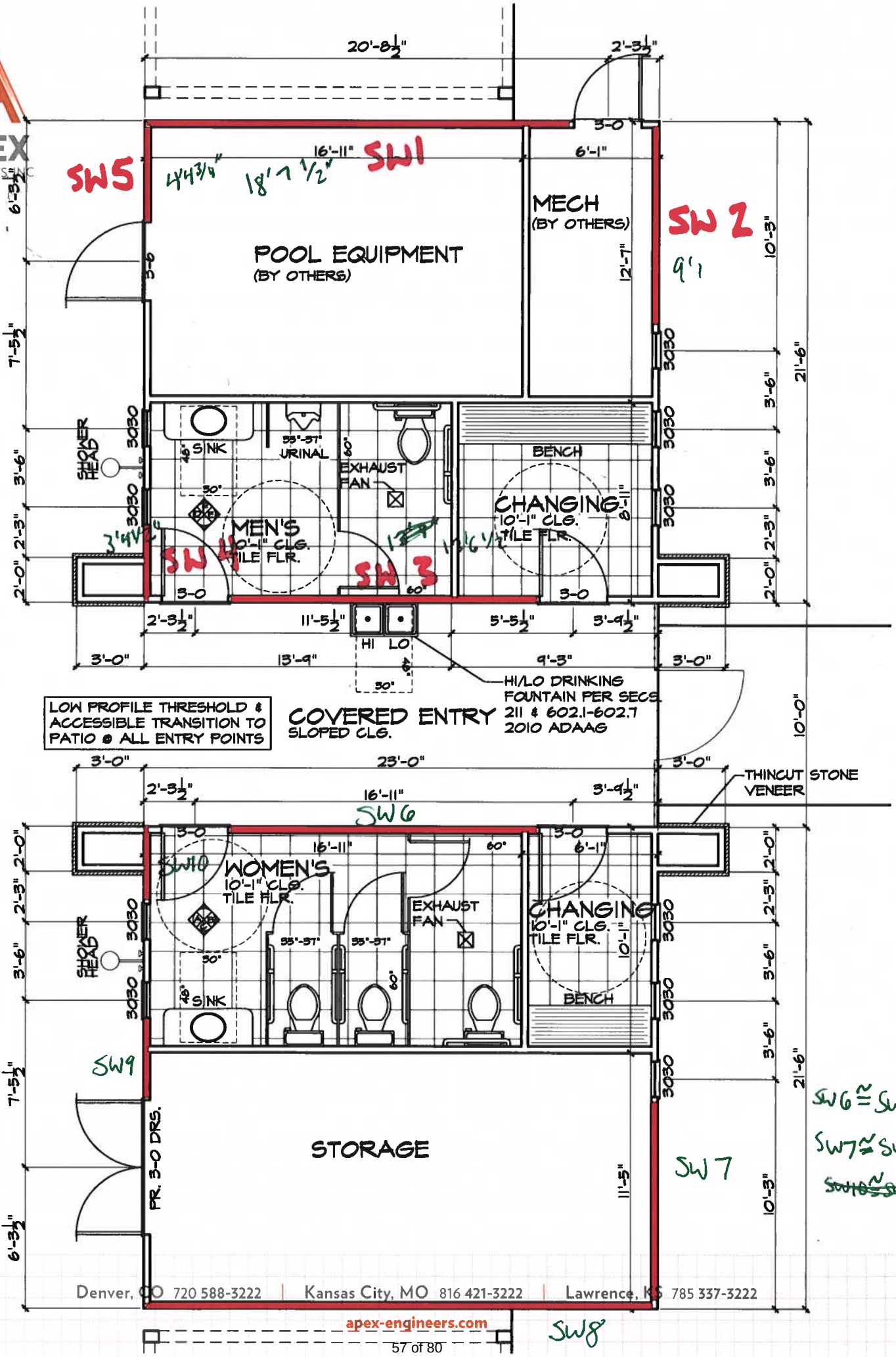
ROOF L_r = 180plf
S = 135plf

$D+L_r = 555\ plf$

$q = P/A$

$1500\ psf = \frac{555\ plf}{(b)(1')}$ OK by inspection 24" wide

Lateral Design


APEX ENGINEERS INC
SW5
SW1
SW2
SW3
SW4
SW6
SW7
SW8
SW9
SW10
POOL EQUIPMENT
(BY OTHERS)
MECH
(BY OTHERS)
MEN'S
10'-1" CLG.
TILE FLR.
WOMEN'S
10'-1" CLG.
TILE FLR.
CHANGING
10'-1" CLG.
TILE FLR.
BENCH
EXHAUST FAN
URINAL
SINK
SHOWER HEAD
HI LO
HI/LO DRINKING FOUNTAIN PER SECS 211 & 602.1-602.7 2010 ADAAG
LOW PROFILE THRESHOLD & ACCESSIBLE TRANSITION TO PATIO @ ALL ENTRY POINTS
COVERED ENTRY
SLOPED CLG.
THINCUT STONE VENEER
STORAGE
PR. 3-0 DRS.
SW6 ≅ SW3
SW7 ≅ SW2
SW10 ≅ SW4

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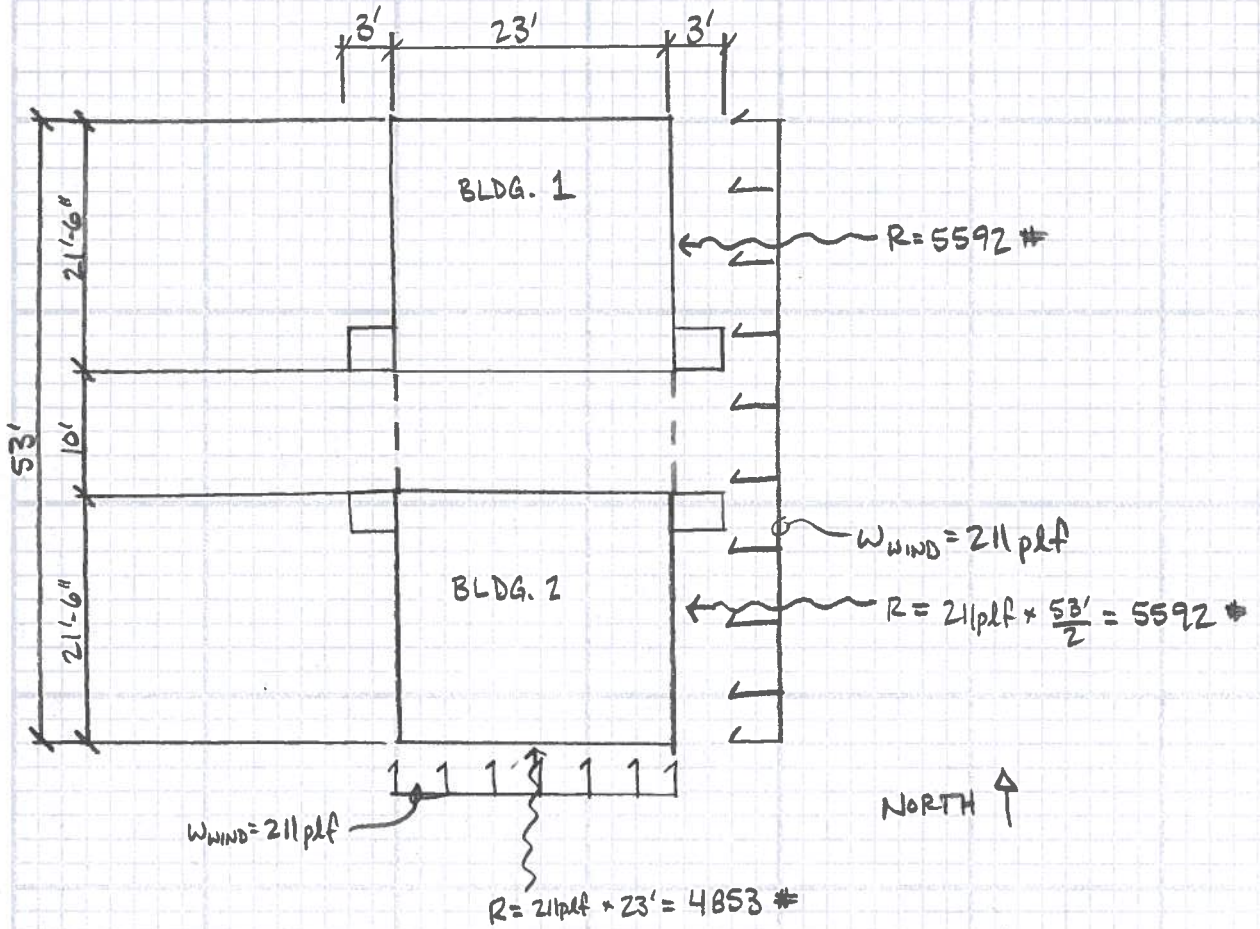
Project: 40390-SUMMIT VIEW FARMS Sheet: 1

Engineer: ACC

Date: 1/15/2021



EACH PORTION TREATED AS SEPARATE BUILDING FOR WALL & DIAPHRAGM DESIGN

WIND BLOWING NORTH/SOUTH:

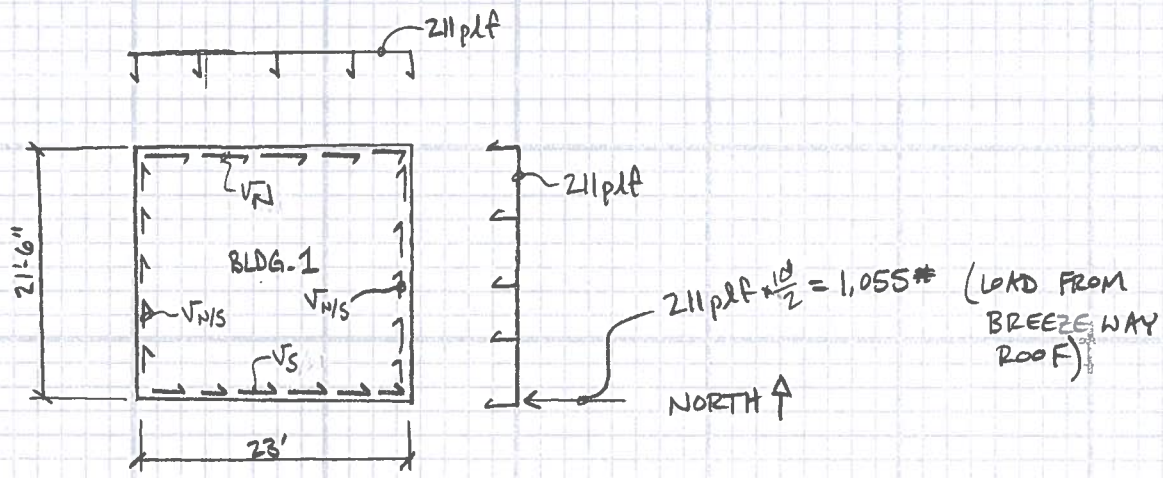
CHORD FORCES: $M = \frac{1}{8}wl^2 = \frac{(211plf)(23')^2}{8} = 13,952\ \#\text{-ft}$

$T = C = \frac{13,952\ \#}{21.5'} = 649\ \#$

WIND BLOWING EAST/WEST

CHORD FORCES: $M = \frac{(211plf)(53'/2)^2}{8} = 18,522\ \#\text{-ft}$

$T = C = \frac{18,522\ \#\text{-ft}}{23'} = 805\ \#$



EAST/WEST WIND

DIAPHRAGM SHEAR:

NORTH WALL, $V_N = \frac{211\text{plf} \times \frac{21.5'}{2}}{23'} = 99\text{ plf} \times 0.6\text{ (ASD)} = 60\text{ plf}$

SOUTH WALL, $V_S = 99\text{ plf} + \frac{1055\#}{23'} = 145\text{ plf} \times 0.6\text{ (ASD)} = 87\text{ plf}$

NORTH/SOUTH WIND

DIAPHRAGM SHEAR $V_{N/S} = \frac{\frac{211\text{plf} \times 23'}{2}}{21.5'} = 113\text{ plf} \times 0.6\text{ (ASD)} = 68\text{ plf}$

USE 7/16" SHEATHING WITH 8d NAILS; EDGE SPACING OF FASTENERS AT 6" OC AND FIELD SPACING AT 12" oc. UNBLOCKED DIAPHRAGM.

$\Omega = 2$; $V_a = \frac{645}{2} = 322\text{ plf}$ (NDS SDPWS TBL. 4.2C)

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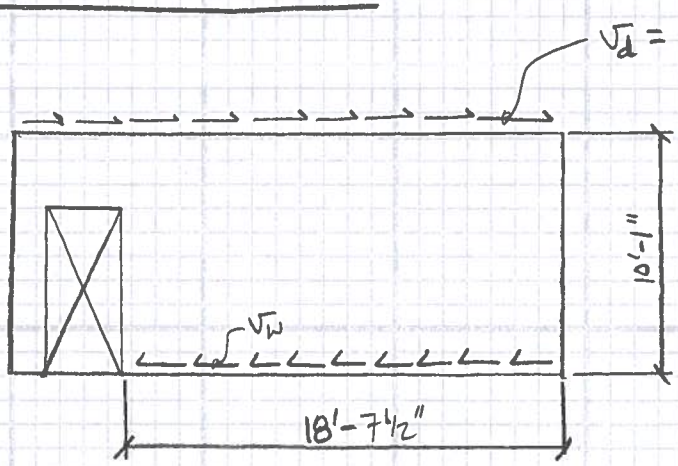
BUILDING 1 SHEAR WALLS



NORTH ELEVATION – SW1

WALL SHEAR, $v_w = \frac{99\,plf \times 23'}{18.625'} = 122\,plf \times 0.6\,(ASD) = 73plf$

ASPECT RATIO, $h/b = \frac{10.08'}{18.625'} = 0.54 < 2 \rightarrow$ USE UNBLOCKED

FOR 6" EDGE/12" FIELD SPACING & STUDS @ 16" oc THE UNBLOCKED SHEAR WALL ADJUSTMENT FACTOR, $C_{ub} = 0.6$

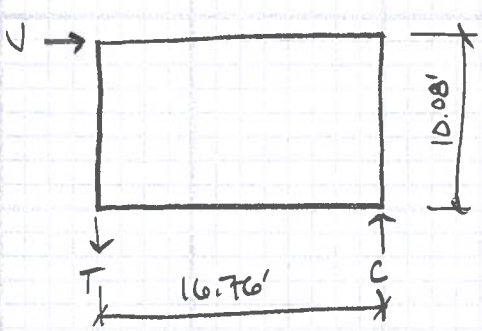
FOR 7/16" SHEATHING WITH 8d NAILS & 6" EDGE SPACING, $v_a = \frac{730\,plf}{2} = 365\,plf > v_w$ OK (730 plf: CAPACITY OF 15/32 USED FOR STUDS @ 16" oc)

SPECIES ADJUSTMENT FOR SPRUCE PINE FIR = 1 - (0.5 - 0.42) = 0.92

$v_a = (365\,plf)(0.92) = 336plf > v_w = 73plf$ OK

CHORD FORCES

CONSERVATIVELY USE $0.9 l_w = 0.9(18.625') = 16.76'$ TO DESIGN FOR CHORD & HOLDOWN FORCES



$V = .6(99plf)(23') = 1366\#$

$T = C = \frac{(1366\#)(10.08')}{16.76'} = 822\#$

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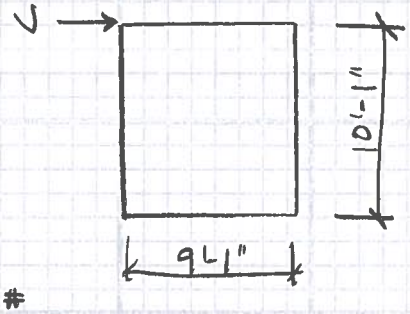
SW2

h = 10'-1"

b = 9'-1"



V = 68 plf x 21.5' = 1,462#

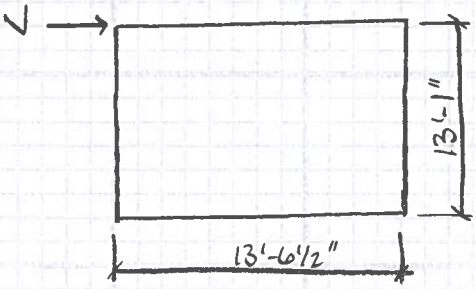
V_{ASD} = 0.6 (1,462#) = 877#

$V_{WALL} = \frac{877\#}{9.08'} = 97$ plf

SW3

h = 13'-1"

b = 13'-6½"

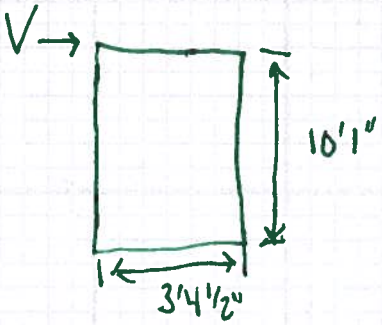


V = 145plf x 23' = 3335#

SW4

h = 10'-1"

b = 3'-4½"



$V_{full\ wall}$ = 21'6" x 113plf = 2429.5#

$V_{SW4} = 2429.5\# \times \frac{(3'-4\frac{1}{2}")}{(4'4\frac{3}{4}" + 3'4\frac{1}{2}")}$

V_{SW4} = 1055.2#

V_{SW5} = 2429.5# - 1055.2#

V_{SW5} = 1374.3#

SW 7+6 match SW 3+2

SW 8

h = 10'1"

b = 23'0"

V = 99 plf x 23' = 2,277 #

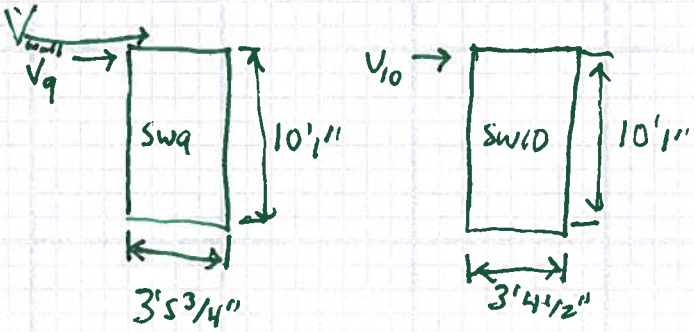
V = 2277 #

SW9 & SW10

h = 10'1"

b_9 = 3'5 3/4"

b_{10} = 3'4 1/2"



V_{wall} = 21'6" x 113 plf = 2429.5 #

V_9 = 2429.5 # x $\frac{3'5\,3/4"}{(3'4\,1/2" + 3'5\,3/4")}$ = 1233.2 #

V_{10} = 2429.5 # − 1233.2 # = 1196.3 #

WIND BLOWING EAST/WEST

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson
Engineer:	AC & GP
Date:	1/15/2021
Description:	Diaphragm Design

Diaphragm Sheathing Capacity

Wall Sheathing Thickness "t"	7/16 in				
Span Rating	24/16				
Fastener Size	8	D nails			
Fastener Notes					
Fastener Spacing @ -Edges	6 in OC				
-Intermediate	12 in OC				
Total Diaphragm Length "L"	26.50 ft				
Total Diaphragm Width "b"	23.00 ft				
Diaphragm Ratio Check	1.15 : 1	OK			3
Diaphragm Blocked ?		enter "x" if true			
Unit Length Wall Shear Capacity "v"	230 plf	unblocked			
Unit Length Wall Shear Capacity "v"	255 plf	blocked			
Shear Load for Wind Only	x	enter "x" if true (40% increase for wind load only)			
Unit Length Loading Adjustment "v"	322 plf		unblocked		
Unit Length Loading Adjustment "v"	357 plf		blocked		
Sheathing Ply # adjustment "v"	322 plf			Wind	Seismic
Final Unit Length Shear Capacity "v"	322 plf			322	322

Diaphragm Unit Shear Capacity "V"

Total Diaphragm Design Load, "V"	5592 lb	
Diaphragm Design Unit Shear	122 plf	OK
Distance blocking no longer req'd	-21.85 ft	(note: will be negative if blck'ing not req'd)

Chord Force/Splice Calculations

Diaphragm Moment	18524 lb-ft				
Maximum Chord Force Tension "T"	805 lb				
Minimum Chord Member Size "T"	(1) 2x4	or	(1) 2x6	SPF #3 (or better)	
Maximum Chord Force Compression "C"	805 lb				
Minimum Chord Member Size "C"	(1) 2x4	or	(1) 2x6	SPF #3 (or better)	
Splice Calculation	4	16D nails/ each side splice			
Edge Nailing - Diaphragm panel to top plate	8	8 D Nails Req'd			
8 D Nails @	38 in OC	O.C.	E-W walls		

Anchorage Force for In-Plane Shear

In-Plane Shear "vmax" =	122 plf		
Edge Nailing - Diaphragm panel to top plate	1.27		
8 D Nails @	9 in OC	O.C.	N-S walls

Diaphragm Deflection

Maximum Shear per foot	122 plf			
Top Plate Depth	3.00 in			
Top Plate Width	3.50 in			
Chord Area "a"	10.50 in²			
Mod. of Elasticity @ boundary element "E"	1200000 psi	(assumed SPF)		
Nail or Staple Deformation "E_n"	0.020			
Average Chord Splice Slip ($\sum D_c X$)	0.922			-1.3375
Panel Rigidity (per panel thickness) "Gt"	36,531		lookup value	83,500
$\Delta = (5vL^3 / 8Eab) + (vL / 4Gt) + (0.188Le_n) + (\sum D_c X / 2b)$	0.15 in			
	L/2169			

WIND BLOWING NORTH/SOUTH

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson
Engineer:	AC & GP
Date:	1/15/2021
Description:	Diaphragm Design

Diaphragm Sheathing Capacity

Wall Sheathing Thickness "t"	7/16 in				
Span Rating	24/16				
Fastener Size	8	D nails			
Fastener Notes					
Fastener Spacing @ -Edges	6 in OC				
-Intermediate	12 in OC				
Total Diaphragm Length "L"	23.00 ft				
Total Diaphragm Width "b"	21.50 ft				
Diaphragm Ratio Check	1.07 : 1	OK			3
Diaphragm Blocked ?		enter "x" if true			
Unit Length Wall Shear Capacity "v"	230 plf	unblocked			
Unit Length Wall Shear Capacity "v"	255 plf	blocked			
Shear Load for Wind Only	x	enter "x" if true (40% increase for wind load only)			
Unit Length Loading Adjustment "v"	322 plf		unblocked		
Unit Length Loading Adjustment "v"	357 plf		blocked		
Sheathing Ply # adjustment "v"	322 plf			Wind	Seismic
Final Unit Length Shear Capacity "v"	322 plf			322	322

Diaphragm Unit Shear Capacity "V"

Total Diaphragm Design Load, "V"	4853 lb	
Diaphragm Design Unit Shear	113 plf	OK
Distance blocking no longer req'd	-21.31 ft	(note: will be negative if blck'ing not req'd)

Chord Force/Splice Calculations

Diaphragm Moment	13952 lb-ft			
Maximum Chord Force Tension "T"	649 lb			
Minimum Chord Member Size "T"	(1) 2x4	or	(1) 2x6	SPF #3 (or better)
Maximum Chord Force Compression "C"	649 lb			
Minimum Chord Member Size "C"	(1) 2x4	or	(1) 2x6	SPF #3 (or better)
Splice Calculation	3	16D nails/ each side splice		
Edge Nailing - Diaphragm panel to top plate	7	8 D Nails Req'd		
8 D Nails @	41 in OC	O.C.	E-W walls	

Anchorage Force for In-Plane Shear

In-Plane Shear "vmax" =	113 plf		
Edge Nailing - Diaphragm panel to top plate	1.18		
8 D Nails @	10 in OC	O.C.	N-S walls

Diaphragm Deflection

Maximum Shear per foot	113 plf			
Top Plate Depth	3.00 in			
Top Plate Width	3.50 in			
Chord Area "a"	10.50 in²			
Mod. of Elasticity @ boundary element "E"	1200000 psi	(assumed SPF)		
Nail or Staple Deformation "E_n"	0.010			
Average Chord Splice Slip ($\sum D_c X$)	0.828			-1.425
Panel Rigidity (per panel thickness) "Gt"	36,531		lookup value	83,500
$\Delta = (5vL^3 / 8Eab) + (vL / 4Gt) + (0.188Le_n) + (\sum D_c X / 2b)$	0.08 in			
	L/3309			

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 1

WOOD SHEAR WALL CALCULATION

Design Philosophy and Code	ASD 7-16 / Wind	
Developed Load on shear wall R	2272 lb	
Factored Load Applied - R	1363 lb	Developed Load * 0.6
Shear Wall Type	Segmented	
Material(s) Used	Wood Sheathing	
Wood Sheathing Type	OSB	
Thickness of Material on first side	7/16 in	Using 15/32 Capacity
Thickness of Material on alt side	N/A	
# of sides sheathed	1	
# of layers on second side	N/A	
Fasteners for first side	8d	
Fasteners for opposite face	N/A	
Fastener Edge Spacing	6	
Fastener Edge Spacing opp Face	N/A	
Stud Spacing	16 in	
Blocked/Unblocked	Blocked	
Stud Species	Spruce Pine Fir	
Wall Thickness	2x6	
Plate to Plate Height - h	10.08 ft	
Shear wall segment length b min = 2.88'	18.63 ft	Aspect Ratio = 0.5 : 1
Number of chord studs	2	
Additional T/C Load on chord studs (lbs)		Note: Optional

Design Summary		
Panel Shear Strength		
V_{allow} =	336 plf	6254 lb
V_{act} =	73 plf	1363 lb
Ratio	**0.22**	**PASS**
Tension/Compression Chord Strength		
F'c =	611 psi	
F't =	560 psi	
f_{act} =	45 psi	736 lb
Ratio	**0.08**	**PASS**
Deflection		
δ_{swAct}=	0.09 in	
Ratio	L/2688	
Base plate Compression		
$F'_{c\perp allow}$ =	425 psi	
$F'_{c\perp act}$	45 psi	
Ratio	**0.1**	**PASS**
Anchorage Devices		
Chord Hold Downs		
Hold Down	Ratio	
HDU2	**0.33**	**PASS**
Studs Reqd	2	
Anchor Require	5/8 in	
CS22	**0.87**	**PASS**
End length	7 in	
Sill Plate Anchors		
1/2 Bolts at	36" OC	
1/2 Titen HD at	36" OC	
16d	1 per LF	12 in OC
#10 Screws	1 per LF	12 in OC

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 1

Shear Wall Design Calculations

Panel Shear Strength

Nominal Shear Capacity	672 plf	
Allowable Shear Capacity -Vn/Ω	336 plf	
Shear wall base -b (ft)	18.63 ft	
Total area of opening - A_o	0.00 ft²	
Sheathing area ratio - r	1.00	eq (4.3-6)
Adjustment Factor Co	1.00	eq (4.3-5)

Vwall = (Vn/Ω*b)

$v_{wall\ 1}$ =	336 plf
$V_{wall\ 1}$ =	6254 lb

Total Panel Shear Strength *V*

Vtot =	336 plf

Tension/Compression Chord Strength

T/C force on chords	736 lb	
T/C force on chords w/ add. Loads	736 lb	Additional: 0 lbs
Area of Chord Member Used	16.50 in²	
Stress on Chord Member	45 psi	
Load duration factor Cd	1.6	
Compr. Member Size Factor Cf	1	
Tens. Member Size Factor Cf	1	

Member properties

Ft =	350 psi
Fc =	725 psi
F_{cE}/F^*_c=	0.64
Column Stability Fctr Cp	0.53

Tension Member Strength

F't = Ft(Cd)(Cm)(Ct)(Cf) NDS 4.3.1

F't =	560 psi

Compression Member Strength

F'c = Fc(Cd)(Cm)(Ct)(Cf)(Cp) NDS 4.3.1

F'c =	611 psi

Deflection

Bending deflection

$\Delta_b = 8v'^*h^3/(E^*A^*b)$ eq (4.3-1)

Δ_b = 0.00 in

Shear deflection and Nail Slip

$\Delta_{v+n} = v'^*h/(1000^*Ga)$ eq. (4.3-1)

Δ_{v+n} = 0.06 in

Anchorage slip

$\Delta_{anchor} = h^*\Delta_a/b$ eq. (4.3-1)

Δ_{anchor}= 0.03 in

Total Deflection

δsw=	0.09 in

Base plate Compression

Stress on baseplate 45 psi

Sill Compression Strength

F'c⊥ = Fc⊥(Cm)(Ct)(Cb) NDS 4.3.1

F'c⊥=	425 psi

Member properties

$F_{c\perp}$=	425 psi
Wet service Factor Cm	1.0
Temperature Factor Ct	1.0
Bearing Area Factor Cb	1.0

Anchorage Devices

Max uplift on chords

R_{max} = 736 lb

Hold down required at foundation

Hold down = HDU2

Hold Down Cap. 2215 lb

Hold down required between floors

Hold Down = CS22

Hold Down Cap. 845 lb

Sole Plate Attachment

Base shear 73 plf

Foundation Anchors

Standard Bolts	Max Spacing	Titen HD	Max Spacing
1/2	36.0 in	1/2	36.0 in
5/8	36.0 in	5/8	36.0 in
3/4	36.0 in		
7/8	36.0 in		
1	36.0 in		

Wood Anchors

16d Nails at	1 per LF
Stress Ratio =	0.44
#10 screws at	1 per LF
Stress Ratio =	0.39

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 2 & 7

WOOD SHEAR WALL CALCULATION

Design Philosophy and Code	ASD 7-16 / Wind	
Developed Load on shear wall R	1462 lb	
Factored Load Applied - R	877 lb	Developed Load * 0.6
Shear Wall Type	Segmented	
Material(s) Used	Wood Sheathing	
Wood Sheathing Type	OSB	
Thickness of Material on first side	7/16 in	Using 15/32 Capacity
Thickness of Material on alt side	N/A	
# of sides sheathed	1	
# of layers on second side	N/A	
Fasteners for first side	8d	
Fasteners for opposite face	N/A	
Fastener Edge Spacing	6	
Fastener Edge Spacing opp Face	N/A	
Stud Spacing	16 in	
Blocked/Unblocked	Blocked	
Stud Species	Spruce Pine Fir	
Wall Thickness	2x6	
Plate to Plate Height - h	10.08 ft	
Shear wall segment length b min = 2.88'	9.08 ft	Aspect Ratio = 1.1 : 1
Number of chord studs	2	
Additional T/C Load on chord studs (lbs)		Note: Optional

Design Summary		
Panel Shear Strength		
V_{allow} =	336 plf	3049 lb
V_{act} =	97 plf	877 lb
Ratio	**0.29**	**PASS**
Tension/Compression Chord Strength		
F'c =	611 psi	
F't =	560 psi	
f_{act} =	59 psi	978 lb
Ratio	**0.11**	**PASS**
Deflection		
δ_{swAct}=	0.12 in	
Ratio	L/2016	
Base plate Compression		
$F'_{c\perp allow}$ =	425 psi	
$F'_{c\perp act}$	59 psi	
Ratio	**0.14**	**PASS**
Anchorage Devices		
Chord Hold Downs		
Hold Down	Ratio	
HDU2	**0.44**	**PASS**
Studs Reqd	2	
Anchor Require	5/8 in	
CS20	**0.95**	**PASS**
End length	9 in	
Sill Plate Anchors		
1/2 Bolts at	36" OC	
1/2 Titen HD at	36" OC	
16d	1 per LF	12 in OC
#10 Screws	1 per LF	12 in OC

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 2 & 7

Shear Wall Design Calculations

Panel Shear Strength

Nominal Shear Capacity	672 plf	
Allowable Shear Capacity -Vn/Ω	336 plf	
Shear wall base -b (ft)	9.08 ft	
Total area of opening - A_o	0.00 ft²	
Sheathing area ratio - r	1.00	eq (4.3-6)
Adjustment Factor Co	1.00	eq (4.3-5)

Vwall = (Vn/Ω*b)

$v_{wall\ 1}$ =	336 plf
$V_{wall\ 1}$ =	3049 lb

Total Panel Shear Strength *V*

Vtot =	336 plf

Tension/Compression Chord Strength

T/C force on chords	978 lb	
T/C force on chords w/ add. Loads	978 lb	Additional: 0 lbs
Area of Chord Member Used	16.50 in²	
Stress on Chord Member	59 psi	
Load duration factor Cd	1.6	
Compr. Member Size Factor Cf	1	
Tens. Member Size Factor Cf	1	

Member properties

Ft =	350 psi
Fc =	725 psi
F_{cE}/F^*_c=	0.64
Column Stability Fctr Cp	0.53

Tension Member Strength

F't = Ft(Cd)(Cm)(Ct)(Cf) NDS 4.3.1

F't =	560 psi

Compression Member Strength

F'c = Fc(Cd)(Cm)(Ct)(Cf)(Cp) NDS 4.3.1

F'c =	611 psi

Deflection

Bending deflection

$\Delta_b = 8v'^*h^3/(E^*A^*b)$ eq (4.3-1)

Δ_b = 0.00 in

Shear deflection and Nail Slip

$\Delta_{v+n} = v'^*h/(1000^*Ga)$ eq. (4.3-1)

Δ_{v+n} = 0.08 in

Anchorage slip

$\Delta_{anchor} = h^*\Delta_a/b$ eq. (4.3-1)

Δ_{anchor}= 0.04 in

Total Deflection

δsw=	0.12 in

Base plate Compression

Stress on baseplate 59 psi

Member properties

$F_{c\perp}$=	425 psi
Wet service Factor Cm	1.0
Temperature Factor Ct	1.0
Bearing Area Factor Cb	1.0

Sill Compression Strength

F'c⊥ = Fc⊥(Cm)(Ct)(Cb) NDS 4.3.1

F'c⊥=	425 psi

Anchorage Devices

Max uplift on chords

R_{max} = 978 lb

Hold down required at foundation

Hold down = HDU2

Hold Down Cap. 2215 lb

Hold down required between floors

Hold Down = CS20

Hold Down Cap. 1030 lb

Sole Plate Attachment

Base shear 97 plf

Foundation Anchors

Standard Bolts	Max Spacing	Titen HD	Max Spacing
1/2	36.0 in	1/2	36.0 in
5/8	36.0 in	5/8	36.0 in
3/4	36.0 in		
7/8	36.0 in		
1	36.0 in		

Wood Anchors

16d Nails at	1 per LF
Stress Ratio =	0.59
#10 screws at	1 per LF
Stress Ratio =	0.52

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 3 & 6

WOOD SHEAR WALL CALCULATION

Design Philosophy and Code	ASD 7-16 / Wind	
Developed Load on shear wall R	3335 lb	
Factored Load Applied - R	2001 lb	Developed Load * 0.6
Shear Wall Type	Segmented	
Material(s) Used	Wood Sheathing	
Wood Sheathing Type	OSB	
Thickness of Material on first side	7/16 in	Using 15/32 Capacity
Thickness of Material on alt side	N/A	
# of sides sheathed	1	
# of layers on second side	N/A	
Fasteners for first side	8d	
Fasteners for opposite face	N/A	
Fastener Edge Spacing	6	
Fastener Edge Spacing opp Face	N/A	
Stud Spacing	16 in	
Blocked/Unblocked	Blocked	
Stud Species	Spruce Pine Fir	
Wall Thickness	2x6	
Plate to Plate Height - h	13.09 ft	
Shear wall segment length b min = 3.74'	13.54 ft	Aspect Ratio = 1 : 1
Number of chord studs	2	
Additional T/C Load on chord studs (lbs)		Note: Optional

Design Summary		
Panel Shear Strength		
V_{allow} =	336 plf	4547 lb
V_{act} =	148 plf	2001 lb
Ratio	**0.44**	**PASS**
Tension/Compression Chord Strength		
F'c =	401 psi	
F't =	560 psi	
f_{act} =	117 psi	1937 lb
Ratio	**0.29**	**PASS**
Deflection		
δ_{swAct}=	0.24 in	
Ratio	L/1309	
Base plate Compression		
$F'_{c\perp allow}$ =	425 psi	
$F'_{c\perp act}$	117 psi	
Ratio	**0.28**	**PASS**
Anchorage Devices		
Chord Hold Downs		
Hold Down	Ratio	
HDU2	**0.87**	**PASS**
Studs Reqd	2	
Anchor Require	5/8 in	
CS14	**0.78**	**PASS**
End length	16 in	
Sill Plate Anchors		
1/2 Bolts at	36" OC	
1/2 Titen HD at	36" OC	
16d	2 per LF	6 in OC
#10 Screws	1 per LF	12 in OC

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 3 & 6

Shear Wall Design Calculations

Panel Shear Strength

Nominal Shear Capacity	672 plf	
Allowable Shear Capacity -Vn/Ω	336 plf	
Shear wall base -b (ft)	13.54 ft	
Total area of opening - A_o	0.00 ft²	
Sheathing area ratio - r	1.00	eq (4.3-6)
Adjustment Factor Co	1.00	eq (4.3-5)

Vwall = (Vn/Ω*b)

$v_{wall\ 1}$ =	336 plf
$V_{wall\ 1}$ =	4547 lb

Total Panel Shear Strength *V*

Vtot =	336 plf

Tension/Compression Chord Strength

T/C force on chords	1937 lb	
T/C force on chords w/ add. Loads	1937 lb	Additional: 0 lbs
Area of Chord Member Used	16.50 in²	
Stress on Chord Member	117 psi	
Load duration factor Cd	1.6	
Compr. Member Size Factor Cf	1	
Tens. Member Size Factor Cf	1	

Member properties

Ft =	350 psi
Fc =	725 psi
F_{cE}/F^*_c=	0.38
Column Stability Fctr Cp	0.35

Tension Member Strength

F't = Ft(Cd)(Cm)(Ct)(Cf) NDS 4.3.1

F't =	560 psi

Compression Member Strength

F'c = Fc(Cd)(Cm)(Ct)(Cf)(Cp) NDS 4.3.1

F'c =	401 psi

Deflection

Bending deflection

$\Delta_b = 8v'*h^3/(E*A*b)$ eq (4.3-1)

Δ_b = 0.01 in

Shear deflection and Nail Slip

$\Delta_{v+n} = v'*h/(1000*Ga)$ eq. (4.3-1)

Δ_{v+n} = 0.15 in

Anchorage slip

$\Delta_{anchor} = h*\Delta_a/b$ eq. (4.3-1)

Δ_{anchor}= 0.08 in

Total Deflection

δsw=	0.24 in

Base plate Compression

Stress on baseplate	117 psi

Member properties

$F_{c\perp}$=	425 psi
Wet service Factor Cm	1.0
Temperature Factor Ct	1.0
Bearing Area Factor Cb	1.0

Sill Compression Strength

F'c⊥ = Fc⊥(Cm)(Ct)(Cb) NDS 4.3.1

F'c⊥=	425 psi

Anchorage Devices

Max uplift on chords

R_{max} = 1937 lb

Hold down required at foundation

Hold down = HDU2

Hold Down Cap. 2215 lb

Hold down required between floors

Hold Down = CS14

Hold Down Cap. 2490 lb

Sole Plate Attachment

Base shear 148 plf

Foundation Anchors

Standard Bolts	Max Spacing	Titen HD	Max Spacing
1/2	36.0 in	1/2	36.0 in
5/8	36.0 in	5/8	36.0 in
3/4	36.0 in		
7/8	36.0 in		
1	36.0 in		

Wood Anchors

16d Nails at	2 per LF
Stress Ratio =	0.45
#10 screws at	1 per LF
Stress Ratio =	0.79

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 4

WOOD SHEAR WALL CALCULATION

Design Philosophy and Code	ASD 7-16 / Wind	
Developed Load on shear wall R	1055 lb	
Factored Load Applied - R	633 lb	Developed Load * 0.6
Shear Wall Type	Segmented	
Material(s) Used	Wood Sheathing	
Wood Sheathing Type	OSB	
Thickness of Material on first side	7/16 in	Using 15/32 Capacity
Thickness of Material on alt side	N/A	
# of sides sheathed	1	
# of layers on second side	N/A	
Fasteners for first side	8d	
Fasteners for opposite face	N/A	
Fastener Edge Spacing	6	
Fastener Edge Spacing opp Face	N/A	
Stud Spacing	16 in	
Blocked/Unblocked	Blocked	
Stud Species	Spruce Pine Fir	
Wall Thickness	2x6	
Plate to Plate Height - h	10.08 ft	
Shear wall segment length b min = 2.88'	3.38 ft	Aspect Ratio = 3 : 1
Number of chord studs	2	
Additional T/C Load on chord studs (lbs)		Note: Optional

Design Summary		
Panel Shear Strength		
V_{allow} =	295 plf	996 lb
V_{act} =	187 plf	633 lb
Ratio	0.63	PASS
Tension/Compression Chord Strength		
F'c =	611 psi	
F't =	560 psi	
f_{act} =	114 psi	1885 lb
Ratio	0.2	PASS
Deflection		
δ_{swAct}=	0.24 in	
Ratio	L/1008	
Base plate Compression		
$F'_{c\perp allow}$ =	425 psi	
$F'_{c\perp act}$	114 psi	
Ratio	0.27	PASS
Anchorage Devices		
Chord Hold Downs		
Hold Down	Ratio	
HDU2	0.85	PASS
Studs Reqd	2	
Anchor Require	5/8 in	
CS14	0.76	PASS
End length	16 in	
Sill Plate Anchors		
1/2 Bolts at	36" OC	
1/2 Titen HD at	36" OC	
16d	2 per LF	6 in OC
#10 Screws	2 per LF	6 in OC

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 4

Shear Wall Design Calculations

Panel Shear Strength

Nominal Shear Capacity	589 plf	
Allowable Shear Capacity -Vn/Ω	295 plf	
Shear wall base -b (ft)	3.38 ft	
Total area of opening - A_o	0.00 ft²	
Sheathing area ratio - r	1.00	eq (4.3-6)
Adjustment Factor Co	1.00	eq (4.3-5)

Vwall = (Vn/Ω*b)

$v_{wall\ 1}$ =	295 plf
$V_{wall\ 1}$ =	996 lb

Total Panel Shear Strength *V*

Vtot =	295 plf

Tension/Compression Chord Strength

T/C force on chords	1885 lb	
T/C force on chords w/ add. Loads	1885 lb	Additional: 0 lbs
Area of Chord Member Used	16.50 in²	
Stress on Chord Member	114 psi	
Load duration factor Cd	1.6	
Compr. Member Size Factor Cf	1	
Tens. Member Size Factor Cf	1	

Member properties

Ft =	350 psi
Fc =	725 psi
F_{cE}/F^*_c=	0.64
Column Stability Fctr Cp	0.53

Tension Member Strength

F't = Ft(Cd)(Cm)(Ct)(Cf) NDS 4.3.1

F't =	560 psi

Compression Member Strength

F'c = Fc(Cd)(Cm)(Ct)(Cf)(Cp) NDS 4.3.1

F'c =	611 psi

Deflection

Bending deflection

$\Delta_b = 8v'*h^3/(E*A*b)$ eq (4.3-1)

Δ_b = 0.02 in

Shear deflection and Nail Slip

$\Delta_{v+n} = v'*h/(1000*Ga)$ eq. (4.3-1)

Δ_{v+n} = 0.14 in

Anchorage slip

$\Delta_{anchor} = h*\Delta_a/b$ eq. (4.3-1)

Δ_{anchor}= 0.07 in

Total Deflection

δsw=	0.24 in

Base plate Compression

Stress on baseplate	114 psi

Member properties

$F_{c\perp}$=	425 psi
Wet service Factor Cm	1.0
Temperature Factor Ct	1.0
Bearing Area Factor Cb	1.0

Sill Compression Strength

F'c⊥ = Fc⊥(Cm)(Ct)(Cb) NDS 4.3.1

F'c⊥=	425 psi

Anchorage Devices

Max uplift on chords

R_{max} = 1885 lb

Hold down required at foundation

Hold down = HDU2

Hold Down Cap. 2215 lb

Hold down required between floors

Hold Down = CS14

Hold Down Cap. 2490 lb

Sole Plate Attachment

Base shear 187 plf

Foundation Anchors

Standard Bolts	Max Spacing	Titen HD	Max Spacing
1/2	36.0 in	1/2	36.0 in
5/8	36.0 in	5/8	36.0 in
3/4	36.0 in		
7/8	36.0 in		
1	36.0 in		

Wood Anchors

16d Nails at	2 per LF
Stress Ratio =	0.57
#10 screws at	2 per LF
Stress Ratio =	0.5

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 5

WOOD SHEAR WALL CALCULATION

Design Philosophy and Code	ASD 7-16 / Wind	
Developed Load on shear wall R	1374 lb	
Factored Load Applied - R	824 lb	Developed Load * 0.6
Shear Wall Type	Segmented	
Material(s) Used	Wood Sheathing	
Wood Sheathing Type	OSB	
Thickness of Material on first side	7/16 in	Using 15/32 Capacity
Thickness of Material on alt side	N/A	
# of sides sheathed	1	
# of layers on second side	N/A	
Fasteners for first side	8d	
Fasteners for opposite face	N/A	
Fastener Edge Spacing	6	
Fastener Edge Spacing opp Face	N/A	
Stud Spacing	16 in	
Blocked/Unblocked	Blocked	
Stud Species	Spruce Pine Fir	
Wall Thickness	2x6	
Plate to Plate Height - h	10.08 ft	
Shear wall segment length b min = 2.88'	4.40 ft	Aspect Ratio = 2.3 : 1
Number of chord studs	2	
Additional T/C Load on chord studs (lbs)		Note: Optional

Design Summary		
Panel Shear Strength		
V_{allow} =	324 plf	1424 lb
V_{act} =	187 plf	824 lb
Ratio	**0.58**	**PASS**
Tension/Compression Chord Strength		
F'c =	611 psi	
F't =	560 psi	
f_{act} =	114 psi	1885 lb
Ratio	**0.2**	**PASS**
Deflection		
δ_{swAct}=	0.24 in	
Ratio	L/1008	
Base plate Compression		
$F'_{c\perp allow}$ =	425 psi	
$F'_{c\perp act}$	114 psi	
Ratio	**0.27**	**PASS**
Anchorage Devices		
Chord Hold Downs		
Hold Down	Ratio	
HDU2	**0.85**	**PASS**
Studs Reqd	2	
Anchor Require	5/8 in	
CS14	**0.76**	**PASS**
End length	16 in	
Sill Plate Anchors		
1/2 Bolts at	36" OC	
1/2 Titen HD at	36" OC	
16d	2 per LF	6 in OC
#10 Screws	2 per LF	6 in OC

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 5

Shear Wall Design Calculations

Panel Shear Strength

Nominal Shear Capacity	647 plf	
Allowable Shear Capacity -Vn/Ω	324 plf	
Shear wall base -b (ft)	4.40 ft	
Total area of opening - A_o	0.00 ft²	
Sheathing area ratio - r	1.00	eq (4.3-6)
Adjustment Factor Co	1.00	eq (4.3-5)

Vwall = (Vn/Ω*b)

^{V}wall 1 =	324 plf
^{V}wall 1 =	1424 lb

Total Panel Shear Strength *V*

Vtot =	324 plf

Tension/Compression Chord Strength

T/C force on chords	1885 lb	
T/C force on chords w/ add. Loads	1885 lb	Additional: 0 lbs
Area of Chord Member Used	16.50 in²	
Stress on Chord Member	114 psi	
Load duration factor Cd	1.6	
Compr. Member Size Factor Cf	1	
Tens. Member Size Factor Cf	1	

Member properties

Ft =	350 psi
Fc =	725 psi
F_{cE}/F^*_c=	0.64
Column Stability Fctr Cp	0.53

Tension Member Strength

F't = Ft(Cd)(Cm)(Ct)(Cf) NDS 4.3.1

F't =	560 psi

Compression Member Strength

F'c = Fc(Cd)(Cm)(Ct)(Cf)(Cp) NDS 4.3.1

F'c =	611 psi

Deflection

Bending deflection

$\Delta_b = 8v'^*h^3/(E^*A^*b)$ eq (4.3-1)

Δ_b = 0.02 in

Shear deflection and Nail Slip

$\Delta_{v+n} = v'^*h/(1000^*Ga)$ eq. (4.3-1)

Δ_{v+n} = 0.14 in

Anchorage slip

$\Delta_{anchor} = h^*\Delta_a/b$ eq. (4.3-1)

Δ_{anchor}= 0.07 in

Total Deflection

δsw=	0.24 in

Base plate Compression

Stress on baseplate 114 psi

Sill Compression Strength

F'c⊥ = Fc⊥(Cm)(Ct)(Cb) NDS 4.3.1

F'c⊥=	425 psi

Member properties

$F_{c\perp}$=	425 psi
Wet service Factor Cm	1.0
Temperature Factor Ct	1.0
Bearing Area Factor Cb	1.0

Anchorage Devices

Max uplift on chords

R_{max} = 1885 lb

Hold down required at foundation

Hold down = HDU2

Hold Down Cap. 2215 lb

Hold down required between floors

Hold Down = CS14

Hold Down Cap. 2490 lb

Sole Plate Attachment

Base shear 187 plf

Foundation Anchors

Standard Bolts	Max Spacing	Titen HD	Max Spacing
1/2	36.0 in	1/2	36.0 in
5/8	36.0 in	5/8	36.0 in
3/4	36.0 in		
7/8	36.0 in		
1	36.0 in		

Wood Anchors

16d Nails at	2 per LF
Stress Ratio =	0.57
#10 screws at	2 per LF
Stress Ratio =	0.5

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 8

WOOD SHEAR WALL CALCULATION

Design Philosophy and Code	ASD 7-16 / Wind	
Developed Load on shear wall R	2277 lb	
Factored Load Applied - R	1366 lb	Developed Load * 0.6
Shear Wall Type	Segmented	
Material(s) Used	Wood Sheathing	
Wood Sheathing Type	OSB	
Thickness of Material on first side	7/16 in	Using 15/32 Capacity
Thickness of Material on alt side	N/A	
# of sides sheathed	1	
# of layers on second side	N/A	
Fasteners for first side	8d	
Fasteners for opposite face	N/A	
Fastener Edge Spacing	6	
Fastener Edge Spacing opp Face	N/A	
Stud Spacing	16 in	
Blocked/Unblocked	Blocked	
Stud Species	Spruce Pine Fir	
Wall Thickness	2x6	
Plate to Plate Height - h	10.08 ft	
Shear wall segment length b min = 2.88'	23.00 ft	Aspect Ratio = 0.4 : 1
Number of chord studs	2	
Additional T/C Load on chord studs (lbs)		Note: Optional

Design Summary

Panel Shear Strength		
V_{allow} =	336 plf	7723 lb
V_{act} =	59 plf	1366 lb
Ratio	**0.18**	**PASS**
Tension/Compression Chord Strength		
F'c =	611 psi	
F't =	560 psi	
f_{act} =	36 psi	595 lb
Ratio	**0.06**	**PASS**
Deflection		
δ_{swAct}=	0.07 in	
Ratio	L/3456	
Base plate Compression		
$F'_{c\perp allow}$ =	425 psi	
$F'_{c\perp act}$	36 psi	
Ratio	**0.08**	**PASS**
Anchorage Devices		
Chord Hold Downs		
Hold Down	Ratio	
HDU2	**0.27**	**PASS**
Studs Reqd	2	
Anchor Require	5/8 in	
CS22	**0.7**	**PASS**
End length	7 in	
Sill Plate Anchors		
1/2 Bolts at	36" OC	
1/2 Titen HD at	36" OC	
16d	1 per LF	12 in OC
#10 Screws	1 per LF	12 in OC

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 8

Shear Wall Design Calculations

Panel Shear Strength

Nominal Shear Capacity	672 plf	
Allowable Shear Capacity -Vn/Ω	336 plf	
Shear wall base -b (ft)	23.00 ft	
Total area of opening - A_o	0.00 ft²	
Sheathing area ratio - r	1.00	eq (4.3-6)
Adjustment Factor Co	1.00	eq (4.3-5)

Vwall = (Vn/Ω*b)

$v_{wall\ 1}$ =	336 plf
$V_{wall\ 1}$ =	7723 lb

Total Panel Shear Strength *V*

Vtot =	336 plf

Tension/Compression Chord Strength

T/C force on chords	595 lb	
T/C force on chords w/ add. Loads	595 lb	Additional: 0 lbs
Area of Chord Member Used	16.50 in²	
Stress on Chord Member	36 psi	
Load duration factor Cd	1.6	
Compr. Member Size Factor Cf	1	
Tens. Member Size Factor Cf	1	

Member properties

Ft =	350 psi
Fc =	725 psi
F_{cE}/F^*_c=	0.64
Column Stability Fctr Cp	0.53

Tension Member Strength

F't = Ft(Cd)(Cm)(Ct)(Cf) NDS 4.3.1

F't =	560 psi

Compression Member Strength

F'c = Fc(Cd)(Cm)(Ct)(Cf)(Cp) NDS 4.3.1

F'c =	611 psi

Deflection

Bending deflection

$\Delta_b = 8v'^*h^3/(E^*A^*b)$ eq (4.3-1)

Δ_b = 0.00 in

Shear deflection and Nail Slip

$\Delta_{v+n} = v'^*h/(1000^*Ga)$ eq. (4.3-1)

Δ_{v+n} = 0.05 in

Anchorage slip

$\Delta_{anchor} = h^*\Delta_a/b$ eq. (4.3-1)

Δ_{anchor}= 0.02 in

Total Deflection

δsw=	0.07 in

Base plate Compression

Stress on baseplate 36 psi

Sill Compression Strength

F'c⊥ = Fc⊥(Cm)(Ct)(Cb) NDS 4.3.1

F'c⊥=	425 psi

Member properties

$F_{c\perp}$=	425 psi
Wet service Factor Cm	1.0
Temperature Factor Ct	1.0
Bearing Area Factor Cb	1.0

Anchorage Devices

Max uplift on chords

R_{max} = 595 lb

Hold down required at foundation

Hold down = HDU2

Hold Down Cap. 2215 lb

Hold down required between floors

Hold Down = CS22

Hold Down Cap. 845 lb

Sole Plate Attachment

Base shear 59 plf

Foundation Anchors

Standard Bolts	Max Spacing	Titen HD	Max Spacing
1/2	36.0 in	1/2	36.0 in
5/8	36.0 in	5/8	36.0 in
3/4	36.0 in		
7/8	36.0 in		
1	36.0 in		

Wood Anchors

16d Nails at	1 per LF
Stress Ratio =	0.36
#10 screws at	1 per LF
Stress Ratio =	0.32

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 9

WOOD SHEAR WALL CALCULATION

Design Philosophy and Code	ASD 7-16 / Wind	
Developed Load on shear wall R	1233 lb	
Factored Load Applied - R	740 lb	Developed Load * 0.6
Shear Wall Type	Segmented	
Material(s) Used	Wood Sheathing	
Wood Sheathing Type	OSB	
Thickness of Material on first side	7/16 in	Using 15/32 Capacity
Thickness of Material on alt side	N/A	
# of sides sheathed	1	
# of layers on second side	N/A	
Fasteners for first side	8d	
Fasteners for opposite face	N/A	
Fastener Edge Spacing	6	
Fastener Edge Spacing opp Face	N/A	
Stud Spacing	16 in	
Blocked/Unblocked	Blocked	
Stud Species	Spruce Pine Fir	
Wall Thickness	2x6	
Plate to Plate Height - h	10.08 ft	
Shear wall segment length b min = 2.88'	3.50 ft	Aspect Ratio = 2.9 : 1
Number of chord studs	2	
Additional T/C Load on chord studs (lbs)		Note: Optional

Design Summary		
Panel Shear Strength		
V_{allow} =	299 plf	1046 lb
V_{act} =	211 plf	740 lb
Ratio	**0.71**	**PASS**
Tension/Compression Chord Strength		
F'c =	611 psi	
F't =	560 psi	
f_{act} =	129 psi	2127 lb
Ratio	**0.23**	**PASS**
Deflection		
δ_{swAct}=	0.27 in	
Ratio	L/896	
Base plate Compression		
$F'_{c\perp allow}$ =	425 psi	
$F'_{c\perp act}$	129 psi	
Ratio	**0.3**	**PASS**
Anchorage Devices		
Chord Hold Downs		
Hold Down	Ratio	
HDU2	**0.96**	**PASS**
Studs Reqd	2	
Anchor Require	5/8 in	
CS14	**0.85**	**PASS**
End length	16 in	
Sill Plate Anchors		
1/2 Bolts at	36" OC	
1/2 Titen HD at	36" OC	
16d	2 per LF	6 in OC
#10 Screws	2 per LF	6 in OC

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 9

Shear Wall Design Calculations

Panel Shear Strength

Nominal Shear Capacity	598 plf	
Allowable Shear Capacity -Vn/Ω	299 plf	
Shear wall base -b (ft)	3.50 ft	
Total area of opening - A_o	0.00 ft²	
Sheathing area ratio - r	1.00	eq (4.3-6)
Adjustment Factor Co	1.00	eq (4.3-5)

Vwall = (Vn/Ω*b)

$v_{wall\ 1}$ =	299 plf
$V_{wall\ 1}$ =	1046 lb

Total Panel Shear Strength *V*

Vtot =	299 plf

Tension/Compression Chord Strength

T/C force on chords	2127 lb	
T/C force on chords w/ add. Loads	2127 lb	Additional: 0 lbs
Area of Chord Member Used	16.50 in²	
Stress on Chord Member	129 psi	
Load duration factor Cd	1.6	
Compr. Member Size Factor Cf	1	
Tens. Member Size Factor Cf	1	

Member properties

Ft =	350 psi
Fc =	725 psi
F_{cE}/F^*_c=	0.64
Column Stability Fctr Cp	0.53

Tension Member Strength

F't = Ft(Cd)(Cm)(Ct)(Cf) NDS 4.3.1

F't =	560 psi

Compression Member Strength

F'c = Fc(Cd)(Cm)(Ct)(Cf)(Cp) NDS 4.3.1

F'c =	611 psi

Deflection

Bending deflection

$\Delta_b = 8v'^*h^3/(E^*A^*b)$ eq (4.3-1)

Δ_b = 0.02 in

Shear deflection and Nail Slip

$\Delta_{v+n} = v'^*h/(1000^*Ga)$ eq. (4.3-1)

Δ_{v+n} = 0.16 in

Anchorage slip

$\Delta_{anchor} = h^*\Delta_a/b$ eq. (4.3-1)

Δ_{anchor}= 0.08 in

Total Deflection

δsw=	0.27 in

Base plate Compression

Stress on baseplate 129 psi

Sill Compression Strength

F'c⊥ = Fc⊥(Cm)(Ct)(Cb) NDS 4.3.1

F'c⊥=	425 psi

Member properties

$F_{c\perp}$=	425 psi
Wet service Factor Cm	1.0
Temperature Factor Ct	1.0
Bearing Area Factor Cb	1.0

Anchorage Devices

Max uplift on chords

R_{max} = 2127 lb

Hold down required at foundation

Hold down = HDU2

Hold Down Cap. 2215 lb

Hold down required between floors

Hold Down = CS14

Hold Down Cap. 2490 lb

Sole Plate Attachment

Base shear 211 plf

Foundation Anchors

Standard Bolts	Max Spacing	Titen HD	Max Spacing
1/2	36.0 in	1/2	36.0 in
5/8	36.0 in	5/8	36.0 in
3/4	36.0 in		
7/8	36.0 in		
1	36.0 in		

Wood Anchors

16d Nails at	2 per LF
Stress Ratio =	0.64
#10 screws at	2 per LF
Stress Ratio =	0.56

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 10

WOOD SHEAR WALL CALCULATION

Design Philosophy and Code	ASD 7-16 / Wind	
Developed Load on shear wall R	1196 lb	
Factored Load Applied - R	718 lb	Developed Load * 0.6
Shear Wall Type	Segmented	
Material(s) Used	Wood Sheathing	
Wood Sheathing Type	OSB	
Thickness of Material on first side	7/16 in	Using 15/32 Capacity
Thickness of Material on alt side	N/A	
# of sides sheathed	1	
# of layers on second side	N/A	
Fasteners for first side	8d	
Fasteners for opposite face	N/A	
Fastener Edge Spacing	6	
Fastener Edge Spacing opp Face	N/A	
Stud Spacing	16 in	
Blocked/Unblocked	Blocked	
Stud Species	Spruce Pine Fir	
Wall Thickness	2x6	
Plate to Plate Height - h	10.08 ft	
Shear wall segment length b min = 2.88'	3.33 ft	Aspect Ratio = 3 : 1
Number of chord studs	2	
Additional T/C Load on chord studs (lbs)		Note: Optional

Design Summary		
Panel Shear Strength		
V_{allow} =	293 plf	975 lb
V_{act} =	215 plf	718 lb
Ratio	**0.73**	**PASS**
Tension/Compression Chord Strength		
F'c =	611 psi	
F't =	560 psi	
f_{act} =	131 psi	2167 lb
Ratio	**0.23**	**PASS**
Deflection		
δ_{swAct}=	0.28 in	
Ratio	L/864	
Base plate Compression		
$F'_{c\perp allow}$ =	425 psi	
$F'_{c\perp act}$	131 psi	
Ratio	**0.31**	**PASS**
Anchorage Devices		
Chord Hold Downs		
Hold Down	Ratio	
HDU2	**0.98**	**PASS**
Studs Reqd	2	
Anchor Require	5/8 in	
CS14	**0.87**	**PASS**
End length	16 in	
Sill Plate Anchors		
1/2 Bolts at	36" OC	
1/2 Titen HD at	36" OC	
16d	2 per LF	6 in OC
#10 Screws	2 per LF	6 in OC

Project:	40390 - Summit View Farms
Client:	Elswood Smith Carlson Architects
Engineer:	GP & AC
Date:	1/18/2021
Description:	Shear wall 10

Shear Wall Design Calculations

Panel Shear Strength

Nominal Shear Capacity	585 plf	
Allowable Shear Capacity -Vn/Ω	293 plf	
Shear wall base -b (ft)	3.33 ft	
Total area of opening - A_o	0.00 ft²	
Sheathing area ratio - r	1.00	eq (4.3-6)
Adjustment Factor Co	1.00	eq (4.3-5)

Vwall = (Vn/Ω*b)

$v_{wall\ 1}$ =	293 plf
$V_{wall\ 1}$ =	975 lb

Total Panel Shear Strength *V*

Vtot =	293 plf

Tension/Compression Chord Strength

T/C force on chords	2167 lb	
T/C force on chords w/ add. Loads	2167 lb	Additional: 0 lbs
Area of Chord Member Used	16.50 in²	
Stress on Chord Member	131 psi	
Load duration factor Cd	1.6	
Compr. Member Size Factor Cf	1	
Tens. Member Size Factor Cf	1	

Member properties

Ft =	350 psi
Fc =	725 psi
F_{cE}/F^*_c=	0.64
Column Stability Fctr Cp	0.53

Tension Member Strength

F't = Ft(Cd)(Cm)(Ct)(Cf) NDS 4.3.1

F't =	560 psi

Compression Member Strength

F'c = Fc(Cd)(Cm)(Ct)(Cf)(Cp) NDS 4.3.1

F'c =	611 psi

Deflection

Bending deflection

$\Delta_b = 8v'^*h^3/(E^*A^*b)$ eq (4.3-1)

Δ_b = 0.03 in

Shear deflection and Nail Slip

$\Delta_{v+n} = v'^*h/(1000^*Ga)$ eq. (4.3-1)

Δ_{v+n} = 0.17 in

Anchorage slip

$\Delta_{anchor} = h^*\Delta_a/b$ eq. (4.3-1)

Δ_{anchor}= 0.09 in

Total Deflection

δsw=	0.28 in

Base plate Compression

Stress on baseplate 131 psi

Sill Compression Strength

F'c⊥ = Fc⊥(Cm)(Ct)(Cb) NDS 4.3.1

F'c⊥=	425 psi

Member properties

$F_{c\perp}$=	425 psi
Wet service Factor Cm	1.0
Temperature Factor Ct	1.0
Bearing Area Factor Cb	1.0

Anchorage Devices

Max uplift on chords

R_{max} = 2167 lb

Hold down required at foundation

Hold down = HDU2

Hold Down Cap. 2215 lb

Hold down required between floors

Hold Down = CS14

Hold Down Cap. 2490 lb

Sole Plate Attachment

Base shear 215 plf

Foundation Anchors

Standard Bolts	Max Spacing	Titen HD	Max Spacing
1/2	36.0 in	1/2	36.0 in
5/8	36.0 in	5/8	36.0 in
3/4	36.0 in		
7/8	36.0 in		
1	36.0 in		

Wood Anchors

16d Nails at	2 per LF
Stress Ratio =	0.65
#10 screws at	2 per LF
Stress Ratio =	0.57