STRUCTURAL DESIGN CALCULATIONS

for

Olson Architectural Group
1916 NW 79^{th} Terrace
Kansas City, MO 64151

SWIG – with Savory Management

August 10, 2023

K. Andrew Gilmore, PE
AGilmore Services, LLC
Professional Structural Engineer

Gravity Load Information

Engineer: KAG
Date: July 19, 2023
Project: SWIG

Live Loads

Roof Live Load (RLL):	20	psf

Dead Loads

Roof Dead Load (RDL):

Roofing:	1	psf
Rigid Insulation:	3	psf
Roof Sheathing 3/4" plywood:	2	psf
Roof Joist @ 16” O.C.:	3	psf
Underhung - 1 layers 1/2" gyp:	2.5	psf
Batt Insulation:	1.5	psf
Total:	13	psf

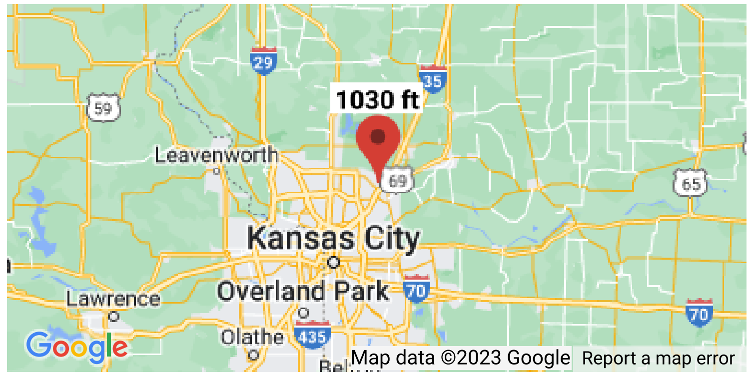
Wall Dead Load (WDL):

1/2" Sheathing:	2	psf
Wall studs - 2x4 @ 16” O.C.:	1.5	psf
1/2" Gyp:	2.5	psf
Batt Insulation:	1.0	psf
Misc:	1.0	psf
Total:	8	psf

ATC Hazards by Location

Search Information

Address:	9705 N Ash Ave, Kansas City, MO 64157, USA
Coordinates:	39.2686359, -94.45099239999999
Elevation:	1030 ft
Timestamp:	2023-07-24T16:43:37.592Z
Hazard Type:	Snow



ASCE 7-16

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.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Disclaimer

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

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Version: 3.1

Designed on: July 24, 2023

Beams/Joists Analysis and Design

wwpa.org

ASD Method

Developed by: **Forum Engineers**

How to Enter Data

Print

Order Pro Version

Setup | Beam or Girder | Joist or Rafter

Member at	Floor	Roof	
Repetitive Use ?	No	Yes	
Incised for PT ?	No	Yes	
Flat Use :	No	Yes	
Moisture Content :	<19%	>19%	
Temperature (° F) :	<100	100~125	125~150

Set Duration Factors | with Cantilever

Set Deflection Limits | with Point Load(s)

Reset Loads to Zero | with Sloped Load(s)

(pressed-down buttons are selected)

Member # Header at Elevated Window

Location : Elevated Wall

Nominal Size : (2) 2 x 6

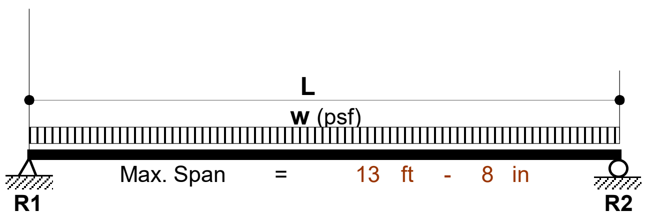
Species = Douglas Fir-Larch

** Dimension Lumber ** **Grade =** No.2

Span (L) = 16 ft - 0 in

Tributary Width (B) = 1 ft - 0 in

Unsupported Length (lu) = 1 ft - 0 in



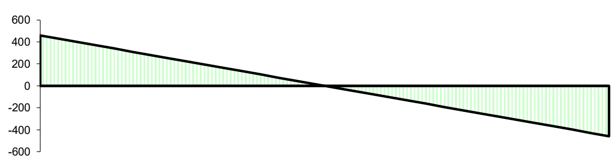
R1 458 lb | R2 458 lb

Stress and/or Deflection Check → **NG**

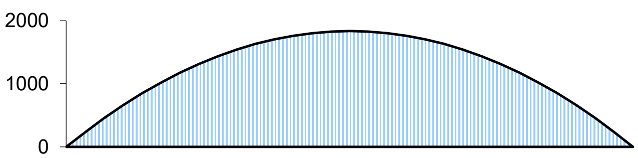
	Actual		Allowable		Ratio
Max fv (psi) & V (lb)	39	432	288	3168	14%
Max fb (psi) & M (lb-ft)	1455	1834	1868	2355	78%
Total Load Max. Defl. (in)	-1.27	L/151	L/180	1.07	**119%**
Live Load Max. Defl. (in)	-1.27	L/151	L/240	0.80	**159%**

LOADING

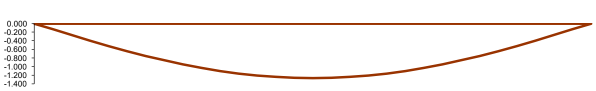
	Load Type		
Dead Load	*Uniform*	w (psf) =	0
Special LL (CD = 1.60)	*Uniform*	w (psf) =	57



Shear Force, V (lb)



Bending Moment, M (lb-ft)



Total Load Deflection (in)

Adjustment Factors

		for Fb	for Fv	for E
Wet Service	C_M =	1.00	1.00	1.00
Temperature	C_t =	1.00	1.00	1.00
Beam Stability	C_L =	1.00	N/A	N/A
Size	C_F =	1.30	N/A	N/A
Flat Use	C_{fu} =	1.00	N/A	N/A
Incising	C_i =	1.00	1.00	1.00
Repetitive Member	C_r =	1.00	N/A	N/A
Buckling Stiffness	C_T =	N/A	N/A	1.06

(C_T for Emin only)

Design Values in psi

	Fb	Fv	E	Emin
Reference	900	180	1600000	580000
Adjusted	1868	288	1600000	580000

Section Properties

breadth (b) =	3	in
depth (d) =	5.5	in
Area (A) =	16.5	in^2
Section Modulus (Sx) =	15.1	in^3
Moment of Inertial (Ix) =	41.6	in^4

Version: 3.1

Designed on: July 24, 2023

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Incised for PT ?	No	Yes	
Flat Use :	No	Yes	
Moisture Content :	<19%	>19%	
Temperature (° F) :	<100	100~125	125~150

Set Duration Factors | with Cantilever

Set Deflection Limits | with Point Load(s)

Reset Loads to Zero | with Sloped Load(s)

(pressed-down buttons are selected)

Member # Header at Elevated Window

Location : Elevated Wall

Nominal Size : (2) 2 x 10

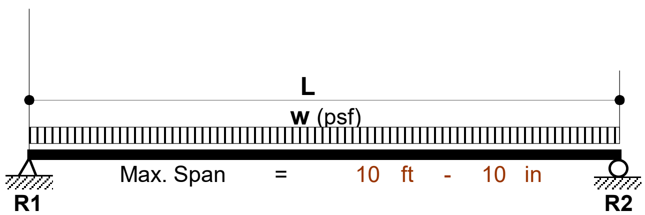
Species = Douglas Fir-Larch

** Dimension Lumber ** **Grade =** No.2

Span (L) = 16 ft - 0 in

Tributary Width (B) = 1 ft - 0 in

Unsupported Length (lu) = 1 ft - 0 in



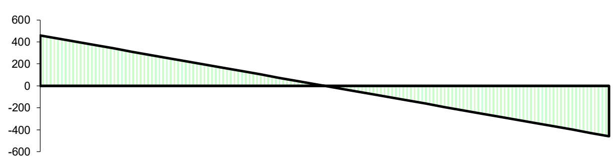
458 lb | 458 lb

Stress and/or Deflection Check → **NG**

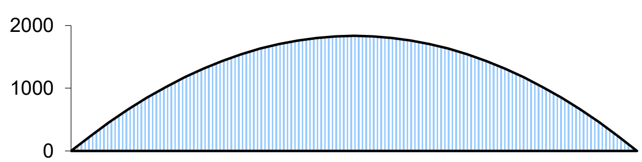
	Actual		Allowable		Ratio
Max fv (psi) & V (lb)	24	444	288	5328	8%
Max fb (psi) & M (lb-ft)	1586	1834	1895	2191	84%
Total Load Max. Defl. (in)	-2.54	L/76	L/180	1.07	238%
Live Load Max. Defl. (in)	-2.54	L/76	L/240	0.80	317%

LOADING **Load Type**

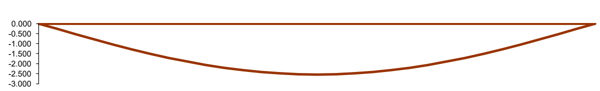
Dead Load	*Uniform* w (psf) =	0
Special LL	*Uniform* w (psf) =	57
CD = 1.60		



Shear Force, V (lb)



Bending Moment, M (lb-ft)



Total Load Deflection (in)

Adjustment Factors

		for Fb	for Fv	for E
Wet Service	C_M =	1.00	1.00	1.00
Temperature	C_t =	1.00	1.00	1.00
Beam Stability	C_L =	1.00	N/A	N/A
Size	C_F =	1.10	N/A	N/A
Flat Use	C_{fu} =	1.20	N/A	N/A
Incising	C_i =	1.00	1.00	1.00
Repetitive Member	C_r =	1.00	N/A	N/A
Buckling Stiffness	C_T =	N/A	N/A	1.06

(C_T for Emin only)

Design Values in psi

	Fb	Fv	E	Emin
Reference	900	180	1600000	580000
Adjusted	1895	288	1600000	580000

Section Properties

breadth (b) =	3	in
depth (d) =	9.25	in
Area (A) =	27.8	in^2
Section Modulus (Sx) =	13.9	in^3
Moment of Inertial (Ix) =	20.8	in^4

SNOW LOADING ANALYSIS

Job Name:	SWIG	Subject:	Snow Drifts
Job No:		Originator:	KAG
		Checker:	

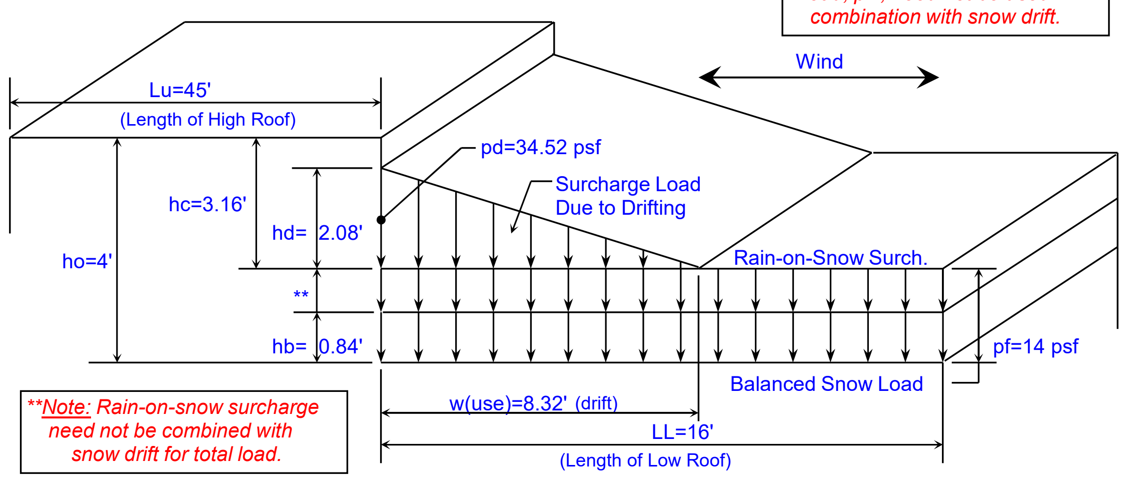
Input Data:

Building Risk Category =	II		Table 1.5-1, page 2
Ground Snow Load, p_g =	20.00	psf	Figure 7-1, pages 34-35 and Table 7-1, page 30
Length of High Roof, L_u =	45.00	ft.	Length of Roof Upwind of the Snow Drift
Length of Low Roof, L_L =	16.00	ft.	Length of Roof Downwind of the Snow Drift
. Dist. from Eave to Ridge, W =	10.00	ft.	Horizontal Distance from Eave to Ridge
Type of Roof =	Monoslope		Type of Roof = Monoslope, Gable, or Hip
Obstruction Height, h_o =	4.00	ft.	High Roof - Low Roof Elevations
Roof Slope, S =	0.25	in./ft.	S = Rise per foot of Run
Exposure Factor, C_e =	1.00		Table 7-2, page 30
Thermal Factor, C_t =	1.00		Table 7-3, page 30

Results:

Roof Angle, θ =	1.1935	deg.	θ = ATAN(S/12)
Importance Factor, I_s =	1.00		Table 1.5-2, page 5
Snow Density, γ =	16.60	pcf	γ = 0.13*p_g+14 <= 30 (Eqn. 7.7-1, page 33)
Flat Roof Snow Load, p_f =	14.00	psf	p_f = 0.7*C_e*C_t*I_s*p_g (Eqn. 7.3-1, page 29)
*Min. Roof Snow Load, p_m =	20.00	psf	p_m = p_g*I_s for p_g <= 20 , p_m = 20*I_s for p_g > 20
Balanced Snow Load Ht., h_b =	0.84	ft.	h_b = $p_{f(use)}/\gamma$ (Section 7.1, page 29)
Clear Height, h_c =	3.16	ft.	h_c = h_o-h_b >= 0 (Section 7.1, page 29)
Leeward Drift Height, h_{dL} =	2.08	ft.	h_{dL} = 0.43*L_u^1/3*(p_g+10)^1/4-1.5, with L_u>=25' (Figure 7-9)
Windward Drift Height, h_{dw} =	1.08	ft.	h_{dw} = 0.75*(0.43*L_L^1/3*(p_g+10)^1/4-1.5), with L_L>=25'
Max. Drift Height, $h_{d(max)}$ =	2.08	ft.	$h_{d(max)}$ = maximum of: (h_{dL} or h_{dw})
Ratio, h_c/h_b =	3.74		If hc/hb >= 0.2 , then snow drifts are required to be applied
Drift Length, w =	8.32	ft.	If $h_{d(max)}$<=h_c: w = 4*$h_{d(max)}$, if $h_{d(max)}$>h_c: w = 4*$h_{d(max)}$^2/h_c
Design Drift Height, h_d =	2.08	ft.	If $h_{d(max)}$ <= h_c: h_d = $h_{d(max)}$, if $h_{d(max)}$ > h_c: h_d = h_c
Drift Length, $w_{(max)}$ =	25.25	ft.	$w_{(max)}$ <= 8*h_c
Drift Length, $w_{(use)}$ =	8.32	ft.	$w_{(use)}$ = minimum of: w or $w_{(max)}$
Wt. of Drift at High End, p_d =	34.52	psf	p_d =h_d*γ (maximum value)
Wt. of Drift at Low End, p_{de} =	0.00	psf	p_{de} = 0, as Low Roof Length (LL) >= $w_{(max)}$
Rain-on-Snow Surch., p_{rs} =	0.00	psf	p_{rs} = 5.0 psf when 0 < p_g <=20 and θ < W/50) (Sect. 7.10)
Balanced Snow Load, $p_{f(bal)}$ =	14.00	psf	$p_{f(bal)}$ = p_f+p_{rs}
**Total Snow Load, $p_{(total)}$ =	48.52	psf	$p_{(total)}$ = $p_{f(bal)}$+p_d

**Note: Minimum flat roof snow load, pm, need not be used in combination with snow drift.*



Configuration of Snow Drift on Lower Roof

RECTANGULAR SPREAD FOOTING ANALYSIS

For Assumed Rigid Footing with from 1 To 8 Piers (Load Points)

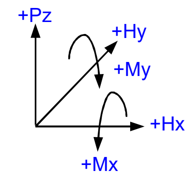
Subjected to Uniaxial or Biaxial Eccentricity

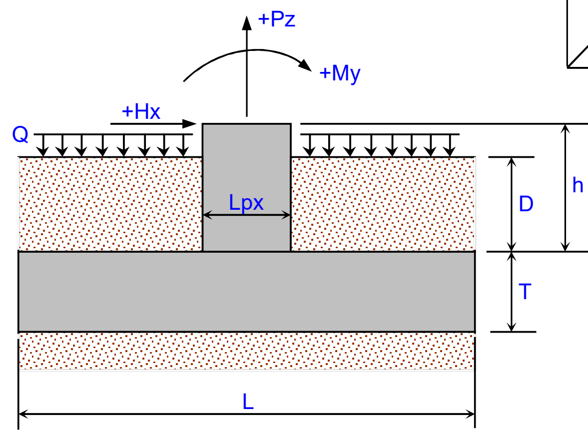
Job Name:	SWIG	Subject:	Shear Wall Footing		
Job Number:		Originator:	KAG	Checker:	

Input Data:

Footing Data:

Footing Length, L =	9.833	ft.
Footing Width, B =	2.000	ft.
Footing Thickness, T =	1.500	ft.
Concrete Unit Wt., γc =	0.150	kcf
Soil Depth, D =	0.000	ft.
Soil Unit Wt., γs =	0.120	kcf
Pass. Press. Coef., Kp =	3.000	
Coef. of Base Friction, μ =	0.400	
Uniform Surcharge, Q =	0.000	ksf



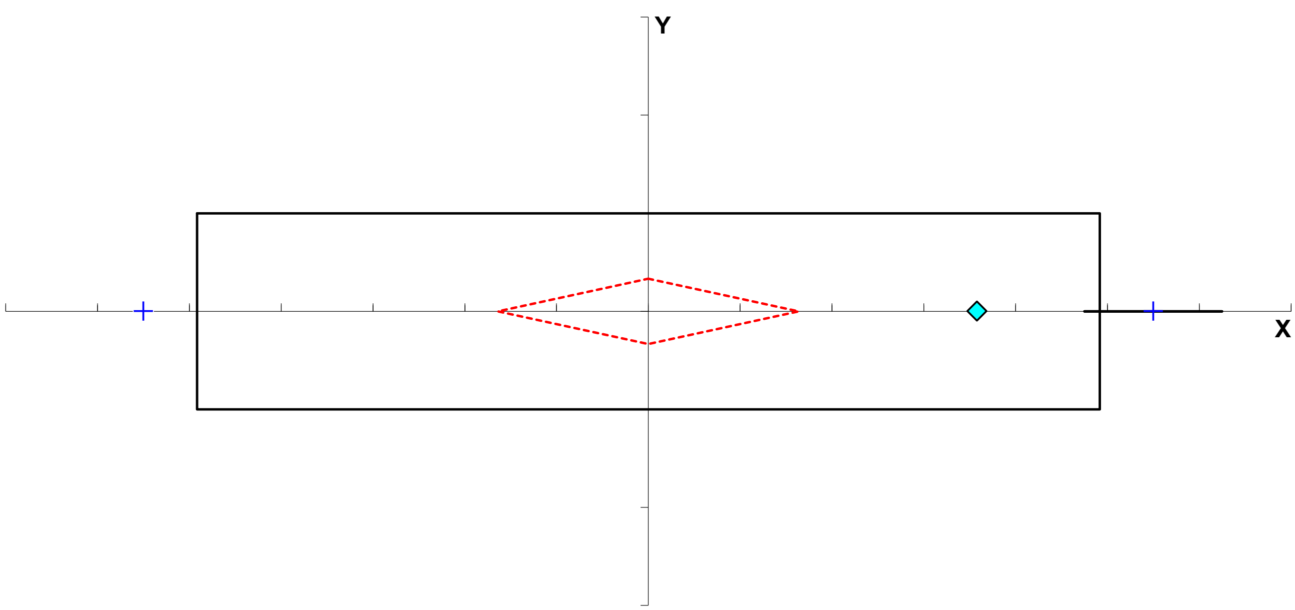


Nomenclature

Pier/Loading Data:

Number of Piers = 2

	Pier #1	Pier #2						
Xp (ft.) =	-5.500	5.500						
Yp (ft.) =	0.000	0.000						
Lpx (ft.) =	0.000	1.500						
Lpy (ft.) =	0.000	0.000						
h (ft.) =	2.500	2.500						
Pz (k) =	-2.13	2.13						
Hx (k) =	0.67	9.17						
Hy (k) =	0.00	0.00						
Mx (ft-k) =	0.00	0.00						
My (ft-k) =	0.00	0.00						



FOOTING PLAN

(continued)

Results:

Total Resultant Load and Eccentricities:

ΣPz =	-4.42	kips
ex =	3.58	ft. (> L/6)
ey =	0.00	

Overturning Check:

ΣMrx =	N.A.	ft-kips
ΣMox =	N.A.	ft-kips
FS(ot)x =	N.A.	
ΣMry =	43.98	ft-kips
ΣMoy =	17.11	ft-kips
FS(ot)y =	2.571	>= 1.5

Sliding Check:

Pass(x) =	0.81	kips	
Frict(x) =	1.77	kips	
FS(slid)x =	0.262	<1.0	ERROR!
Pass(y) =	3.98	kips	
Frict(y) =	1.77	kips	
FS(slid)y =	N.A.		

Uplift Check:

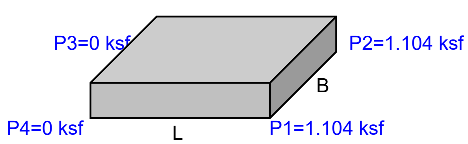
ΣPz(down) =	-6.56	kips
ΣPz(uplift) =	2.13	kips
FS(uplift) =	3.074	>= 1.5

Bearing Length and % Bearing Area:

Dist. x =	N.A.	ft.
Dist. y =	N.A.	ft.
Brg. Lx =	4.010	ft.
Brg. Ly =	2.000	ft.
%Brg. Area =	40.78	%
Biaxial Case =	N.A.	

Gross Soil Bearing Corner Pressures:

P_1 =	1.104	ksf
P_2 =	1.104	ksf
P_3 =	0.000	ksf
P_4 =	0.000	ksf



CORNER PRESSURES

Maximum Net Soil Pressure:

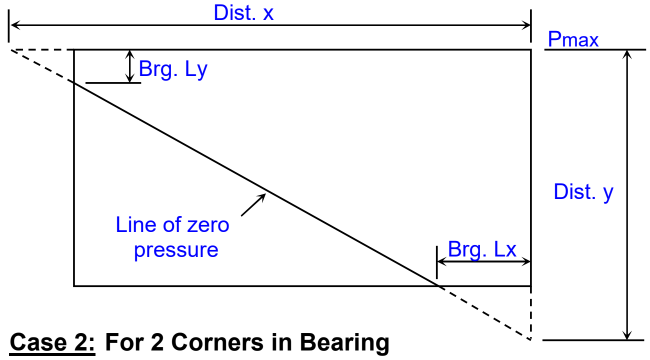
P_{max}(net) = P_{max}(gross)-(D+T)*γ_s

P_{max}(net) =	0.924	ksf

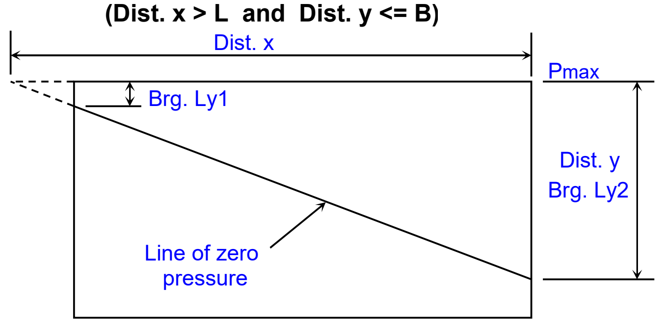
Comments:

Nomenclature for Biaxial Eccentricity:

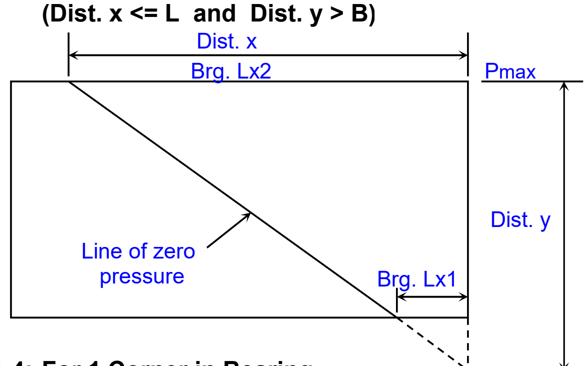
Case 1: For 3 Corners in Bearing
(Dist. x > L and Dist. y > B)



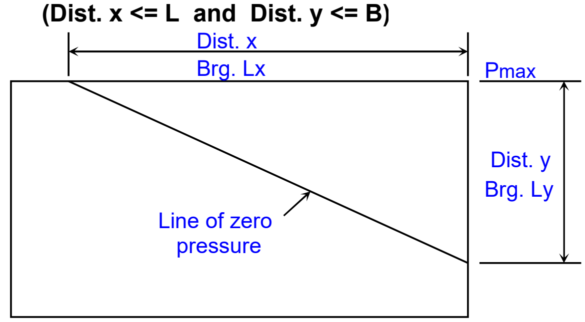
Case 2: For 2 Corners in Bearing
(Dist. x > L and Dist. y <= B)



Case 3: For 2 Corners in Bearing
(Dist. x <= L and Dist. y > B)



Case 4: For 1 Corner in Bearing
(Dist. x <= L and Dist. y <= B)

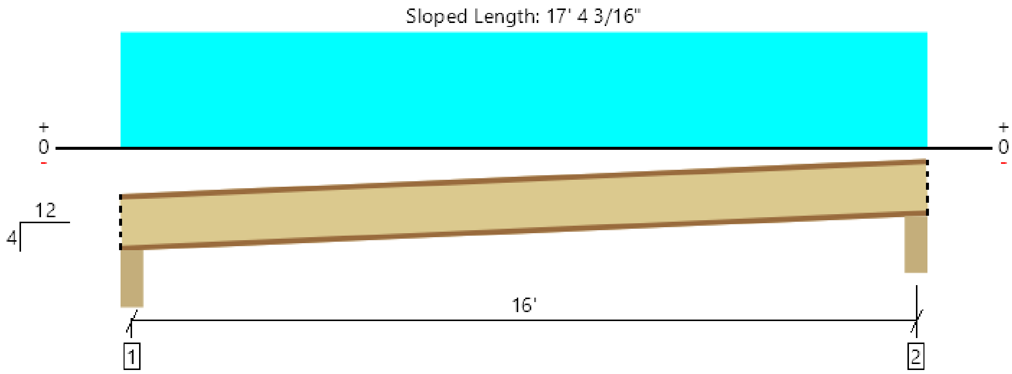


FORTEWEB

MEMBER REPORT

PASSED

Roof, Roof: Joist Full Drift
1 piece(s) 11 7/8" TJI® 210 @ 24" OC



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

Member Length : 17' 8 1/8"

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	785 @ 4 1/2"	1679 (3.50")	Passed (47%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	741 @ 5 1/2"	1903	Passed (39%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	2943 @ 8' 2 3/4"	4364	Passed (67%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.366 @ 8' 2 3/4"	0.828	Passed (L/542)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.514 @ 8' 2 3/4"	1.104	Passed (L/387)	--	1.0 D + 1.0 S (All Spans)

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

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 4/12

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Beveled Plate - DF	5.50"	5.50"	1.75"	226	329	560	785	Blocking
2 - Beveled Plate - DF	5.50"	5.50"	1.75"	226	329	560	785	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' 3" o/c	
Bottom Edge (Lu)	17' 4" o/c	

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

Vertical Load	Location	Spacing	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 16' 5 1/2"	24"	13.0	20.0	34.0	Default Load

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

ForteWEB Software Operator	Job Notes
Andy Gilmore AGilmore Services, LLC (913) 660-3778 andy.gilmore22@gmail.com	

Weyerhaeuser

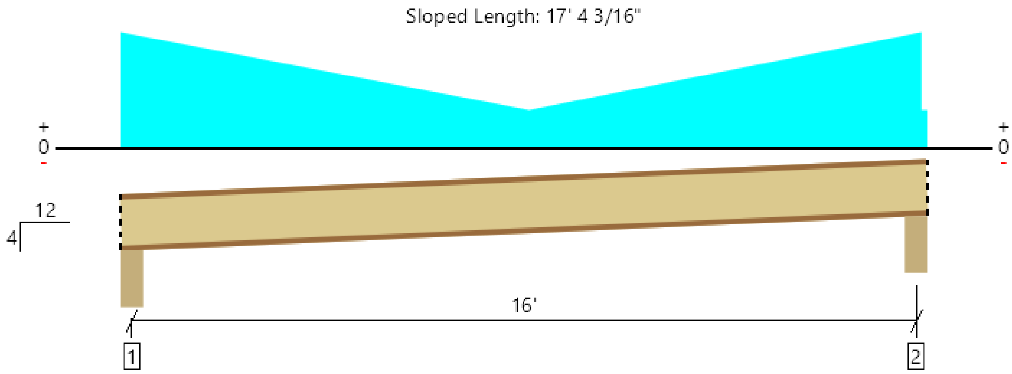
FORTEWEB

MEMBER REPORT

PASSED

Roof, Roof: Joist w/ Drift

1 piece(s) 11 7/8" TJI® 210 @ 24" OC



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Member Length : 17' 8 1/8"

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 4/12

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	511 @ 4 1/2"	1679 (3.50")	Passed (30%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	524 @ 5 1/2"	2069	Passed (25%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	2079 @ 8' 2 3/4"	4744	Passed (44%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.215 @ 8' 2 3/4"	0.828	Passed (L/922)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.363 @ 8' 2 3/4"	1.104	Passed (L/547)	--	1.0 D + 1.0 Lr (All Spans)

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Beveled Plate - DF	5.50"	5.50"	1.75"	226	329	286	555	Blocking
2 - Beveled Plate - DF	5.50"	5.50"	1.75"	226	329	278	555	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 1" o/c	
Bottom Edge (Lu)	17' 4" o/c	

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

Vertical Loads	Location	Spacing	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 16' 5 1/2"	24"	13.0	20.0	-	Default Load
2 - Tapered (PLF)	0 to 8' 4"	N/A	-	-	69.0 to 0.0	
3 - Tapered (PLF)	8' 4" to 16' 4"	N/A	-	-	0.0 to 69.0	

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

ForteWEB Software Operator	Job Notes
Andy Gilmore AGilmore Services, LLC (913) 660-3778 andy.gilmore22@gmail.com	

Weyerhaeuser

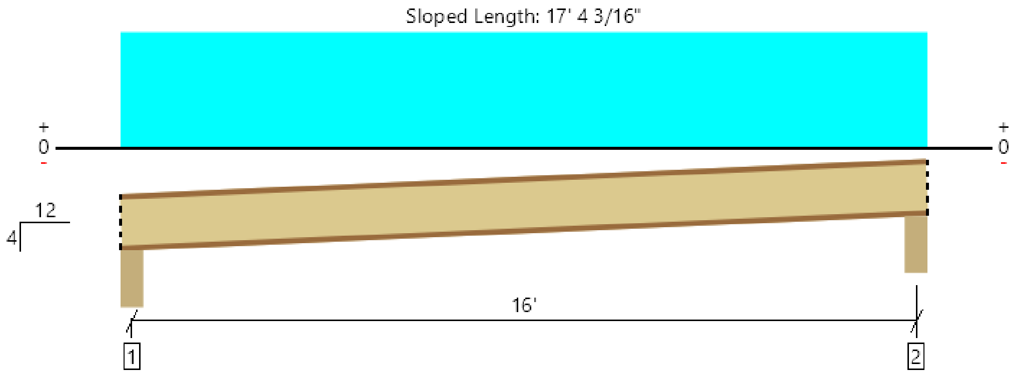
FORTEWEB

MEMBER REPORT

PASSED

Roof, Roof: Joist

1 piece(s) 11 7/8" TJI® 110 @ 24" OC



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

Member Length : 17' 8 1/8"

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	555 @ 4 1/2"	1719 (3.50")	Passed (32%)	1.25	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	524 @ 5 1/2"	1950	Passed (27%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	2079 @ 8' 2 3/4"	3950	Passed (53%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.250 @ 8' 2 3/4"	0.828	Passed (L/794)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.422 @ 8' 2 3/4"	1.104	Passed (L/471)	--	1.0 D + 1.0 Lr (All Spans)

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

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 4/12

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Roof Live	Factored	
1 - Beveled Plate - DF	5.50"	5.50"	1.75"	226	329	555	Blocking
2 - Beveled Plate - DF	5.50"	5.50"	1.75"	226	329	555	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 11" o/c	
Bottom Edge (Lu)	17' 4" o/c	

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

Vertical Load	Location	Spacing	Dead (0.90)	Roof Live (non-snow: 1.25)	Comments
1 - Uniform (PSF)	0 to 16' 5 1/2"	24"	13.0	20.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Andy Gilmore AGilmore Services, LLC (913) 660-3778 andy.gilmore22@gmail.com	

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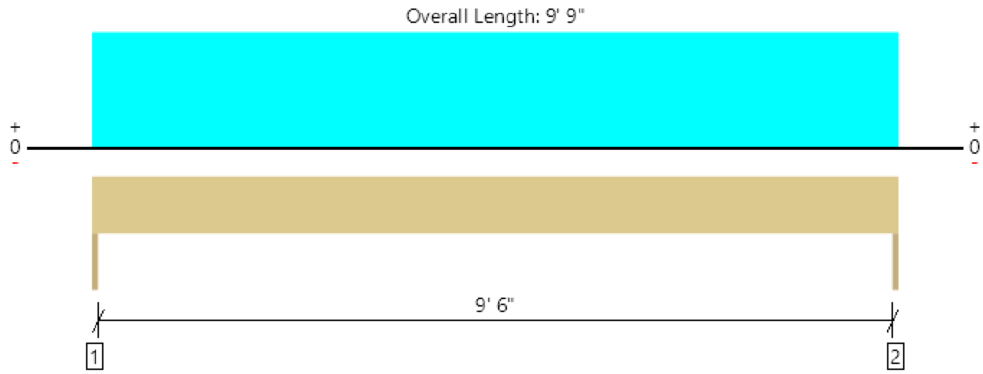
FORTEWEB

MEMBER REPORT

PASSED

Headers, Wall: Header 9.5'

2 piece(s) 1 3/4" x 11 1/4" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3215 @ 0	3938 (1.50")	Passed (82%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	2514 @ 1' 3/4"	8603	Passed (29%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	7837 @ 4' 10 1/2"	18558	Passed (42%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.154 @ 4' 10 1/2"	0.325	Passed (L/758)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.184 @ 4' 10 1/2"	0.488	Passed (L/635)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Trimmer - DF	1.50"	1.50"	1.50"	524	780	2691	3215	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	524	780	2691	3215	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	9' 9" o/c	
Bottom Edge (Lu)	9' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 9' 9"	N/A	11.5	--	--	
1 - Uniform (PSF)	0 to 9' 9"	8'	12.0	20.0	69.0	Default Load

Weyerhaeuser Notes

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

ForteWEB Software Operator	Job Notes
Andy Gilmore AGilmore Services, LLC (913) 660-3778 andy.gilmore22@gmail.com	

Weyerhaeuser

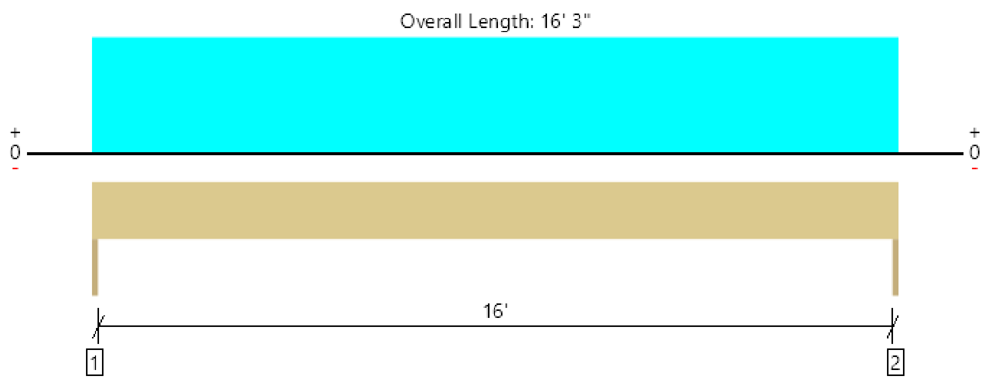
FORTEWEB

MEMBER REPORT

PASSED

Roof, Wall: Header (Vert Load)

2 piece(s) 2 x 10 DF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	366 @ 0	2813 (1.50")	Passed (13%)	--	1.0 D (All Spans)
Shear (lbs)	326 @ 10 3/4"	2997	Passed (11%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	1486 @ 8' 1 1/2"	3177	Passed (47%)	0.90	1.0 D (All Spans)
Live Load Defl. (in)	0.000 @ 0	0.542	Passed (2L/999+)	--	1.0 D (All Spans)
Total Load Defl. (in)	0.223 @ 8' 1 1/2"	0.813	Passed (L/874)	--	1.0 D (All Spans)

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

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)		Accessories
	Total	Available	Required	Dead	Factored	
1 - Trimmer - DF	1.50"	1.50"	1.50"	366	366	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	366	366	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	16' 3" o/c	
Bottom Edge (Lu)	16' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Comments
0 - Self Weight (PLF)	0 to 16' 3"	N/A	7.0	
1 - Uniform (PSF)	0 to 16' 3"	1'	38.0	Default Load

Member Notes
Vert Load

Weyerhaeuser Notes

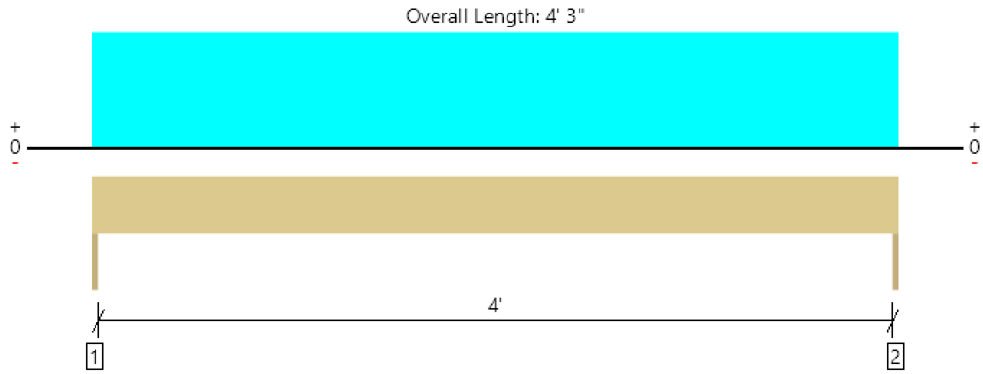
ForteWEB Software Operator	Job Notes
Andy Gilmore AGilmore Services, LLC (913) 660-3778 andy.gilmore22@gmail.com	

FORTEWEB

MEMBER REPORT

PASSED

Headers, Wall: Header
2 piece(s) 2 x 8 DF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1389 @ 0	2813 (1.50")	Passed (49%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	912 @ 8 3/4"	3002	Passed (30%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1476 @ 2' 1 1/2"	2720	Passed (54%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.027 @ 2' 1 1/2"	0.142	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.031 @ 2' 1 1/2"	0.213	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Trimmer - DF	1.50"	1.50"	1.50"	216	340	1173	1389	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	216	340	1173	1389	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' 3" o/c	
Bottom Edge (Lu)	4' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 4' 3"	N/A	5.5	--	--	
1 - Uniform (PSF)	0 to 4' 3"	8'	12.0	20.0	69.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Andy Gilmore AGilmore Services, LLC (913) 660-3778 andy.gilmore22@gmail.com	

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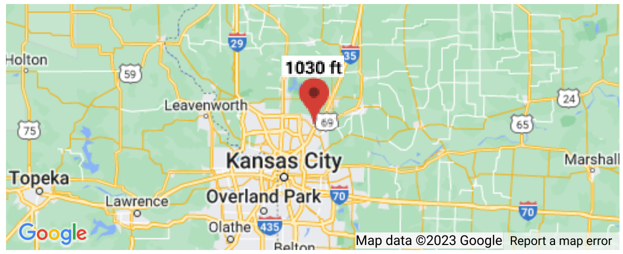
This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

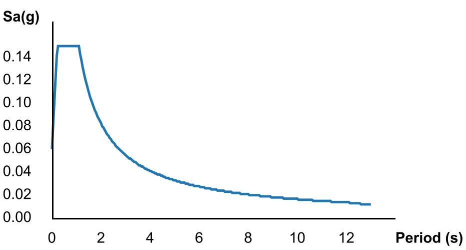
ATC Hazards by Location

Search Information

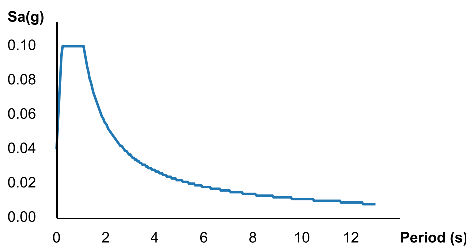
Address:	9705 N Ash Ave, Kansas City, MO 64157, USA
Coordinates:	39.2686359, -94.45099239999999
Elevation:	1030 ft
Timestamp:	2023-07-24T16:45:18.853Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D-default



MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.094	MCE_R ground motion (period=0.2s)
S_1	0.069	MCE_R ground motion (period=1.0s)
S_{MS}	0.15	Site-modified spectral acceleration value
S_{M1}	0.165	Site-modified spectral acceleration value
S_{DS}	0.1	Numeric seismic design value at 0.2s SA
S_{D1}	0.11	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	0.931	Coefficient of risk (0.2s)
CR_1	0.877	Coefficient of risk (1.0s)
PGA	0.044	MCE_G peak ground acceleration
F_{PGA}	1.6	Site amplification factor at PGA
PGA_M	0.071	Site modified peak ground acceleration
T_L	12	Long-period transition period (s)
SsRT	0.094	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.101	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.069	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.

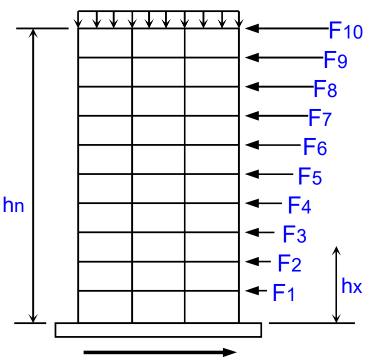
Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

SEISMIC BASE SHEAR AND VERTICAL SHEAR DISTRIBUTION

Job Name:	SWIG	Subject:	Main Seismic Force
Job Number:		Originator:	KAG	Checker:

Input Data:

Risk Category =	II	IBC 2012, Table 1604.5, page 336
Importance Factor, I =	1.00	ASCE 7-10 Table 1.5-2, page 5
Soil Site Class =	D	ASCE 7-10 Table 20.3-1, page 204
Location Zip Code =	64157	
Spectral Accel., S_S =	0.094	ASCE 7-10 Figures 22-1 to 22-11
Spectral Accel., S_1 =	0.069	ASCE 7-10 Figures 22-1 to 22-11
Long. Trans. Period, T_L =	12.000	sec. ASCE 7 Fig's. 22-12 to 22-18
Structure Height, h_n =	10.000	ft.
Actual Calc. Period, T_c =	0.000	sec. from independent analysis
Seismic Resist. System =	A15	Light-framed walls sheathed with wood structural panels rated for shear or steel sheets (ASCE 7-10 Table 12.2-1)



V = Cs*W = Σ(Fi) = 0.37 kips

Seismic Base Shear

(Regular Bldg. Configurations Only)

Structure Weight Distribution:

No. of Seismic Levels = 1

Seismic Level x	Height, hx (ft.)	Weight, Wx (kips)
1	10.000	24.00

Total Weight, W = ΣW_x = 24.00 kips (ASCE 7-10 Section 12.7.2)

Results:

Site Coefficients:

F_a =	1.600	ASCE 7-10 Table 11.4-1, page 66
F_v =	2.400	ASCE Table 11.4-2, page 66

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

S_{MS} =	0.150	$S_{MS} = F_a*S_S$, ASCE Eqn. 11.4-1, page 65
S_{M1} =	0.166	$S_{M1} = F_v*S_1$, ASCE Eqn. 11.4-2, page 65

Design Spectral Response Accelerations for Short and 1-Second Periods :

S_{DS} =	0.100	$S_{DS} = 2*S_{MS}/3$, ASCE 7-10 Eqn. 11.4-3, page 65
S_{D1} =	0.110	$S_{D1} = 2*S_{M1}/3$, ASCE Eqn. 11.4-4, page 65

(continued:)

Seismic Design Category:

Category(for S_{DS}) =	A	ASCE 7 Table 11.6-1, page 67
Category(for S_{D1}) =	B	ASCE 7 Table 11.6-2, page 67
Use Category =	B	Most critical of either category case above controls

Fundamental Period:

Period Coefficient, C_T =	0.020	ASCE 7 Table 12.8-2, page 90
Period Exponent, x =	0.75	ASCE 7 Table 12.8-2, page 90
Approx. Period, T_a =	0.112	sec., $T_a = C_T*h_n$^(x), ASCE 7 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., C_u =	1.679	ASCE 7 Table 12.8-1, page 90
Period max., $T_{(max)}$ =	0.189	sec., $T_{(max)} = C_u*T_a$, ASCE 7 Section 12.8.2, page 90
Fundamental Period, T =	0.112	sec., T = Ta <= Cu*Ta, ASCE 7 Section 12.8.2, page 90

Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	6.5	ASCE 7 Table 12.2-1, pages 73-75
Overstrength Factor, Ω_o =	3	ASCE 7 Table 12.2-1, pages 73-75
Defl. Amplif. Factor, C_d =	4	ASCE 7 Table 12.2-1, pages 73-75
C_S =	0.015	$C_S = S_{DS}/(R/I)$, ASCE 7 Section 12.8.1.1, Eqn. 12.8-2
$C_{S(max)}$ =	0.151	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7 Eqn. 12.8-3
$C_{S(min)}$ =	0.010	CS(min) = 0.044*SDS*I >= 0.01, ASCE 7 Eqn. 12.8-5
Use: C_S =	0.015	$C_{S(min)} <= C_S <= C_{S(max)}$

Seismic Base Shear:

V = 0.37 kips, $V = C_S*W$, ASCE 7 Section 12.8.1, Eqn. 12.8-1

Seismic Shear Vertical Distribution:

Distribution Exponent, k = 1.00 k = 1 for T<=0.5 sec., k = 2 for T>=2.5 sec.
k = (2-1)*(T-0.5)/(2.5-0.5)+1, for 0.5 sec. < T < 2.5 sec.

Lateral Force at Any Level: $F_x = C_{vx}*V$, ASCE 7 Section 12.8.3, Eqn. 12.8-11, page 91

Vertical Distribution Factor: $C_{vx} = W_x*h_x$^k/(ΣW_i*h_i^k), ASCE 7 Eqn. 12.8-12, page 91

Seismic Level x	Weight, Wx (kips)	hx^k (ft.)	Wx*h^k (ft-kips)	Cvx (%)	Shear, Fx (kips)	Σ Story Shears
1	24.00	10.000	240.0	1.000	0.37	0.37
Σ =	24.00		240.0	1.000	0.37	

Comments:

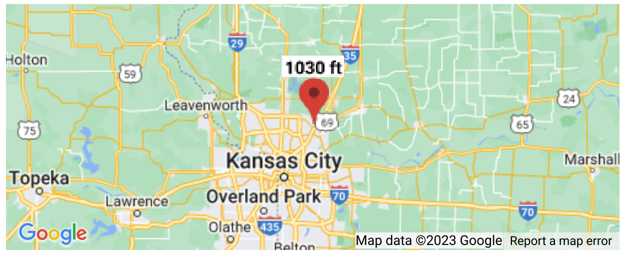
This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Hazards by Location

Search Information

Address:	9705 N Ash Ave, Kansas City, MO 64157, USA
Coordinates:	39.2686359, -94.45099239999999
Elevation:	1030 ft
Timestamp:	2023-07-24T16:38:35.723Z
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

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Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

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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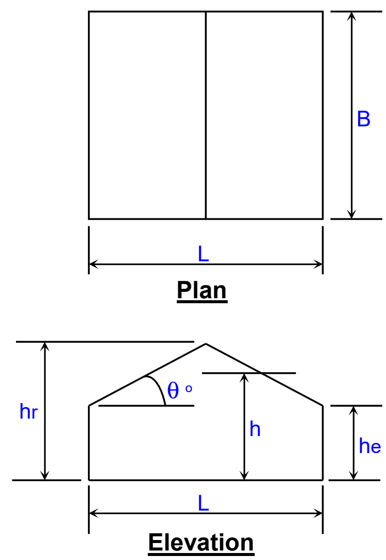
WIND LOADING ANALYSIS - Wall Components and Cladding

Job Name:	SWIG	Subject:			
Job Number:		Originator:	KAG	Checker:	

Input Data:



Wind Speed, V =	110	mph (Wind Map, Figure 26.5-1A-C)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	10.00	ft. (hr >= he)
Eave Height, he =	10.00	ft. (he <= hr)
Building Width =	16.00	ft. (Normal to Building Ridge)
Building Length =	45.00	ft. (Parallel to Building Ridge)
Roof Type =	Monoslope	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	Y	(Sect. 28.6-1 & Figure 26.11-1)
Hurricane Region?	N	
Component Name =	Wall	(Girt, Siding, Wall, or Fastener)
Effective Area, Ae =	533	ft.^2 (Area Tributary to C&C)

Resulting Parameters and Coefficients:

Roof Angle, θ =	0.00	deg.
Mean Roof Ht., h =	10.00	ft. (h = he, for roof angle <=10 deg.)

Wall External Pressure Coefficients, GCp:

GCp Zone 4 Pos. =	0.63	(Fig. 30.4-1, GCp is reduced by 10% for roof angle <=10 deg.)
GCp Zone 5 Pos. =	0.63	(Fig. 30.4-1, GCp is reduced by 10% for roof angle <=10 deg.)
GCp Zone 4 Neg. =	-0.72	(Fig. 30.4-1, GCp is reduced by 10% for roof angle <=10 deg.)
GCp Zone 5 Neg. =	-0.72	(Fig. 30.4-1, GCp is reduced by 10% for roof angle <=10 deg.)

Positive & Negative Internal Pressure Coefficients, GCpi (Figure 26.11-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If z <= 15 then: Kz = 2.01*(15/zg)^(2/α) , If z > 15 then: Kz = 2.01*(z/zg)^(2/α) (Table 30.3-1)

α =	9.50	(Table 26.9-1)
zg =	900	(Table 26.9-1)
Kh =	0.85	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 30.3.2, Eq. 30.3-1)

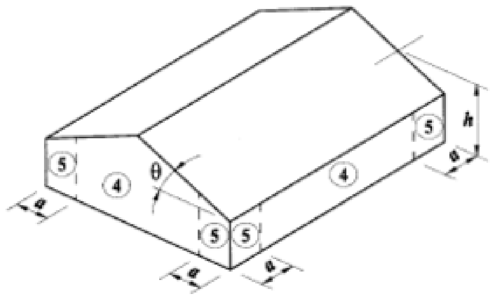
qh = 22.35 psf   qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)

Design Net External Wind Pressures (Sect. 30.4 & 30.6):
For h <= 60 ft.: p = qh*((GCp) - (+/-GCpi)) (psf)
For h > 60 ft.: p = q*(GCp) - qi*(+/-GCpi) (psf)
where: q = qz for windward walls, q = qh for leeward walls and side walls
qi = qh for all walls (conservatively assumed per Sect. 30.6)

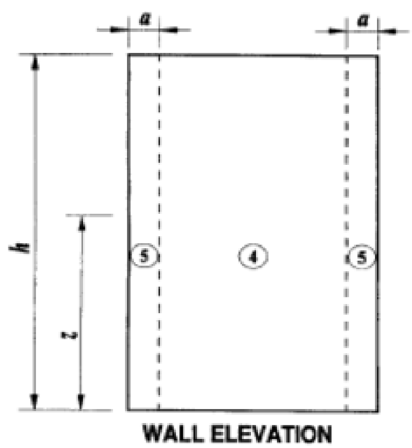
Wind Load Tabulation for Wall Components & Cladding							
Component	z (ft.)	Kh	qh (psf)	p = Net Design Pressures (psf)			
				Zone 4 (+)	Zone 4 (-)	Zone 5 (+)	Zone 5 (-)
Wall	0	0.85	22.35	18.10	-20.12	18.10	-20.12
For z = hr:	10.00	0.85	22.35	18.10	-20.12	18.10	-20.12
For z = he:	10.00	0.85	22.35	18.10	-20.12	18.10	-20.12
For z = h:	10.00	0.85	22.35	18.10	-20.12	18.10	-20.12

Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 5 (end zones), 'a' = 3.00 ft.
3. Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.
4. References : a. ASCE 7-10, "Minimum Design Loads for Buildings and Other Structures".
b. "Guide to the Use of the Wind Load Provisions of ASCE 7
by: Kishor C. Mehta and James M. Delahay

Wall Components and Cladding:



Wall Zones for Buildings with h <= 60 ft.



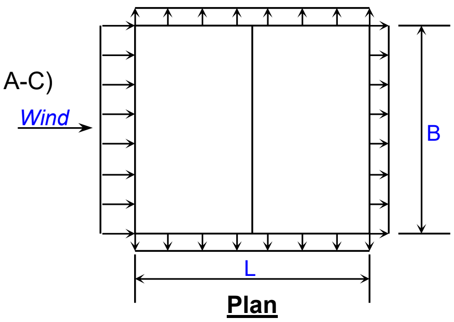
Wall Zones for Buildings with h > 60 ft.

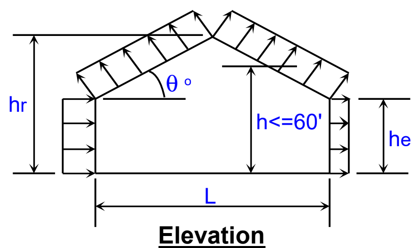
WIND LOADING ANALYSIS - Main Wind-Force Resisting System

Job Name:	SWIG	Subject:			
Job Number:		Originator:	KAG	Checker:	

Input Data:

Wind Speed, V =	110	mph (Wind Map, Figure 26.5-1A-C)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	10.00	ft. (hr >= he)
Eave Height, he =	10.00	ft. (he <= hr)
Building Width =	16.00	ft. (Normal to Building Ridge)
Building Length =	45.00	ft. (Parallel to Building Ridge)
Roof Type =	Monoslope	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	Y	(Sect. 26.2 & Table 26.11-1)
Hurricane Region?	N	





Resulting Parameters and Coefficients:

Roof Angle, θ =	0.00	deg.
Mean Roof Ht., h =	10.00	ft. (h = he, for angle <=10 deg.)

Check Criteria for a Low-Rise Building:

1. Is h <= 60' ? Yes, O.K. 2. Is h <= Lesser of L or B? Yes, O.K.

External Pressure Coeff's., GCpf (Fig. 28.4-1):
(For values, see following wind load tabulations.)
Positive & Negative Internal Pressure Coefficients, GCpi (Table 26.11-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If h < 15 then: Kh = 2.01*(15/zg)^(2/α) (Table 28.3-1)
If h >= 15 then: Kh = 2.01*(z/zg)^(2/α) (Table 28.3-1)

α =	9.50	(Table 26.9-1)
zg =	900	(Table 26.9-1)
Kh =	0.85	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 28.3.2, Eq. 28.3-1)

qh = 22.35 psf qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)

Design Net External Wind Pressures (Sect. 28.4.1):
p = qh*[(GCpf) - (+/-GCpi)] (psf, Eq. 28.4-1)

Wall and Roof End Zone Widths 'a' and '2*a' (Fig. 28.4-1):

a =	3.00	ft.
2*a =	6.00	ft.

MWFRS Wind Load for Load Case A				MWFRS Wind Load for Load Case B			
Surface	GCpf	p = Net Pressures (psf)		Surface	*GCpf	p = Net Pressures (psf)	
		(w/ +GCpi)	(w/ -GCpi)			(w/ +GCpi)	(w/ -GCpi)
Zone 1	0.40	4.92	12.96	Zone 1	-0.45	-14.08	-6.03
Zone 2	-0.69	-19.45	-11.40	Zone 2	-0.69	-19.45	-11.40
Zone 3	-0.37	-12.29	-4.25	Zone 3	-0.37	-12.29	-4.25
Zone 4	-0.29	-10.50	-2.46	Zone 4	-0.45	-14.08	-6.03
Zone 5	---	---	---	Zone 5	0.40	4.92	12.96
Zone 6	---	---	---	Zone 6	-0.29	-10.50	-2.46
Zone 1E	0.61	9.61	17.66	Zone 1E	-0.48	-14.75	-6.71
Zone 2E	-1.07	-27.94	-19.89	Zone 2E	-1.07	-27.94	-19.89
Zone 3E	-0.53	-15.87	-7.82	Zone 3E	-0.53	-15.87	-7.82
Zone 4E	-0.43	-13.63	-5.59	Zone 4E	-0.48	-14.75	-6.71
Zone 5E	---	---	---	Zone 5E	0.61	9.61	17.66
Zone 6E	---	---	---	Zone 6E	-0.43	-13.63	-5.59

*Note: Use roof angle θ = 0 degrees for Longitudinal Direction.

For Case A when GCpf is neg. in Zones 2/2E:
Zones 2/2E dist. = 8.00 ft.

For Case B when GCpf is neg. in Zones 2/2E:
Zones 2/2E dist. = 22.50 ft.

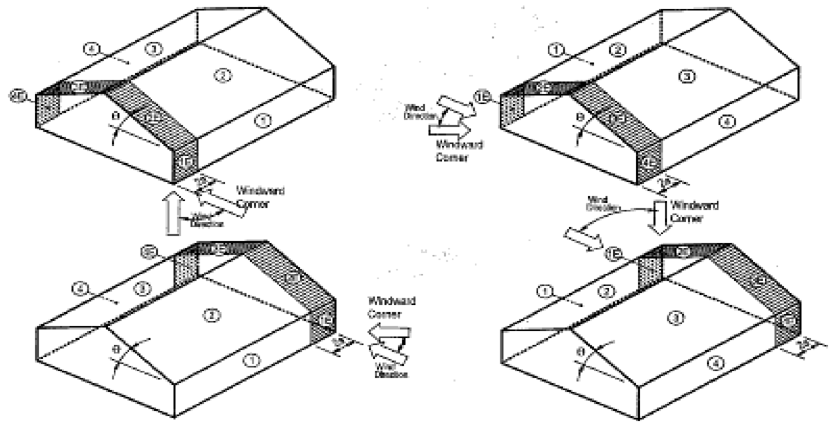
Remainder of roof Zones 2/2E extending to ridge line shall use roof Zones 3/3E pressure coefficients.

MWFRS Wind Load for Load Case A, Torsional Case				MWFRS Wind Load for Case B, Torsional Case			
Surface	GCpf	p = Net Pressure (psf)		Surface	GCpf	p = Net Pressure (psf)	
		(w/ +GCpi)	(w/ -GCpi)			(w/ +GCpi)	(w/ -GCpi)
Zone 1T	---	1.23	3.24	Zone 1T	---	-3.52	-1.51
Zone 2T	---	-4.86	-2.85	Zone 2T	---	-4.86	-2.85
Zone 3T	---	-3.07	-1.06	Zone 3T	---	-3.07	-1.06
Zone 4T	---	-2.63	-0.61	Zone 4T	---	-3.52	-1.51
Zone 5T	---	---	---	Zone 5T	---	1.23	3.24
Zone 6T	---	---	---	Zone 6T	---	-2.63	-0.61

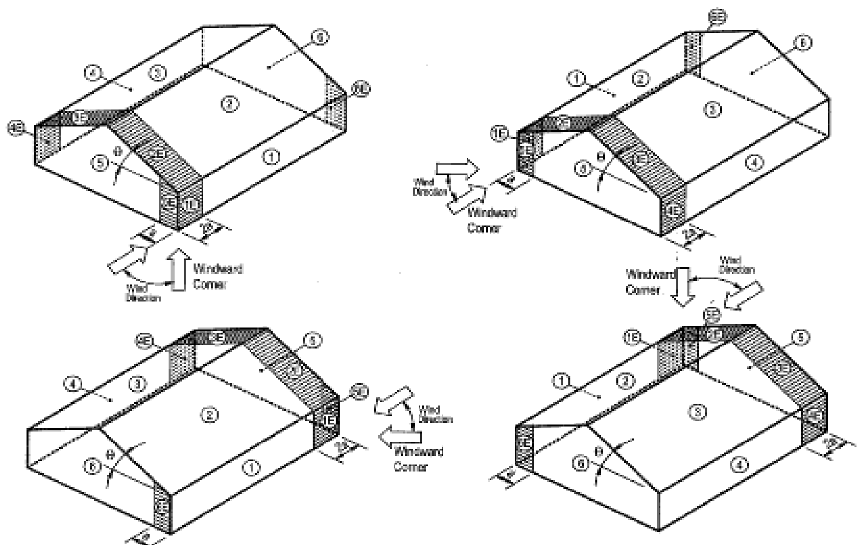
Notes: 1. For Load Case A (Transverse), Load Case B (Longitudinal), and Torsional Cases:
Zone 1 is windward wall for interior zone.
Zone 2 is windward roof for interior zone.
Zone 3 is leeward roof for interior zone.
Zone 4 is leeward wall for interior zone.
Zones 5 and 6 are sidewalls.
Zone 1T is windward wall for torsional case
Zone 3T is leeward roof for torsional case
Zones 5T and 6T are sidewalls for torsional case.
Zone 1E is windward wall for end zone.
Zone 2E is windward roof for end zone.
Zone 3E is leeward roof for end zone.
Zone 4E is leeward wall for end zone.
Zone 5E & 6E is sidewalls for end zone.
Zone 2T is windward roof for torsional case.
Zone 4T is leeward wall for torsional case.

2. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
3. Building must be designed for all wind directions using the 8 load cases shown below. The load cases are applied to each building corner in turn as the reference corner.
4. Wind loads for torsional cases are 25% of respective transverse or longitudinal zone load values. Torsional loading shall apply to all 8 basic load cases applied at each reference corner.
   Exception: One-story buildings with "h" <= 30', buildings <= 2 stories framed with light frame construction, and buildings <=2 stories designed with flexible diaphragms need not be designed for torsional load cases.
5. Per Code Section 28.4.4, the minimum wind load for MWFRS shall not be less than 16 psf.

Low-Rise Buildings h <= 60'

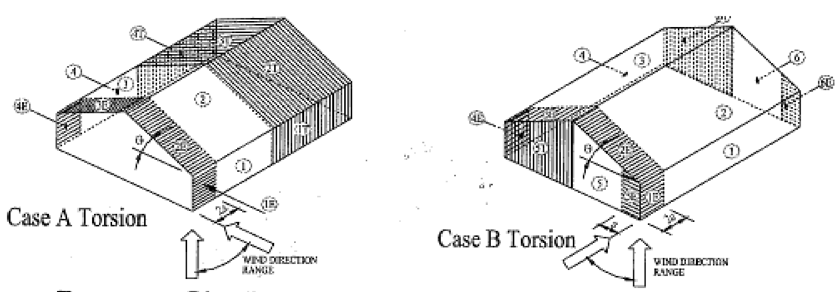


Load Case A



Load Case B

Basic Load Cases



Transverse Direction

Longitudinal Direction

Torsional Load Cases

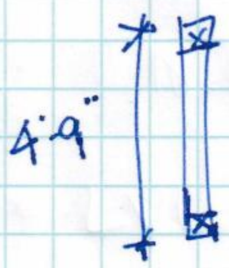
SWIG. 9705 Ash KCMO ①

Wall that Spans. Roof

Span = 16'-0"

Allowable Wind Load = $0.6\left[20.12\ psf \times 2\right] = \cancel{\#}\ 24.1\ psf.$

Wall Height



$w_{WL} = 24.1\ psf\ (1/2)(4.75')$
$= 57.24\ plf. \leftrightarrow$

$w_{DL} = 8psf\ (4.75') = 38\ plf \downarrow$

Try Double 2x10.

Vert Load

per Forte. Moment @ 47%

$M_{all} = 3177\ lb \cdot ft$
$M_{act} = 1486\ lb \cdot ft.$

Double 2x12

Moment @ 16%.

$M_{all} = 7568\ lb \cdot ft$
$M_{act} = 1216\ lb \cdot ft$

Horizontal Load

~~Per Forte.~~

Moment actual = ~~1977~~ ~~2191~~ 1834. lb·ft.

Double 2x10. Mallowable = ~~1895.~~ 2191

@ 84%.

SW1G.

Double 2x6 @ Horiz

Mactual = ~~[illegible]~~ 1834 lb·ft @ 78%

M allowable = 2355

Combined Section - Horizontal Force.

2x10s

$f_b = 1586$ psi

$F_b = 1895$ psi

2x6

$f_b = 1455$ psi

$F_b = 1868$ psi

$$\frac{1586 + 1455}{1895 + 1869} = 0.801$$

Combined Stress = 0.801 + 0.161 = 0.962

Horiz Deflection

$$\frac{w\ell^4}{1289\, I} = \Delta.$$

Ieq

$$I_{eq} = \frac{w\ell^4}{1289\,\Delta.}$$

2x10s 2x6s

$$= \frac{.057\,(16)^4}{1289\,(2.54\,in^4)} + \frac{0.057\,(16)^4}{1289\,(1.22\,in^4)}$$

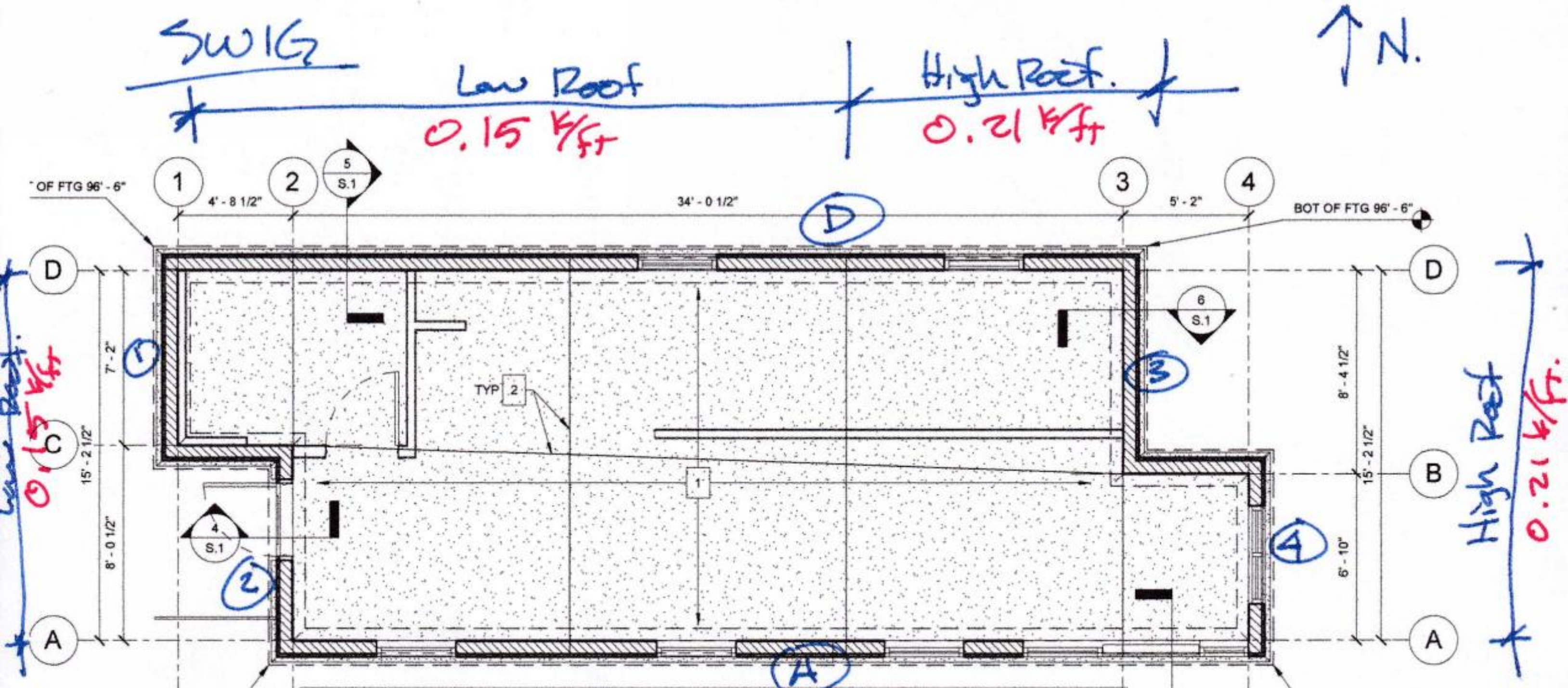
$$I_y = 1.141\ in^4 + 2.282\,in^4 = 3.423\,in^4.$$

$$\Delta_{combined} = \frac{0.057\,(16)^4}{1289\,(3.423)}$$

$$= 0.8466''$$

$$\cancel{\frac{16\cdot 12}{0.8}}$$

$$\frac{16'\cdot 12}{0.8466} = L/227.$$



FOUNDATION PLAN NOTES

1. 4" CONCRETE SLAB ON GRADE WITH 10 MIL VAPOR RETARDER AND 4" CRUSHED ROCK DRAINAGE FILL. REINFORCE PER TYPICAL DETAILS. TOP OF CONCRETE ELEVATION = 100'-0" U.N.O.
2. CONTROL JOINTS PER DETAIL 3/S.0.
3. HATCH PATTERN INDICATES LOCATION OF SHEAR WALLS. PROVIDE 15/32" STRUCTURAL SHEATHING ATTACHED TO WALL STUDS WITH 10d COMMON NAILS SPACED @ 12" O.C. IN THE FIELD AND 4" O.C. AT THE EDGE.

MWF Pressure :

$W_{allowable}$: $0.6[9.61 psf + 17.66 psf] = 16.36 psf.$

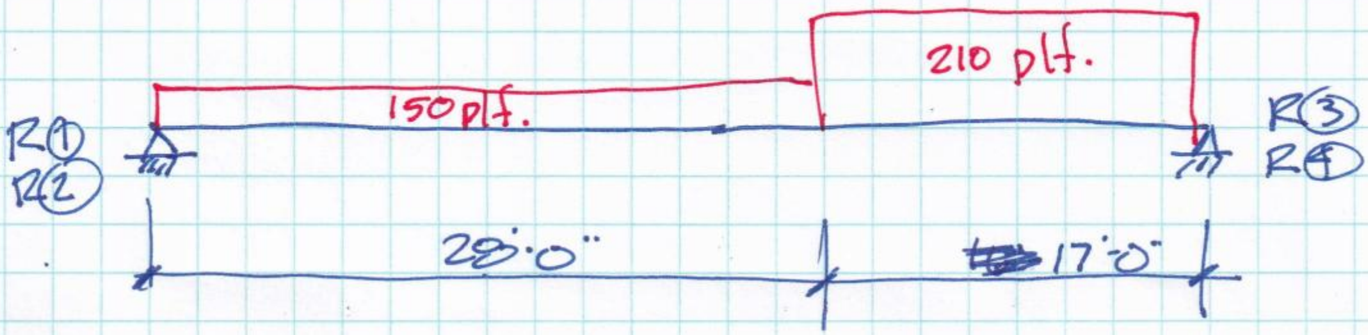
$w_{Reaction}$ @ Roof (~~N-S~~)

High - 149.1 plf. = 0.15 k/ft.

Low - 209.4 plf. = 0.21 k/ft.

SW1G.

Wall A or D



$$R_{1,2} = 3568 \text{ lbs.}$$

$$R_{3,4} = 4202 \text{ lbs.}$$

Pro Rate Based on Length.

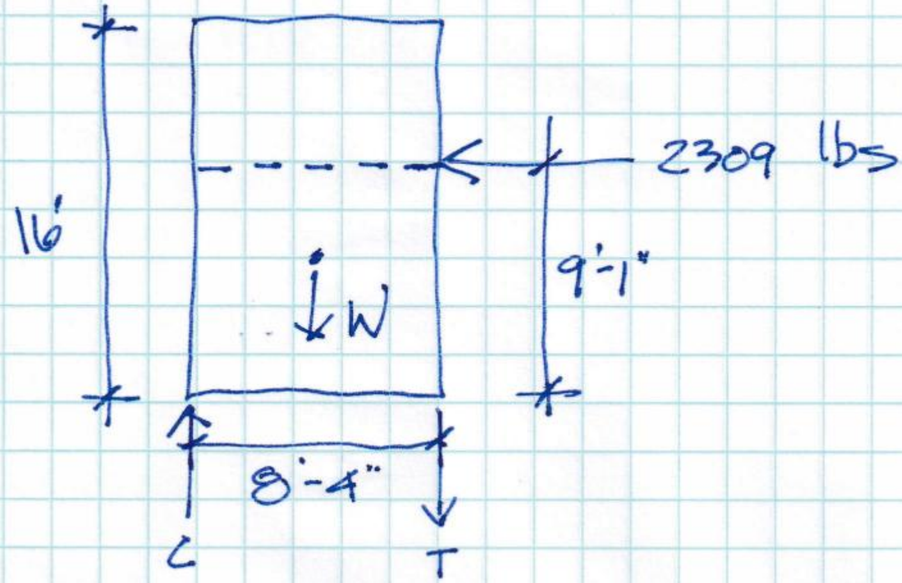
$$R_1 = 3568 \text{ lbs} \left(\frac{7.167'}{15.167'} \right) = 1686 \text{ lbs.}$$

$$R_2 = 3568 \text{ lbs} \left(8/15.167' \right) = 1882 \text{ lbs.}$$

$$R_3 = 4202 \text{ lbs} \left(\frac{8.333'}{15.167'} \right) = 2309 \text{ lbs.}$$

$$R_4 = 4202 \text{ lbs} \left(\frac{6.833'}{15.167'} \right) = 1893 \text{ lbs.}$$

End Wall Analysis



$$M_o = 2309\ lbs\ (9.083') = 20{,}973\ lb\cdot ft.$$

$$W = 8\,psf\,(8.333' \times 16') + 13\,psf\,(\cancel{0.667'}\ 1')(16')$$

$$= 1067\ lbs + 208\ lbs.$$

$$M_R = 0.6\left[(1067\ lbs + 208\ lbs)\left(\tfrac{1}{2}\right)(8.333')\right]$$

F.S.

$$= 3187\ lb\cdot ft \ll 20{,}973\ lb\cdot ft$$

$$M_{net} = 20{,}973\ lb\cdot ft - 3187\ lb\cdot ft = 17{,}786\ lb\cdot ft$$

$$T = C = \frac{17{,}786\ lb\cdot ft}{8.333'} = \boxed{2134\ lbs.\ Uplift}$$

$$w = \frac{2309\ lbs}{8.333'} = \boxed{277\ plf}$$ Shear wall Force

Concrete Holddown

Influence Width = 4' + 4'

Uplift Force = 2134 lbs.

Ftg = [1.5' × 1'-0"] 8' × 150pcf = 1,800 #

Wall = [2.5' × 0.5'] 8' × 150pcf = 1,500 #

Slab = $[0.333' \times \underbrace{8^2 \times 1/2}_{\text{Triangle Influence Area}}] \times 150$pcf. = 1,600 #

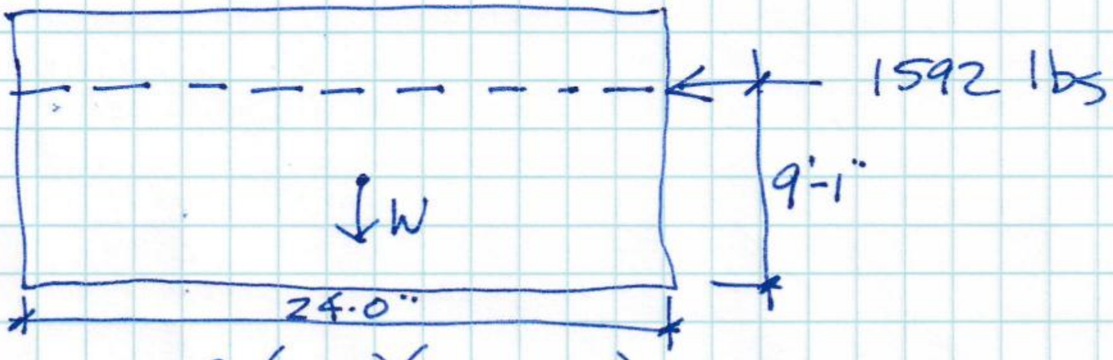
4900 #.

0.6[4900] = 2940 lbs > 2134 lbs.

SWIG. (6).

Long Wall Analysis. (Non-Segmented Shear Wall)

Wall (A)



$F_A = 210\,plf\left(\tfrac{1}{2}\right)(15.167') = 1592\ lbs.$

$M_O = 1592\ lbs\,(9.083') = 14{,}460\ lb\cdot ft.$

$W = 8\,psf\,(24')(13.5') + 13\,psf\left(\tfrac{1}{2}\right)(15.167')(24')$

$= 2592\ lbs + 2366\ lbs.$

$= 4958\ lbs.$

$M_R = 0.6\left[4958\ lbs \cdot 12'\right]$

$= 35{,}697\ lb\cdot ft > 14{,}460\ lb\cdot ft$

$\therefore$ No Uplift.

SWIG Seismic Analysis

Per spreadsheet - Base Shear = 370 lbs.

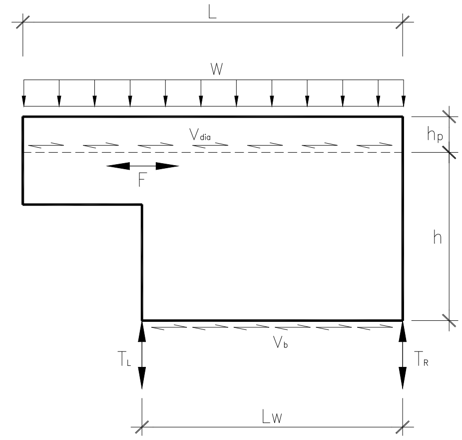
by observation - Wind Controls.

PROJECT : SWIG
SUBJECT : Exterior Shear Walls
JOB NO. :
DATE : 7/25/2023

PAGE :
DESIGN BY : KAG
REVIEW BY :

Shear Wall Design Based on IBC

INPUT DATA



LATERAL FORCE ON DIAPHRAGM: $v_{dia,\ WIND}$ = 277 plf,for wind
$v_{dia,\ SEISMIC}$ = 15 plf,for seismic, ASD

GRAVITY LOADS ON THE ROOF: w_{DL} = 153 plf,for dead load
w_{LL} = 25 plf,for live load

DIMENSIONS: L_w = 8.333 ft , h = 9.083 ft
L = 8.333 ft , h_p = 9.083 ft

PANEL GRADE (0 or 1) = 1 <= Sheathing and Single-Floor
MINIMUM NOMINAL PANEL THICKNESS = 15/32 in
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) 2 10d
SPECIFIC GRAVITY OF FRAMING MEMBERS 0.5
EDGE STUD SECTION 1 pcs, b = 5.5 in , h = 5.5 in
SPECIES (1 = DFL, 2 = SP) 1 DOUGLAS FIR-LARCH
GRADE (1, 2, 3, 4, 5, or 6) 6 No. 2
STORY OPTION (1=ground level, 2=upper level) 1 ground level shear wall

THE SHEAR WALL DESIGN IS ADEQUATE.

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 6 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 38 in O.C.

HOLD-DOWN FORCES: T_L = 1.69 k , T_R = 1.69 k (USE PHD2-SDS3 SIMPSON HOLD-DOWN)
DRAG STRUT FORCES: F = 0.00 k
EDGE STUD: 1 - 6" x 6" DOUGLAS FIR-LARCH No. 2, CONTINUOUS FULL HEIGHT.
SHEAR WALL DEFLECTION: Δ = 0.26 in

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO L / B = 1.1 < 3.5 **[Satisfactory]**
DETERMINE REQUIRED CAPACITY v_b = 277 plf, (1 Side Diaphragm Required, the Max. Nail Spacing = 6 in)

THE SHEAR CAPACITIES PER IBC Table 2306.3 / SDPWS-08 Table 4.3A with ASD reduction factor 2.0)

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Sheathing and Single-Floor	10d	1 5/8	15/32	310	460	600	770

Note: 1. The indicated shear numbers have reduced by specific gravity factor per IBC note a.
2. Since the wall is blocked, SDPW-08 Table 4.3.3.2 does not apply.

DETERMINE DRAG STRUT FORCE: F = $(L-L_w)$ MAX($v_{dia,\ WIND}$, $\Omega_0 v_{dia,\ SEISMIC}$) = 0.00 k (Ω_0 = 1) (Sec. 1633.2.6)

DETERMINE MAX SPACING OF 5/8" DIA ANCHOR BOLT (NDS 2005, Tab.11E)
5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 38 in O.C.
THE HOLD-DOWN FORCES:

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	15	242	3335	Left	10358	0.9	T_L =	0	PHD2-SDS3
				Right	10358	0.9	T_R =	0	
WIND	277		20966	Left	10358	2/3	T_L =	1687	
				Right	10358	2/3	T_R =	1687	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

CHECK MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3 / SDPWS-08 4.3.2)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ \ slip} + \Delta_{Chord\ \ splice\ \ slip} = \frac{8 v_b h^3}{E A L_w} + \frac{v_b h}{G t} + 0.75 h e_n + \frac{h d_a}{L_w} = 0.264 \text{ in, ASD} < $$

$\delta_{xe,allowable,\ ASD}$ = 0.389 in
[Satisfactory] (ASCE 7 12.8.6)

Where: v_b = 277 plf , ASD L_w = 8 ft E = 1.7E+06 psi
A = 16.50 in^2 h = 9 ft G = 9.0E+04 psi C_d = 4 I = 1
t = 0.298 in e_n = 0.000 in d_a = 0.15 in ,(ASCE 7 Tab 12.2-1 & Tab 11.5-1)
Δ_a = 0.02 h_{sx}
, (ASCE 7 Tab 12.12-1)

CHECK EDGE STUD CAPACITY

P_{max} = 2.51 kips, (2/3 value. This value should include upper level DOWNWARD loads if applicable)
F_c = 700 psi C_D = 1.60 C_P = 0.57 A = 25 in^2
E = 1300 ksi C_F = 1.00 F_c' = 641 psi > f_c = 100 psi
[Satisfactory]