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Structural
Mechanical
Electrical
Plumbing
Energy

Cota Vera Swim Club for Homefed Corporation

to be constructed in Chula Vista, CA

Structural Calculations per 2022 CBC for Plan # Segment 1

Harris & Sloan Job # HS22244

REGISTERED PROFESSIONAL ENGINEER
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STATE OF CALIFORNIA

Initial Submittal Date: 1/13/2023

Design Loads

Gravity System

Gravity loads are summarized on the following pages, based on typical light framing and the details and specifications provided by the project architect. Loads are supported through plated wood trusses at the roof level and manufactured wood I-joists at the floor levels; framing members are supported on light-framed wood bearing walls, with wood beams and posts provided where required. Building loads are supported on a foundation designed in accordance with the recommendations of the project soils report.

Lateral Force-Resisting System

Wind design utilizes the directional procedure outlined in ASCE 7 chapter 27; seismic design is based on the Equivalent Lateral Force procedure outlined in ASCE 7 chapter 11 and chapter 12. Lateral loads are calculated in accordance with ASCE 7 using building geometry, gravity loads as determined above. Resistance to lateral forces is provided by wood shearwalls, with Simpson Steel Strong-walls provided at the first floor along the front of the garages. Lateral loads are transferred into the vertical elements of the Main Force Resisting System (MFRS) using horizontal wood diaphragms, with collectors provided along each line of lateral force resistance. Uplift forces at the wood shearwalls are resisted through metal strap holdowns at the third-to-second and second-to-first floor levels and metal holdowns at the foundation level.

The seismic dead loads were determined by combining the total dead load (21 psf at the roof; 15 psf at the floor) and a portion of the wall dead load perpendicular to the direction of the loading. The wall dead loads used (9 psf at the roof; 15 psf at the floor) are approximated based on the tributary area of the diaphragm. The wall dead load at the roof is a conservative estimate to account for gable end scenarios. This seismic dead load is separate from the dead load reduction used for overturning calculations per ASCE 7-16 §12.4.3.

Structural Calculation Package

Client Information

Homefed Corporation
1903 Wright Place, Suite 200
Carlsbad, CA 92008

Project Information

Cota Vera Swim Club
Chula Vista, CA
Plan No. Segment 1

Loading Information

Roof Loads

Roofing (Tile)	10.0	psf
Sheathing	1.8	psf
Framing	2.5	psf
Insulation	1.0	psf
Ceiling	2.5	psf
Sprinklers	1.0	psf
Solar	1.2	psf
Misc.	1.0	psf
Wall (Seismic only)	9.0	psf
Total DL	21.0	psf
Total DL (Seismic)	30.0	psf
Total LL	20.0	psf

Floor Loads

Flooring	3.0	psf
Sheathing	2.5	psf
Framing	2.5	psf
Insulation	1.0	psf
Ceiling	2.5	psf
Sprinklers	1.0	psf
Misc.	2.5	psf
Wall (Seismic only)	15.0	psf
Total DL	15.0	psf
Total DL (Seismic)	30.0	psf
Total LL	40.0	psf

Exterior Wall Loads

Stucco (7/8")	9.0	psf
Gyp Board (One Face)	2.5	psf
Sheathing (1/2")	1.7	psf
Framing (2x6)	1.3	psf
Insulation	1.0	psf
Misc	0.5	psf
Total DL	16.0	psf

Interior Wall Loads

Gyp Board (Ea Face)	5.0	psf
Framing (2x6)	1.0	psf
Insulation	1.0	psf
Misc	0.5	psf
Total DL	7.5	psf

Governing Building Codes & Design Standards

- 2022 California Building Code

- ASCE 7-16
- 2018 NDS
- 2021 SDPWS
- PTI Manual, 6th Edition
- TMS 402/ACI530/ASCE 7
- AISC 360

Wind Design Per IBC/ASCE 7 Chapters 26, 27, & 30

Building Information

Roof Pitch (worst case)	12.00 : 12	pitch
Mean Roof Height (h)	18.25	ft
Directionality Factor (K_d)	0.85	(ASCE 7 Table 26.6-1)
Gust Factor (*G*)	0.85	(ASCE 7 26.11.1)
Risk Category	II	(ASCE Table 1.5-1)
Site Elevation (z_g)	0	ft

Building Dimensions	Max	Min
Length (L)	50.5 ft.	9.0 ft.
Width (B)	37.0 ft.	24.0 ft.

Site Information

Basic Wind Speed (*V*)	96	mph
Exposure Category	C	(ASCE 7 26.7.3)
Hill Type	None	
Hill Height, (*H*)	NA	ft
Hill Length, (L_h)	NA	ft
Distance to Peak, (*x*)	NA	ft
K_1	0.000	
K_2	1.000	
K_e	1.000	

Principal Code Equations

ASCE 7 - Eqn 26.10-1 (MWFRS) ASCE 7 - Eqn 26.10-1 (C&C)

$$q_z = 0.00256 K_z K_{zt} K_d K_e V^2 \text{ (lb/ft}^2\text{); } V \text{ in mi/h}$$

ASCE 7 - Figure 26.8-1 Eqns (Topo Effects)

$$K_{zt} = (1 + K_1 K_2 K_3)^2$$

ASCE 7 - Eqn 28.3-1 (MWFRS)

$$p = qGC_p - q_i(GC_{pi}) \text{ (lb/ft}^2\text{)}$$

ASCE 7 - Eqn30.3-1 (C&C)

$$p = q_h[(GC_p) - (GC_{pi})] \text{ (lb/ft}^2\text{)}$$

$$K_2 = (1 - \frac{|x|}{\mu L_h}) \qquad K_3 = e^{-\gamma z/L_h}$$

Velocity Pressures by Height

Adjustment Factors & Pressures by Height						
Heignt	Height Factors		MWFRS		Comp's and Cladding	
z (ft)	K_3	K_{zt}	K_z	q_z (psf)	K_z	q_z (psf)
15	1.000	1.000	0.849	10.21	0.849	10.21
15.41	1.000	1.000	0.854	10.27	0.854	10.27
15.81	1.000	1.000	0.858	10.33	0.858	10.33
16.22	1.000	1.000	0.863	10.38	0.863	10.38
16.63	1.000	1.000	0.867	10.44	0.867	10.44
17.03	1.000	1.000	0.872	10.49	0.872	10.49
17.44	1.000	1.000	0.876	10.54	0.876	10.54
17.84	1.000	1.000	0.880	10.59	0.880	10.59
18.25	1.000	1.000	0.885	10.64	0.885	10.64
23	1.000	1.000	0.931	11.20	0.931	11.20

K_z Per ASCE 7 Table 26.10-1; K_{zt} Per ASCE 7 Figure 26.8-1

Pressure at Mean Roof Height, qh =	10.6 psf	(MWFRS)
Pressure at Mean Roof Height, qh =	10.6 psf	(C&C)

Horizontal Wind Pressures, C&C

Horizontal wind pressures used for the design of the component and cladding elements are determined using the procedure outlined in ASCE 7, Chapter 30

Walls (Components & Cladding)								
Stud	Min Stud	GCp (min)		GCp (max)		Gcpi	p (psf)	
Height (ft)	Spacing (in)	Zone 4	Zone 5	Zone 4	Zone 5		Zone 4	Zone 5
8	12	-1.04	-1.28	1.00	1.00	-0.18	13.01	15.58
9	12	-1.02	-1.25	1.00	1.00	-0.18	12.81	15.20
10	12	-1.01	-1.22	1.00	1.00	-0.18	12.64	14.85
11	12	-0.99	-1.19	0.99	0.99	-0.18	12.49	14.54
12	12	-0.98	-1.16	0.99	0.99	-0.18	12.41	14.26
15	12	-0.95	-1.09	0.97	0.97	-0.18	12.23	13.53
19	12	-0.91	-1.02	0.95	0.95	-0.18	12.04	12.76
22	12	-0.89	-0.97	0.94	0.94	-0.18	11.92	12.28

Horizontal Wind Pressures, MWFRS

Horizontal wind pressures used for the design of the main wind force resisting system are determined using the directional procedure outlined in ASCE 7, Chapter 27

Horizontal Wind Coefficients by Surface, Cp						
	Walls		Pitched Roof			Parapet
Direction	Left-Right	Front-Back	Either Direction			
L/B_{min}, H/L_{max}	0.48	0.24	0.25	0.50	1.00	N/A
$Windward_1$	0.8	0.8	0.00	0.00	0.00	1.50
$Windward_2$	0.8	0.8	0.40	0.40	0.30	1.50
Leeward	-0.50	-0.5	-0.60	-0.60	-0.60	-1.00
Total	1.30	1.3	1.00	1.00	0.90	2.50

Wind Pressure by Surface & Height								
Height	Single-Sided Wind			Two-Sided (Standard) Wind				Parapet
	Walls	Pitched Roof		Walls		Pitched Roof		
		Left-Right	Front-Back	Left-Right	Front-Back	Left-Right	Front-Back	
15	8.86	5.21	5.21	11.47	11.47	9.05	9.05	21.71
15	8.90	5.24	5.24	11.51	11.51	9.05	9.05	21.83
16	8.94	5.27	5.27	11.55	11.55	9.05	9.05	21.95
16	8.98	5.30	5.30	11.58	11.58	9.05	9.05	22.06
17	9.01	5.32	5.32	11.62	11.62	9.05	9.05	22.18
17	9.05	5.35	5.35	11.66	11.66	9.05	9.05	22.29
17	9.09	5.38	5.38	11.69	11.69	9.05	9.05	22.40
18	9.12	5.40	5.40	11.73	11.73	9.05	9.05	22.51
18	9.15	5.43	5.43	11.76	11.76	9.05	9.05	22.62
23	9.53	5.71	5.71	12.14	12.14	8.82	9.05	23.80

Vertical Wind Pressures, MWFRS

Calculation of roof dead load available to offset overturning of shearwalls.

Avg. Pressure Coeff. (C_p)	-0.59	
Int. Pressure Coeff. (GC_{pi})	-0.18	(ASCE 7 Table 26.13-1)
Wind Uplift Pressure (p)	-8 psf	

Controlling Load Combo	0.6D+0.6W	(ASCE 7 2.4.1)
Net pressure from Roof	4.4 psf	Available to offset overturning from wind

Calculation comparing C&C Wind Loads to capacity of roofing nails in withdrawal

Calculation does not account for any dead load and assumes smooth shank stainless steel roof nails (worst-case).

Worst-Case Ext. Pressure Coeff. (GC_{pi})	-3.60	(ASCE 7 Figure 30.3-2B)
Wind Uplift Pressure (p)	-40.2 psf	
Net Uplift on 4'x8' piece of shtg	-1288 lbs	

# of nails in a 4'x8' piece of shtg nailed at 6" oc edge, 12" oc field — 57 nails

Per NDS Table 12.2D, 8d nails are good for 22 lb/inch in withdrawal

Assuming 23/32" roof shtg (worst-case), ea nail will have 1.78" penetration — 39.2 lbs per nail

Therefore, 4'x8' piece of roof shtg is capable of withstanding — 2232 lbs in uplift

12" oc field nailing **OK**

Seismic Design Per IBC Section 1613 & ASCE 7 Chapters 11 & 12

Building Information

R	6.50	ASCE Table 12.2-1
Risk Category	II	ASCE Table 1.5-1
Number of Stories	1	
Importance Factor	1.0	
Structural Height	12	ft
Design Approach	Equivalent Lateral Force	

Site Information

S_s	0.754	IBC Sect. 1613.3.1
S_1	0.275	IBC Sect. 1613.1.1
Site Class	C	

Seismic Loads: ASCE 7 Section 12.8 Equivalent Lateral Force Procedure

Principal Code Equations

ASCE Eqn. 12.8-1

$$V = C_s W$$

ASCE Eqn. 12.8-2

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$

ASCE Eqn. 12.8-3

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$$

ASCE Eqn. 12.8-5

$$C_s = 0.044 S_{DS} I_e \geq 0.01$$

ASCE Eqn. 12.8-6

$$C_s = 0.5 S_1 / (R/I_e)$$

Short Period Response

F_a	1.200	CBC 1613.2.3
$S_{MS} = F_a S_s$	0.905	CBC Eqn. 16-36
$S_{DS} = (^2/_3) S_{MS}$	0.603	CBC Eqn. 16-38
SDC per S_{DS}	D	CBC Table 1613.2.5(1)

1-Second Period Response

F_v	1.500	CBC 1613.2.3
$S_{M1} = F_v S_1$	0.413	CBC Eqn. 16-37
$S_{D1} = (^2/_3) S_{M1}$	0.275	CBC Eqn. 16-39
SDC per S_{D1}	D	CBC Table 1613.2.5(2)

Seismic Design Category

Period, T	0.13	s, ASCE 7 12.8.2.1
0.8 Ts	0.36	s, ASCE 7 11.4.6
SDC Required	D	CBC Sect. 1613.2.5
SDC Used	**D**	

ASD Seismic Response Coefficient

C_s	0.093	ASCE Eqn. 12.8-2
C_s (upper limit)	0.328	ASCE Eqn. 12.8-3
C_s (lower limit)	0.027	ASCE Eqn. 12.8-5
C_s (alt low limit)	0.021	ASCE Eqn. 12.8-6
C_s	0.093	

Seismic Design Factors

Overstrength Factor	2.5	Table 12.2-1, Footnote b
Dead Load Reduction:		
(0.6 - 0.14 Sds)D	0.516 D	ASCE Sect 2.4.5 & Eqn. 12.4-4a
Rho, left to right	1.0	ASCE Sect 12.3.4
Rho, front to back	1.0	

Base Shear, V **0.065 W**

(Includes 0.7 factor from ASD Basic LC)

Seismic Loads: ASCE 7 Section 12.8 Equivalent Lateral Force Procedure

Principal Code Equations

ASCE Eqn. 12.8-11

$$F_x = C_{vx} V$$

ASCE Eqn. 12.8-12

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

ASCE Eqn. 12.10-1

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$

ASCE Eqn. 12.10-2

$$F_{px} = 0.2 S_{DS} I_e w_{px}$$

ASCE Eqn. 12.10-3

$$F_{px} = 0.4 S_{DS} I_e w_{px}$$

Vertical Shear Distribution

Vertical distribution of shear is per ASCE 7 Eqn 12.8-12. The total force at each level (F_{px}) is distributed to each line of lateral force-resistance based on the seismic weigh tributary to that line of resistance (wx)

Vertical Force Distribution							
Level	h (ft)	Area (sq ft)	DL (psf)	w_x (lb)	w_x x h	C_{vx}	F_x
1	18.25	1545	30	46350	845887.5	1.0000	3011 lb
Totals		1545		46350	845887.5		**3011 lb**

Diaphragm Forces

Diaphragm shear loads are determined per ASCE 7 Eqn 12.10-1 through 12.10-3. The total force at each level (F_{px}) is distributed to each line of lateral force-resistance based on the seismic weigh tributary to that line of resistance (wx).

Diaphragm Forces							
Story	Fx	∑ Fi	w_x (lb)	∑ wi	∑ Fi / ∑ wi	F_{px} (lb)	% of F_x
1	3011 lb	3011 lb	46350 lb	46350 lb	0.0650	3914 lb	130%

Typical Header Capacities (plf)

The following table is a summary of the maximum amount of load a typical header can take in pounds per linear foot. These capacities are based on analysis using Enercalc software in which each of the typical headers is loaded to the point before failure. Full calculations supporting the capacity table are available upon request.

Header Size/ Span	3 ft	5 ft	6 ft
(2) 2x6	1190	440	310
(2) 2x8	1920	710	494
(2) 2x10	2850	1050	740
(2) 1.25x9.5 1.3E SCL	4240	1550	1070
4x4	650	240	125
4x6	1390	520	360
4x8	2420	900	630
4x10	3640	1340	940
3.5x9.5 1.5E SCL	7910	2940	2040
4x6 flat	890	330	200
6x6	2260	840	580
6x8	4200	1560	1080
6x10	7500	2800	1960

Typical Header Specifications

Below are calculations for typical headers based on the capacity table above. Note that header capacities highlighted in red symbolize the demand load exceeding capacity.

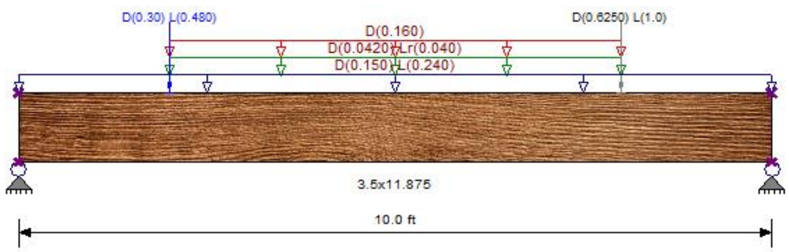
1st Floor Bearing Wall Headers								
Opening	Tributary Widths			Total Load		Header		Trimmers
	Roof	Floor	Walls	Distributed	Reaction	Size	Capacity	
3 ft	17 ft	0 ft	0	697 plf	1046 #	(2) 2x6	1190 plf	1
						4x6	1390 plf	1
						6x6	2260 plf	1
5 ft	13 ft	0 ft	0	533 plf	1333 #	(2) 2x8	710 plf	1
						4x8	900 plf	1
						6x6	840 plf	1
6 ft	13 ft	0 ft	0	533 plf	1599 #	(2) 2x10	740 plf	1
						4x8	630 plf	1
						6x8	1080 plf	1

Beam Calculation Summary

Simply supported beams have been designed using the shear and bending equations outlined in the NDS. The beam analysis allows for three distributed loads based on tributary wall/roof/floor widths (W_A-W_C), one trapezoidal load (W_{D1}/W_{D2}) , as well as six point loads (P_A-P_F). This beam analysis allows for a simply supported beam with a left and right cantilever. Based on the input loads, the applicable hanger/post/trimmer is shown for each individual beam. The beam analysis also outputs the unfactored reactions, stresses and deflections at the bottom of each beam. See below for a sample beam. For 24F-V4 Glulam beams, the total deflection displayed accounts for a built in camber assuming a 3500' radius.

Sample Beam Calculation Comparison

Loading Diagram



H&S Calculation Package

Sample Beam Calculation - 3.5 x 11.875 1.5E SCL with KN 5C trimmer at left & KN 6 post at right

Sample Beam Calculation — Inputs

Lumber			Spans			Bracing	Support Condition			
Size	Grade	Type	Left Cant.	Main	Right Cant.	Braced?	Left	trimmer	KN	5C
3.5 x 11.875	1.5E	SCL	0.0	10.0	0.0	No	Right	post	KN	6

Loading

Distributed Loads											Point Loads (lbs)							
	Location		Tributary Lengths (ft)				Distributed Loads (plf)					Location	D	L	Lr	E	W	S
	Start	End	Wall	Roof	Floor	Alt Flr	D	L	Lr	S	P_A	2.0	300	480				
w_A	0.0	10.0			6.0		150	240	0	0	P_B	8.0	625	1000				
w_B	2.0	8.0	10.0	2.0			202	0	40	0	P_C							
w_C							0	0	0	0	P_D							
w_{D1}							0	0	0	0	P_E							
w_{D2}							0	0	0	0	P_F							

Results

Unfactored Reactions (lb)							Stresses						Deflection (in)			
	D	L	Lr	E	W	S		f	F	D/C	@ (ft)	L-C		Δ_{act}	Δ_{all}	@ (ft)
Left	1721	1784	120	0	0	0	shear	-143	285	0.50	9.9	16-9	ΔLL	-0.125	-0.250	5.1
Right	1916	2096	120	0	0	0	bending	1375	2115	0.65	5.3	16-9	ΔTL	-0.247	-0.333	5.1

Reactions match.

Enercalc

Wood Beam Design : Sample Beam Calculation

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

BEAM Size : **3.5x11.875, Parallam PSL, Fully Unbraced**
Using Allowable Stress Design with HSBeams Load Combinations, Major Axis Bending
Wood Species : H&S SCL Values
Wood Grade : 1.5E SCL

Fb - Tension	2,250.0 psi	Fc - Prll	1,600.0 psi	Fv	285.0 psi	Ebend- xx	1,500.0 ksi	Density	35.020 pcf
Fb - Compr	2,250.0 psi	Fc - Perp	750.0 psi	Ft	1,500.0 psi	Eminbend - xx	700.0 ksi		

Applied Loads

Unif Load: D = 0.0250, L = 0.040 k/ft, Trib= 6.0 ft
Unif Load: D = 0.0210, Lr = 0.020 k/ft, 2.0 to 8.0 ft, Trib= 2.0 ft
Unif Load: D = 0.0160 k/ft, 2.0 to 8.0 ft, Trib= 10.0 ft
Point: D = 0.30, L = 0.480 k @ 2.0 ft
Point: D = 0.6250, L = 1.0 k @ 8.0 ft

Shear and bending stresses match.

Governing load combinations match.
See following page for L-C #16-9 in H&S calculation package.

Design Summary

Max fb/Fb Ratio = **0.651** : 1
fb : Actual : 1,374.93 psi at 5.300 ft in Span # 1
Fb : Allowable : 2,113.35 psi
Load Comb : +D+L+H
Max fv/FvRatio = **0.508** : 1
fv : Actual : 144.79 psi at 10.000 ft in Span # 1
Fv : Allowable : 285.00 psi
Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.72	1.78	0.12				
Right Support	1.92	2.10	0.12				

Max Deflections

Transient Downward	0.126 in		Total Downward	0.249 in
Ratio	954		Ratio	482
	LC: +Lr+L			LC: +D+Lr+L+H
Transient Upward	0.000 in		Total Upward	0.000 in
Ratio	9999		Ratio	9999
	LC:			LC:

Load Combination Comparison

Load combinations used in H&S calculation package uses the same load combinations in Enercalc. The reactions listed in the H&S calculation package output are provided at service level, and all supports for the beam are designed using the appropriate CBC and ASCE 7 load combinations.

H&S Calculation Package

CBC Section 1605.3.1, Load Combinations						
Equations 12-2/14/16 are modified per ASCE7-10 12.4.2.3						
Equation	D	L	L_R/Snow	E	W	
16-9	1	1				
16-10	1		1			
16-11	1	0.75	0.75			
16-12-1	1				0.6	"4-10"
16-12-2	1.07007			0.7		"5-11"
16-13	1	0.75	0.75		0.45	"6-12"
16-14	1.07007	0.75	0.75	0.525		"7-13"
16-15	0.6				0.6	"8-14"
16-16	0.52993			0.7		"9-15"

ASCE 7-10 Section 12.4.3.2, ASD Combos w/ Overstrength					
	Combo	D	L	L_R/Snow	E*
1	5	1.07007			0.7
2	6b	1.05005	0.75	0.75	0.525
3	8	0.52993			0.7

* Overstrength Factor = 2.5 reduced by rho

Enercalc

Run	Load Combination	Cd	Dead Load Factor	0.2*SDS* Seismic Fac	Roof Live	Floor Live	Snow	Wind	Seismic Factor	Seismic Rho	Earth
Yes	+D+H	0.900	1.000								1.000
Yes	+D+L+H	1.000	1.000			1.000					1.000
Yes	+D+Lr+H	1.250	1.000		1.000						1.000
Yes	+D+S+H	1.150	1.000				1.000				1.000
Yes	+D+0.750Lr+0.750L+H	1.250	1.000		0.750	0.750					1.000
Yes	+D+0.750L+0.750S+H	1.150	1.000			0.750	0.750				1.000
Yes	+D+W+H	1.600	1.000					1.000			1.000
Yes	+1.210D+2.50E+H	1.600	1.210						2.500	1.000	1.000
Yes	+D+0.750Lr+0.750L+0.750W+H	1.600	1.000		0.750	0.750		0.750			1.000
Yes	+D+0.750L+0.750S+0.750W+H	1.600	1.000			0.750	0.750	0.750			1.000
Yes	+1.158D+0.750L+0.750S+1.875E+H	1.600	1.158			0.750	0.750		1.875	1.000	1.000
Yes	+0.60D+W+0.60H	1.600	0.600					1.000			0.600
Yes	+0.390D+2.50E+0.390H	1.600	0.390						2.500	1.000	0.390
Yes	+1.210D+1.920E+H	1.600	1.210						1.920	1.000	1.000
Yes	+1.158D+0.750L+0.750S+1.442E+H	1.600	1.158			0.750	0.750		1.442	1.000	1.000
Yes	+0.390D+1.920E+0.390H	1.600	0.390						1.920	1.000	0.390
Yes	+1.210D	0.900	1.210								

Beam Calculations

F-B header at right of Covered Patio - 3.5 x 11.875 1.5E SCL with (1) 2x trimmer at left & (1) 2x trimmer at right

F-B header at right of Covered Patio

Inputs

Lumber			Spans			Bracing	Support Condition			
Size	Grade	Type	Left Cant.	Main	Right Cant.	Braced?	Left	trimmer	KN	-
3.5 x 11.875	1.5E	SCL	0.0	11.3	0.0	No	Right	trimmer	KN	-

Loading

Distributed Loads

	Location		Tributary Lengths (ft)				Distributed Loads (plf)			
	Start	End	Wall	Roof	Floor	Alt Flr	D	L	Lr	S
w_A	0.0	11.3		12.8			268	0	255	0
w_B							0	0	0	0
w_C							0	0	0	0
w_{D1}							0	0	0	0
w_{D2}							0	0	0	0

Point Loads (lbs)

Location		D	L	Lr	E	W	S
P_A							
P_B							
P_C							
P_D							
P_E							
P_F							

Results

Unfactored Reactions (lb)							Stresses						Deflection (in)			
	D	L	Lr	E	W	S		f	F	D/C	@ (ft)	L-C		Δ_{act}	Δ_{all}	@ (ft)
Left	1506	0	1434	0	0	0	shear	106	356	0.30	0.0	16-10	ΔLL	-0.125	-0.281	5.6
Right	1506	0	1434	0	0	0	bending	1206	2479	0.49	5.6	16-10	ΔTL	-0.257	-0.375	5.6

F-B header at left of Covered Patio - 4 x 6 No. 2 Lumber with (1) 2x trimmer at left & (1) 2x trimmer at right

F-B header at left of Covered Patio

Inputs

Lumber			Spans			Bracing	Support Condition			
Size	Grade	Type	Left Cant.	Main	Right Cant.	Braced?	Left	trimmer	KN	-
4 x 6	No. 2	Lumber	0.0	5.3	0.0	No	Right	trimmer	KN	-

Loading

Distributed Loads

	Location		Tributary Lengths (ft)				Distributed Loads (plf)			
	Start	End	Wall	Roof	Floor	Alt Flr	D	L	Lr	S
w_A		5.3		12.8			270	0	257	0
w_B							0	0	0	0
w_C							0	0	0	0
w_{D1}							0	0	0	0
w_{D2}							0	0	0	0

Point Loads (lbs)

Location		D	L	Lr	E	W	S
P_A							
P_B							
P_C							
P_D							
P_E							
P_F							

Results

Unfactored Reactions (lb)							Stresses						Deflection (in)			
	D	L	Lr	E	W	S		f	F	D/C	@ (ft)	L-C		Δ_{act}	Δ_{all}	@ (ft)
Left	708	0	674	0	0	0	shear	108	225	0.48	0.0	16-10	ΔLL	-0.057	-0.131	2.6
Right	708	0	674	0	0	0	bending	1233	1453	0.85	2.6	16-10	ΔTL	-0.116	-0.175	2.6

L-R header at rear of Covered Patio - 4 x 14 No. 2 Lumber with (1) 2x trimmer at left & (1) 2x trimmer at right

L-R header at rear of Covered Patio

Inputs

Lumber			Spans			Bracing	Support Condition			
Size	Grade	Type	Left Cant.	Main	Right Cant.	Braced?	Left	trimmer	KN	-
4 x 14	No. 2	Lumber	0.0	15.0	0.0	No	Right	trimmer	KN	-

Loading

Distributed Loads

	Location		Tributary Lengths (ft)				Distributed Loads (plf)			
	Start	End	Wall	Roof	Floor	Alt Flr	D	L	Lr	S
w_A		15.0		2.0			42	0	40	0
w_B							0	0	0	0
w_C							0	0	0	0
w_{D1}							0	0	0	0
w_{D2}							0	0	0	0

Point Loads (lbs)

Location		D	L	Lr	E	W	S
P_A							
P_B							
P_C							
P_D							
P_E							
P_F							

Results

Unfactored Reactions (lb)							Stresses						Deflection (in)			
	D	L	Lr	E	W	S		f	F	D/C	@ (ft)	L-C		Δ_{act}	Δ_{all}	@ (ft)
Left	315	0	300	0	0	0	shear	20	225	0.09	0.0	16-10	ΔLL	-0.042	-0.375	7.5
Right	315	0	300	0	0	0	bending	270	1060	0.25	7.5	16-10	ΔTL	-0.086	-0.500	7.5

Beam Calculations

L-R header at front of Office - 6 x 12 No. 1 (P-T) Lumber with (1) 2x trimmer at left & (1) 2x trimmer at right

L-R header at front of Office

Inputs

Lumber			Spans			Bracing	Support Condition			
Size	Grade	Type	Left Cant.	Main	Right Cant.	Braced?	Left	trimmer	KN	-
6 x 12	No. 1 (P-T)	Lumber	0.0	13.6	0.0	No	Right	trimmer	KN	-

Loading

	Distributed Loads										Point Loads (lbs)							
	Location		Tributary Lengths (ft)				Distributed Loads (plf)				Location		D	L	Lr	E	W	S
	Start	End	Wall	Roof	Floor	Alt Flr	D	L	Lr	S	P_A							
w_A		13.6		2.0			42	0	40	0	P_B							
w_B							0	0	0	0	P_C							
w_C							0	0	0	0	P_D							
w_{D1}							0	0	0	0	P_E							
w_{D2}							0	0	0	0	P_F							

Results

	Unfactored Reactions (lb)						Stresses						Deflection (in)			
	D	L	Lr	E	W	S		f	F	D/C	@ (ft)	L-C		Δ_{act}	Δ_{all}	@ (ft)
Left	285	0	272	0	0	0	shear	13	213	0.06	0.0	16-10	ΔLL	-0.027	-0.340	6.8
Right	285	0	272	0	0	0	bending	187	1476	0.13	6.8	16-10	ΔTL	-0.056	-0.453	6.8

L-R header at front of Covered Patio - 6 x 14 No. 1 (P-T) Lumber with (1) 2x trimmer at left & (1) 2x trimmer at right

L-R header at front of Covered Patio

Inputs

Lumber			Spans			Bracing	Support Condition			
Size	Grade	Type	Left Cant.	Main	Right Cant.	Braced?	Left	trimmer	KN	-
6 x 14	No. 1 (P-T)	Lumber	0.0	14.3	0.0	No	Right	trimmer	KN	-

Loading

	Distributed Loads										Point Loads (lbs)							
	Location		Tributary Lengths (ft)				Distributed Loads (plf)				Location		D	L	Lr	E	W	S
	Start	End	Wall	Roof	Floor	Alt Flr	D	L	Lr	S	P_A							
w_A		14.3		2.0			42	0	40	0	P_B							
w_B							0	0	0	0	P_C							
w_C							0	0	0	0	P_D							
w_{D1}							0	0	0	0	P_E							
w_{D2}							0	0	0	0	P_F							

Results

	Unfactored Reactions (lb)						Stresses						Deflection (in)			
	D	L	Lr	E	W	S		f	F	D/C	@ (ft)	L-C		Δ_{act}	Δ_{all}	@ (ft)
Left	299	0	285	0	0	0	shear	12	213	0.06	0.0	16-10	ΔLL	-0.021	-0.356	7.1
Right	299	0	285	0	0	0	bending	150	1450	0.10	7.1	16-10	ΔTL	-0.042	-0.475	7.1

Hanger Capacities

		Key Note	Specification	Downward Capacity (Lb)			Uplift (Lb)
				Floor	Snow	Roof	
Face Mount Hangers	Standard	**21**	**HUSI.81/10**	5135	5295	5400	2675
		21C	**HHUS410**	5635	6380	6445	3565
		21D	**HUS412**	2635	2985	3220	3435
		21K	**HHUS5.50/10**	5635	6380	6880	3565
		21Z	**(2) A35 Clips (Rim-Rim)**	1180	1180	1180	1300
	Typical						
	Custom						

		Key Note	Specification	Downward Capacity (Lb)			Uplift (Lb)
				Floor	Snow	Roof	
Top Flange Hangers	Standard						
	Typical						
	Custom						

Notes

The Floor/Snow/Roof capacities listed are for a Cd factor of 1.0, 1.15, 1.25 respectively. If the max demand on a hanger is based on a Cd factor of 1.6, the roof capacity (Cd = 1.25) is used. The uplift value correlates to a Cd factor of 1.6

Post Capacities (Pounds)

4" Wall Width								
KN	Post Size	8'	9'	10'	12'	15'	20'	21'
5	Double 2x Stud Post	1701	2300	1880	893	N/A	N/A	N/A
-	Single 2x Trimmer	3281	3281	3281	3281	N/A	N/A	N/A
5C	Double 2x Trimmer	6563	6563	6563	6563	N/A	N/A	N/A
6	4X4 Post	6603	5268	4263	2928	N/A	N/A	N/A
6C	4X4 Trimmer	7656	7656	7656	7656	N/A	N/A	N/A
7	4X6 (W) Post	10280	8201	6641	4562	N/A	N/A	N/A
7C	4X6 (W) Trimmer	12031	12031	12031	12031	N/A	N/A	N/A
8	4X8 Post	13474	10784	8754	5989	N/A	N/A	N/A
8C	4X8 Trimmer	15859	15859	15859	15859	N/A	N/A	N/A
8E	4X10 Post	17062	13662	11105	7608	N/A	N/A	N/A
8G	4X12 Post	20672	16538	13466	9214	N/A	N/A	N/A

6" Wall Width								
KN	Post Size	8'	9'	10'	12'	15'	20'	21'
5A	Single 2x Stud Post	5156	4216	3086	1469	N/A	N/A	N/A
5	Double 2x Stud Post	10313	10313	9026	5709	2855	594	314
-	Single 2x Trimmer	5156	5156	5156	5156	5156	5156	5156
5C	Double 2x Trimmer	10313	10313	10313	10313	10313	10313	10313
7	4X6 (S) Post	12031	12031	12031	11242	7354	4062	3658
7C	4X6 (S) Trimmer	12031	12031	12031	12031	12031	12031	12031
9	6X6 Post	18906	18906	18906	16426	11314	6443	5838
9C	6X8 Post	25781	25781	25781	22358	15386	8745	7879
9D	6X10 Post	32656	32656	32656	27745	19385	11077	10032
9E	6X12 Post	39531	39531	39531	33523	23403	13409	12081

Notes

1) Loads are limited by the lesser of the buckling load and the bearing capacity, Cd = 1.0
2) Buckling loads are designed w/ 5 psf code minimum lateral load applied to the surface of the post only. Adjacent studs take the tributary loads of the wall. See exception under note 5.
3) Trimmer loads are designed for the adjacent king post to prevent buckling in the trimmer and therefore the loads are based on bearing capacity only.
4) 2x posts/studs are designed for the strong axis loading only. 2x4 posts/studs are calculated as stud grade at 8', DFL #2 at 9', and DFL#1 for 10' and 12'. 2x6 posts are calculated as DFL #2. All post heights 12' and lower are designed for both 2x4 and 2x6 walls. All post heights greter than 12' are based on 2x6 walls only.
5) 2x and Dbl. 2x studs have 16" lateral tributary area and were designed with the C&C wind load from a 30.5' tall bulding . They may double as posts and standard stud spacing.
6) King posts need to be checked w/ location specific tributary loads and not using this chart.
7) (W) signifies weak and (S) signifies strong axis loading.

Top Plate Capacity - 2018 NDS

Design Equations

Bending:

Allowable Bending Stress: $F_b' = C_D C_F C_{fu} F_b$

Applied Bending Stress: $f_b = M/S = [Pl/6]/S^*$

* Moment equation based on semi-rigid end fixity

Allowable Point Load on Top Plates: $P \leq 6F_b'S/l$

Shear:

Allowable Shear Stress: $F_v' = C_D F_v$

Applied Shear Stress: $f_v = 1.5V/A$

* Maximum shear occurs at "d" from support, eqn based on semi-continous plates

Allowable Point Load on Top Plates: $P \leq F_v'A/1.5V$

Properties & Layout

Top plate size:	2-2x4	2-2x6
Top plate species/grade:	DF #2	DF #2
Load Duration Factor, C_D:	1.25	1.25
Size Factor, C_F:	1.50	1.30
Flat Use Factor, C_{fu}:	1.1	1.15
Bending stress, F_b:	900 psi	900 psi
Bending stress, F_b':	1856 psi	1682 psi
Shear stress, F_v:	180 psi	180 psi
Shear stress, F_v':	225 psi	225 psi

Top Plate Bearing Capacity

Top Plate Size	Stud Specs	Top Plate Span	b	d	P(shear)	P(bending)	Pmax	Max Continuous Truss Span
2-2x4	16" oc 12" oc	14.5" 10.5"	3.5"	3"	2147# 2384#	2016# 2784#	**2016#** **2384#**	**40.0 ft** **47.0 ft**
2-2x6	16" oc 12" oc	14.5" 10.5"	5.5"	3.0"	3374# 3746#	2871# 3964#	**2871#** **3746#**	**57.0 ft** **> 60 ft**

Top Plate Lateral Capacity

Typical plate splice: (24) 16d nails, (12) nails each side of splice

Nailing Splice Capacity = 4531# (118# / nail) x (1.6 duration factor) x (24 nails)

TP Tension Capacity = 7245# (1.5" x 3.5") x (1.6 duration factor) x (1.5 size factor) x (575 psf F_t)

TP Compression Capacity = 4600#

Note: plates are braced along the strong axis at no more than 24" on-center by connection to the floor/roof framing members, and along the weak axis at no more than 16" on-center by the connections to the studs.

Design Top Plate Capacity = 4531#

Typical Ledger Sizes & Connections

16d Nail Capacity	118	lb (per NDS Ch.11)
1/4" x 3 1/2" SDS Capacity	340	lb (per ESR-2236)

Ledger Capacity & Max Supported Spans								
Ledger Specification	Ledger Size	Connection to Rim/Bm		Connection to Stud		Capacity (plf)	Max Supported Span (ft)	
		#/ft	Spec	#	Spec		Roof	Floor
Typical 2x6	2x6	4	16d	3	16d	265.5	12	9
KN 12	2x6	4	16d	4	16d	354	17	12
KN 12A	2x8	6	16d	6	16d	531	25	19
KN 12B	2x10	8	16d	4	$^{1}/_{4}$" x 3 $^{1}/_{2}$" SDS	944	46	34
KN 12F	1 3/4" wide	8	16d	5	$^{1}/_{4}$" x 3 $^{1}/_{2}$" SDS	944	--	34

FOUNDATION BY OTHERS

King Stud Calculations

King stud calculations include deflection checked with 42% of strength level wind for noted deflection limit and 60% of strength level wind for deflection limit outlined in section 1604.3.7.
The wind pressures noted already account for the 60% of stregth level wind (conversion from strength to ASD).
The calculations below support the king stud schedules shown on the plans

Principal Code Equations & General Data

$M' = F_b' S$ $\quad$ $\Delta = \dfrac{5w\ell^4}{384EI}$ $\quad$ Load Duration Factor (Wind): 1.6

Stud Calculations by Plate Height & Opening Width (2x4 Walls, L/360 Deflection Limit)

9 ' Plate Height										
Opening Width (ft)	Stud Data #	Stud Data Size & Grade	Wind Load (psf)	Moment (lb-in)	Demand fb (psi)	Capacity F'b (psi)	Deflection Δ (in) @ 42%	Deflection Δallow (in)	Deflection (1604.3.7) Δ (in) @ 60%	Deflection (1604.3.7) Δallow (in)
3	(1)	2x4 DF #2	15.20	3746	612	2160	0.174	0.300	0.248	0.617
5	(1)	2x4 DF #2	15.11	5442	889	2160	0.253	0.300	0.361	0.617
6	(2)	2x4 DF #2	14.87	6202	675	2160	0.192	0.300	0.274	0.617
8	(2)	2x4 DF #2	14.48	7684	836	2160	0.238	0.300	0.340	0.617
10	(3)	2x4 DF #2	14.16	9127	745	2160	0.212	0.300	0.303	0.617
12	(3)	2x4 DF #2	13.89	10536	860	2160	0.245	0.300	0.349	0.617
16	(4)	2x4 DF #2	13.46	13275	867	2160	0.247	0.300	0.352	0.617
6	(1)	4x4 DF #2	14.87	6202	608	2160	0.173	0.300	0.247	0.617
8	(1)	4x4 DF #2	14.48	7684	753	2160	0.214	0.300	0.306	0.617
10	(1)	4x4 DF #2	14.16	9127	894	2160	0.254	0.300	0.363	0.617
12	(1)	4x4 DF #2	13.89	10536	1032	2160	0.294	0.300	0.419	0.617
16	(1)	4x6 DF #2 (W)	13.46	13275	929	1872	0.264	0.300	0.377	0.617
10 ' Plate Height										
Opening Width (ft)	Stud Data #	Stud Data Size & Grade	Wind Load (psf)	Moment (lb-in)	Demand fb (psi)	Capacity F'b (psi)	Deflection Δ (in) @ 42%	Deflection Δallow (in)	Deflection (1604.3.7) Δ (in) @ 60%	Deflection (1604.3.7) Δallow (in)
3	(1)	2x4 DF #2	14.85	4550	743	2160	0.263	0.333	0.375	0.686
5	(2)	2x4 DF #2	14.85	6650	724	2160	0.256	0.333	0.365	0.686
6	(2)	2x4 DF #2	14.70	7619	829	2160	0.293	0.333	0.419	0.686
8	(3)	2x4 DF #2	14.30	9437	770	2160	0.272	0.333	0.389	0.686
10	(4)	2x4 DF #2	13.99	11205	732	2160	0.259	0.333	0.369	0.686
12	(4)	2x4 DF #2	13.72	12933	845	2160	0.299	0.333	0.426	0.686
16	(6)	2x4 DF #2	13.29	16288	760	2160	0.269	0.333	0.384	0.686
6	(1)	4x4 DF #2	14.70	7619	746	2160	0.264	0.333	0.377	0.686
8	(1)	4x6 DF #2 (W)	14.30	9437	660	1872	0.233	0.333	0.333	0.686
10	(1)	4x6 DF #2 (W)	13.99	11205	784	1872	0.277	0.333	0.396	0.686
12	(1)	4x8 DF #2 (W)	13.72	12933	724	1872	0.256	0.333	0.366	0.686
16	(1)	4x10 DF #2 (W)	13.29	16288	742	1728	0.262	0.333	0.375	0.686

King Stud Calculations

King stud calculations include deflection checked with 42% of strength level wind for noted deflection limit and 60% of strength level wind for deflection limit outlined in section 1604.3.7.
The wind pressures noted already account for the 60% of stregth level wind (conversion from strength to ASD).
The calculations below support the king stud schedules shown on the plans

Principal Code Equations & General Data

$M' = F_b' S$ $\Delta = \frac{5w\ell^4}{384EI}$ Load Duration Factor (Wind): 1.6

Stud Calculations by Plate Height & Opening Width (2x6 Walls, L/360 Deflection Limit)

Opening Width (ft)	Stud Data #	Stud Data Size & Grade	Wind Load (psf)	Moment (lb-in)	Demand fb (psi)	Capacity F'b (psi)	Deflection Δ (in) @ 42%	Deflection Δallow (in)	Deflection (1604.3.7) Δ (in) @ 60%	Deflection (1604.3.7) Δallow (in)
10 ' Plate Height										
3	(1)	2x6 DF Stud	14.85	4550	301	1120	0.077	0.333	0.110	0.686
5	(1)	2x6 DF Stud	14.85	6650	440	1120	0.113	0.333	0.161	0.686
6	(1)	2x6 DF Stud	14.70	7619	504	1120	0.129	0.333	0.185	0.686
8	(1)	2x6 DF Stud	14.30	9437	624	1120	0.160	0.333	0.229	0.686
10	(1)	2x6 DF Stud	13.99	11205	741	1120	0.190	0.333	0.272	0.686
12	(1)	2x6 DF Stud	13.72	12933	855	1120	0.220	0.333	0.314	0.686
16	(2)	2x6 DF Stud	13.29	16288	718	1120	0.185	0.333	0.264	0.686
6	(1)	4x6 DF #2 (S)	14.70	7619	302	1872	0.068	0.333	0.097	0.686
8	(1)	4x6 DF #2 (S)	14.30	9437	374	1872	0.084	0.333	0.120	0.686
10	(1)	4x6 DF #2 (S)	13.99	11205	445	1872	0.100	0.333	0.143	0.686
12	(1)	4x6 DF #2 (S)	13.72	12933	513	1872	0.115	0.333	0.165	0.686
16	(1)	4x6 DF #2 (S)	13.29	16288	646	1872	0.145	0.333	0.208	0.686
12 ' Plate Height										
Opening Width (ft)	#	Size & Grade	Wind Load (psf)	Moment (lb-in)	Demand fb (psi)	Capacity F'b (psi)	Δ (in) @ 42%	Δallow (in)	Δ (in) @ 60%	Δallow (in)
3	(1)	2x6 DF Stud	14.26	6352	420	1120	0.157	0.400	0.224	0.823
5	(1)	2x6 DF Stud	14.26	9284	614	1120	0.229	0.400	0.328	0.823
6	(1)	2x6 DF Stud	14.26	10750	711	1120	0.266	0.400	0.380	0.823
8	(1)	2x6 DF Stud	14.01	13440	889	1120	0.332	0.400	0.475	0.823
10	(2)	2x6 DF Stud	13.69	15951	703	1120	0.263	0.400	0.376	0.823
12	(2)	2x6 DF Stud	13.42	18402	811	1120	0.303	0.400	0.433	0.823
16	(2)	2x6 DF Stud	13.00	23159	1021	1120	0.382	0.400	0.545	0.823
6	(1)	4x6 DF #2 (S)	14.26	10750	426	1872	0.140	0.400	0.199	0.823
8	(1)	4x6 DF #2 (S)	14.01	13440	533	1872	0.174	0.400	0.249	0.823
10	(1)	4x6 DF #2 (S)	13.69	15951	633	1872	0.207	0.400	0.296	0.823
12	(1)	4x6 DF #2 (S)	13.42	18402	730	1872	0.239	0.400	0.341	0.823
16	(1)	4x6 DF #2 (S)	13.00	23159	919	1872	0.301	0.400	0.429	0.823

Stud Calculations Per 2018 NDS

The following stud calculations include deflection checked with 42% of strength level wind and a deflection limit of either L/240 or L/360 as outlined in the Stud Design Overview.

Load Combinations & Principal Code Equations:

Load Combo #1 $D + L + (L_r \text{ or } S \text{ or } R)$

Load Combo #2 $D + (0.6W \text{ or } 0.7E)$

Load Combo #3 $D + 0.75L + 0.75\,(0.6W \text{ or } 0.7E) + 0.75(L_r \text{ or } S \text{ or } R)$

$$F_{cE} = \frac{0.822\,E_{min}'}{(\ell_e/d)^2}$$

$$\Delta = \frac{5w\ell^4}{384EI} \qquad M' = F_b'\,S$$

Location-Specific Stud Calculations

Exterior 6x Wall

Stud and Loading Data										
Size & Grade	# of Studs	Height (ft)	Spacing (in)	Nailing to Shtg	Loads (Tributary Lengths, ft) Roof	Floor	Public	Wall	Lateral Loads (psf) Wind	Seismic
2x6 DF Stud	1	12	16		13				14.3	2.7

Calculations and Deflection Checks Using L/360 Deflection Limit										
Load Combination	Loads Axial	Loads Moment	Stresses F'c	fc	F'b	fb	Combined Stress	Deflection (in) Δ @ 42%	Δallow	Fire Wall Assembly
1	711	1440	515	86	1006	190	0.248	0.068	0.400	None
2	364	4106	540	44	1288	543	0.461	0.193	0.400	
3	624	3080	540	76	1288	407	0.380	0.145	0.400	

Exterior 4x Wall

Stud and Loading Data										
Size & Grade	# of Studs	Height (ft)	Spacing (in)	Nailing to Shtg	Loads (Tributary Lengths, ft) Roof	Floor	Public	Wall	Lateral Loads (psf) Wind	Seismic
2x4 DF #2	2	12	16		12.5				14.3	2.7

Calculations and Deflection Checks Using L/360 Deflection Limit										
Load Combination	Loads Axial	Loads Moment	Stresses F'c	fc	F'b	fb	Combined Stress	Deflection (in) Δ @ 42%	Δallow	Fire Wall Assembly
1	683	1440	273	65	1941	235	0.214	0.115	0.400	None
2	350	4106	275	33	2484	670	0.321	0.328	0.400	
3	600	3080	275	57	2484	503	0.297	0.246	0.400	

Interior 6x Wall

Stud and Loading Data										
Size & Grade	# of Studs	Height (ft)	Spacing (in)	Nailing to Shtg	Loads (Tributary Lengths, ft) Roof	Floor	Public	Wall	Lateral Loads (psf) Wind	Seismic
2x6 DF Stud	1	12	16		19.7375				5.0	1.3

Calculations and Deflection Checks Using L/360 Deflection Limit										
Load Combination	Loads Axial	Loads Moment	Stresses F'c	fc	F'b	fb	Combined Stress	Deflection (in) Δ @ 42%	Δallow	Fire Wall Assembly
1	1079	1440	515	131	1006	190	0.305	0.068	0.400	None
2	553	1440	540	67	1288	190	0.181	0.068	0.400	
3	947	1080	540	115	1288	143	0.182	0.051	0.400	

Interior 4x Wall

Stud and Loading Data

Size & Grade	# of Studs	Height (ft)	Spacing (in)	Nailing to Shtg	Loads (Tributary Lengths, ft) Roof	Floor	Public	Wall	Lateral Loads (psf) Wind	Seismic
2x4 DF #2	1	12	16		15				5.0	1.3

Calculations and Deflection Checks Using L/360 Deflection Limit

Load Combination	Loads Axial	Moment	Stresses F'c	fc	F'b	fb	Combined Stress	Deflection (in) Δ @ 42%	Δallow	Fire Wall Assembly
1	820	1440	273	156	1941	470	0.872	0.230	0.400	None
2	420	1440	275	80	2484	470	0.349	0.230	0.400	
3	720	1080	275	137	2484	353	0.526	0.173	0.400	

Exterior 4x Wall

Stud and Loading Data

Size & Grade	# of Studs	Height (ft)	Spacing (in)	Nailing to Shtg	Loads (Tributary Lengths, ft) Roof	Floor	Public	Wall	Lateral Loads (psf) Wind	Seismic
2x4 DF #2	1	10	16		12.5				14.9	2.7

Calculations and Deflection Checks Using L/360 Deflection Limit

Load Combination	Loads Axial	Moment	Stresses F'c	fc	F'b	fb	Combined Stress	Deflection (in) Δ @ 42%	Δallow	Fire Wall Assembly
1	683	1000	386	130	1941	327	0.361	0.109	0.333	None
2	350	2971	391	67	2484	970	0.496	0.323	0.333	
3	600	2228	391	114	2484	727	0.493	0.242	0.333	

Balloon Frame Walls

Stud and Loading Data

Size & Grade	# of Studs	Height (ft)	Spacing (in)	Nailing to Shtg	Loads (Tributary Lengths, ft) Roof	Floor	Public	Wall	Lateral Loads (psf) Wind	Seismic
2x6 DF Stud	1	15.67	12		6.5				13.5	2.7

Calculations and Deflection Checks Using L/360 Deflection Limit

Load Combination	Loads Axial	Moment	Stresses F'c	fc	F'b	fb	Combined Stress	Deflection (in) Δ @ 42%	Δallow	Fire Wall Assembly
1	267	1842	329	32	1006	244	0.276	0.151	0.522	None
2	137	4983	337	17	1288	659	0.539	0.409	0.522	
3	234	3737	337	28	1288	494	0.424	0.307	0.522	

Lateral Analysis Calculation Summary

Main Force-Resisting System (MFRS)

Resistance to lateral forces is provided by wood shearwalls and by manufactured shearwalls where required. Uplift forces at the wood shearwalls are resisted through metal strap holdowns at the third-to-second and second-to-first floor levels and metal holdowns at the foundation level.

Diaphragms, Chords, and Collectors

Lateral loads are transferred into the vertical elements of the MFRS using horizontal wood diaphragms, with collectors provided along each line of lateral force resistance. Note that diaphragms are modeled as flexible in accordance with ASCE 7-16 §12.3.1 Diaphragm forces are designed per ASCE 7-16 §12.10. The seismic collector load includes load from the shearwalls above plus the diaphragm load per ASCE Section 12.10. A 25% increase is applied per Sections 12.10.2.1 & 12.3.3.4.

Force Transfer at Opening

Shearwalls with openings have been designed using a rational analysis as permitted in the Force Transfer Around Openings method outlined in 2015 NDS SDPWS §4.3.5.2. Where the shearwall has sufficient capacity to transfer the loads around the opening without needing holdowns at the king studs, the Diekmann (SEAOC) method of analysis is used. Where the shearwall used does not have sufficient capacity, king stud holdowns are added and a simple static analysis is used (Drag-Strut). Note that traditional implementation of the drag-strut method has yielded underconservative horizontal strapping because engineers have typically not added the required holdown straps at the kings. Our implementation of the method includes the required holdown straps and is therefore an accurate method of analysis. In addition, when the drag-strut method is used the horizontal strap forces have been amplified by a factor of 2.0 to be more in alignment with the APA "drag-strut" method. The seismic capacity of the shearwall is adjusted according to the requirements of NDS SDPWS § 4.3.4 using the worst-case height-to-width ratio of the overall shearwall and the smaller wall piers within the wall. Also, as shown in the corresponding details on the framing plans (eg. detail 650 & 658) the shearwall sheathing is edge-nailed to the king studs for the full height of the shearwall.
See the example calculation on the following page, which uses the Diekmann method.

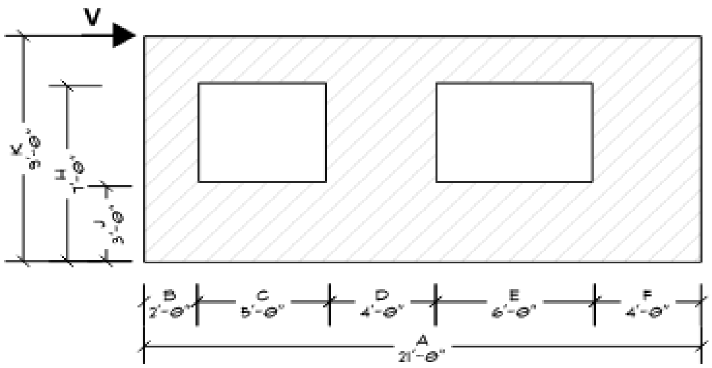
Perforated Shearwalls

Shearwalls with openings that are not designed to transer forces around the openings are designed as perforated shearwalls in accordange with 2015 NDS SDPWS §4.3.5.3. The seismic capacity of the piers are adjusted according to the requirements of NDS SDPWS § 4.3.4. Also, as shown in the corresponding details on the framing plans (eg. detail 655) the shearwall sheathing is edge-nailed to the king studs for the full height of the shearwall.

Force Transfer Around Opening Sample Calculation

Shear Wall w/ Force Transfer Around 2 Openings

Shear Wall Information



Shear Wall
Type = 3*
*3/8" SHEATHING W/ 8d COMMON NAILS AT 3" OC EDGE AND 12" OC FIELD
Capacity = 490plf (Seismic)
H:W = (7' - 3' + 0.5') / 2' = 2.25

General Notes:
- Diekmann method shown
- This line of lateral force resistance has one (1) shear wall
- Seismic Force, V = 4724lb
- For simplicity, dead and wind loads are not considered in sample calculation

Shear Wall Design

H&S Calculation Package Design

Geometry		SW 1	
Total Length (A)		21.00 ft	
To 1st Opening (B)		2.00 ft	
1st Opening Width (C)		5.00 ft	
1st to 2nd Openings (D)		4.00 ft	
2nd Opening Width (E)		6.00 ft	
2nd to 3rd Opening (F)			
3rd Opening Width (G)			
Net Length		10.00 ft	
Max Header Height (H)		7.00 ft	
Min Sill Height (J)		3.00 ft	
Plate Height (K)		9.00 ft	
H:W		2.25	
Loads		Wind	Seismic
Trib Length Roof			
Trib Length Floor			
Total Shear Load		1768 lb	4724 lb
Add'l Uplift	Left		
	King		
	Right		
SW Info	Type	3	
	Capacity	600 plf	490 plf
Analysis Method Used		Diekmann	
Shears	Top/Bottom	168 plf	450 plf
	Piers	177 plf	472 plf
	Corners	-8 plf	-22 plf
Horiz. Strap Load		741 lb	1979 lb
Strap Specification		(2) CS16	
Total Uplift	Left	776 lb	2074 lb
	King	0 lb	0 lb
	Right	776 lb	2074 lb
Holdowns	Left	2	
	King	NONE	
	Right	2	

Sample Calculation

Determine Analysis Method

Check if there is additional uplift at king studs
No additional uplift
King stud holdowns are not required
Check sill height
Sill Height of 3' is greater than 1'
Wood structural panels exist both above and below the openings
Check shear load against shearwall capacity
V = (4724lb x 9' / 21')/(9' - (7' - 3') - 0.5')
V = 450plf
450plf < 490plf
Shear wall has sufficient capacity to transfer the loads around the opening without needing holdowns at the king studs

Use Diekmann Method

Determine Wall Shears

@ Top/Bottom of Opening
$V_{top/bot}$ = (4724lb x 9' / 21') / (9' - (7' - 3') - 0.5')
$V_{top/bot}$ = 450plf

@ Piers
V_{pier} = 4724lb / 10'
V_{pier} = 472plf

@ Corners
$V_{corners}$ = 472plf - 450plf x (21' - 10') / 10'
$V_{corners}$ = -22plf

Determine Horizontal Strap Load

$V_{horiz\ strap\ load}$ = 4' / 10' x 450plf x (21' - 10')
$V_{horiz\ strap\ load}$ = 1979lb

Use (2) CS16 straps (3410lb capacity)

Determine Uplift Force

Uplift = 4724lb x 9' / (21' - 0.5')
Uplift = 2074lb

Use Type 2 holdown straps: (2) CS16 (3410lb capacity)

Lateral Analysis Calculation: P1 - 1st Floor; Rear; Left to Right

Wall Location

Level	1st Floor
Location of Line	Rear
Direction of Load	Left to Right

Building Data

Plate Height Above	0.00 ft
Plate Height Below	12.00 ft
Rho (Left to Right)	1.0

Diaphragm Geometry

Location	To Rear	To Front
Diaphragm Type	Cantilever	Simple
Diaphragm Width	8 ft	43 ft
Diaphragm Depth	24 ft	32 ft
Structure Above	Pitched Roof	Pitched Roof
Avg Height Above	12.50 ft	10.42 ft

Additional Loads

Source		
% of Total		
Wind	0 lb	0 lb
Seismic	0 lb	0 lb
% To Rear		
% To Front		
% Direct	100%	100%

Wind & Seismic Loads

Wind Loading											
Location	Loading Condition	Wall (including gable)			Pitched Roof			Parapet		Add'l Load	**Total Wind**
		Avg Area	Add'l Area	Pressure	Avg Area	Add'l Area	Pressure	Area	Pressure		
To Rear	Two-Sided	48 sf	0 sf	11.5 psf	100 sf	0 sf	8.8 psf	0 sf	23.8 psf	0 lb	1432 lb
To Front	Two-Sided	128 sf	0 sf	11.5 psf	221 sf	0 sf	9.0 psf	45 sf	22.4 psf	0 lb	4474 lb
Total		176 sf	0 sf		321 sf	0 sf		45 sf		0 lb	**5906 lb**

Seismic Loading							
Location	Tributary Area	Add'l Area	Story Force	Add'l Load	**Total Seismic**	125% Seismic	Seismic Collector
To Rear	192 sf	0 sf	374 lb	0 lb	374 lb	468 lb	608 lb
To Front	680 sf	0 sf	1325 lb	0 lb	1325 lb	1656 lb	2153 lb
Total	872 sf	0 sf	1699 lb	0 lb	**1699 lb**	2124 lb	2761 lb

Shear Wall Calculations

Summary of Inputs (See Below)

# of Walls	3
Total Net Length	17.42 ft
Adjusted Length	15.17 ft

Worst Case Design Values

Wind Shear	389 plf
Seismic Shear	112 plf

Shearwall Summary

Type Required	4
Override	N/A
SW TYPE USED	**4**

Shear Wall & Holdown Calculations												
Net Length / Wall Height	H:W Ratio	Total Load (W/E)	Roof Trib / Floor Trib	Additional Uplifts			Total Uplifts		Anchorage Spec	Holdown Spec	Add'l Reinf KN
				Wind	Seismic	Location	Wind	Seismic			
8.42 ft. / 9.00 ft.	1.07	FTAO shearwall w/ CS16 horiz strap(s) @ openings and holdowns: 19 @ Left; NONE @ Kings; 19 @ Right. Calculations follow.									
4.50 ft. / 12.00 ft.	2.67	1314 lb	2.0 ft	1427 lb	453 lb	0.0 ft	5090 lb	1315 lb	Interior	19	
		378 lb		0 lb	0 lb		3663 lb	863 lb	Interior	17	
4.50 ft. / 12.00 ft.	2.67	1314.0 lb	2.0 ft	0 lb	0 lb		3663 lb	863 lb	Interior	17	
		378.1 lb		4004 lb	1125 lb	4.5 ft	6751 lb	2636 lb	Corner	19	

Vertical Lateral Elements Above Plate

P1 - 1st Floor; Rear; Left to Right

Shear Panels in Roof

Length
Height
Trib Roof
Shear (W)
Shear (E)
Uplift (W)
Uplift (E)

Truss Overturning

Truss Overturning Calculations

Length	Drag Load (lbs) Wind	Drag Load (lbs) Seismic	Pitch	Config	Trib Roof (ft)	Wind Uplift (#)	Seismic Uplift (#)
24.5	3071	1436	12	Common	2	1427 lb	453 lb
8.5	2835	1325	12	Mono Gable	2	2451 lb	935 lb
			12				

Shearwall Deflection Calculations

P1 - 1st Floor; Rear; Left to Right

$$\delta_{ex} = \frac{8vh^3}{EAb} + \frac{vh}{1000\,G_a} + \Delta_a \frac{h}{b}$$

Shearwall Construction

Typical Wall Width	2x6
Sheathing Type	Ply

Shearwall Deflection

Deflection, δ_{ex}	0.20 in
Deflection, δ_x	0.80 in
Allowable Drift	2.88 in

Diaphragm Calculations

Diaphragm Shear

	Diaph Length	Floor/ Roof	Diaph Case	Add'l Load (W)	Add'l Load (E)	Wind Shear	Seismic Shear	Blkg and Nailing
To Rear	24 ft	Roof	3	0 lb	0 lb	60 plf	25 plf	Unblocked
To Front	32 ft	Roof	3	0 lb	0 lb	140 plf	67 plf	Unblocked
Total	32 ft					Material		Ply

Summary of Inputs

Location	To Rear	To Front
Type	Cantilever	Simple
Width	8.0 ft.	42.5 ft.
Depth	24.0 ft.	32.0 ft.

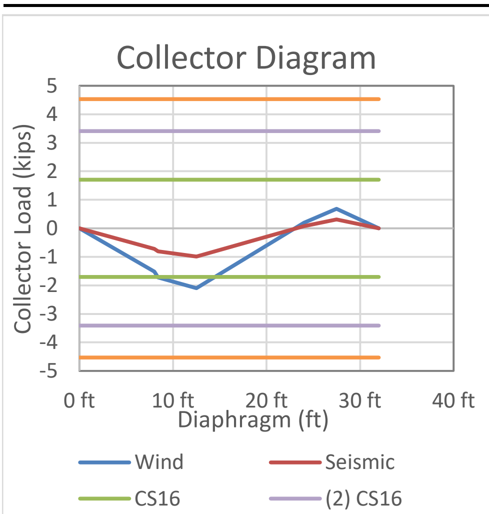
Chord Forces

Location	To Rear	To Front
w_{wind}	358 lb	211 lb
$w_{seismic}$	152 lb	101 lb
T/C Load	477 lb	1486 lb

Diaphragm Deflections

Location	To Rear	To Front
Top Plates	(2) 2x4	(2) 2x4
Deflection, δ_{ex} (in)	0.05 in	0.67 in
Deflection, δ_x (in)	0.21 in	2.68 in

Collector Calculations

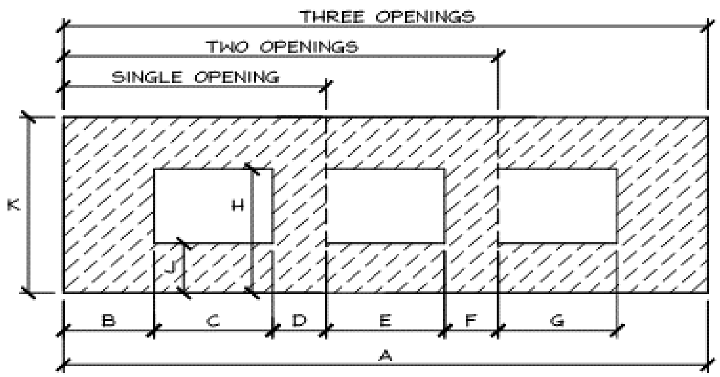


Shear Wall & Diaphragm Data

Shear Data (W/E)	At Level		0		0	
Design SW Shear	389 plf	182 plf				
Diaph: To Rear	60 plf	25 plf				
To Front	140 plf	67 plf				
Wall	Start	End	Start	End	Start	End
1	0.0 ft	8.4 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
2	8.0 ft	12.5 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
3	27.5 ft	32.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
4	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
5	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
6	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
7	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
8	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
Diaph: To Rear	0.0 ft	24.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
To Front	0.0 ft	32.0 ft				

Shear Walls w/ Openings

P1 - 1st Floor; Rear; Left to Right



Shearwalls w/ Force Transfer Around Openings (FTAO)								
Geometry	**SW 1**		**SW 2**		**SW3**		**SW4**	
Total Length (A)	8.42 ft							
To 1st Opening (B)	3.25 ft							
1st Opening Width (C)	2.00 ft							
1st to 2nd Openings (D)								
2nd Opening Width (E)								
2nd to 3rd Opening (F)								
3rd Opening Width (G)								
Net Length	6.42 ft							
Max Header Height (H)	8.00 ft							
Min Sill Height (J)	3.00 ft							
Plate Height (K)	12.00 ft							
H:W	1.74							
Loads	Wind	Seismic	Wind	Seismic	Wind	Seismic	Wind	Seismic
Trib Length Roof	2.00 ft							
Trib Length Floor								
Total Shear Load	3278 lb	943 lb						
Add'l Uplift: Left	2451 lb	935 lb						
King								
Right	2451 lb	935 lb						
SW Info Type	4							
Capacity	750 plf	640 plf						
Analysis Method Used	Diekmann							
Shears Top/Bottom	719 plf	207 plf						
Piers	511 plf	147 plf						
Corners	287 plf	82 plf						
Horiz. Strap Load	728 lb	209 lb						
Strap Specification	**CS16**							
Total Uplift Left	6895 lb	1857 lb						
King	0 lb	0 lb						
Right	6895 lb	1857 lb						
Holdowns Left	19							
King	NONE							
Right	19							

Lateral Analysis Calculation: P2 - 1st Floor; Front; Left to Right

Wall Location

Level	1st Floor
Location of Line	Front
Direction of Load	Left to Right

Building Data

Plate Height Above	0.00 ft
Plate Height Below	12.00 ft
Rho (Left to Right)	1.0

Diaphragm Geometry

Location	To Rear	To Front
Diaphragm Type	Simple	Simple
Diaphragm Width	43 ft	0 ft
Diaphragm Depth	32 ft	32 ft
Structure Above	Pitched Roof	Pitched Roof
Avg Height Above	12.50 ft	12.50 ft

Additional Loads

Source		
% of Total		
Wind	0 lb	0 lb
Seismic	0 lb	0 lb
% To Rear		
% To Front		
% Direct	100%	100%

Wind & Seismic Loads

Wind Loading

Location	Loading Condition	Wall (including gable) Avg Area	Add'l Area	Pressure	Pitched Roof Avg Area	Add'l Area	Pressure	Parapet Area	Pressure	Add'l Load	**Total Wind**
To Rear	Two-Sided	128 sf	0 sf	11.5 psf	266 sf	-44 sf	8.8 psf	44 sf	23.8 psf	0 lb	4461 lb
To Front	Two-Sided	0 sf	0 sf	11.5 psf	0 sf	0 sf	8.8 psf	0 sf	23.8 psf	0 lb	0 lb
Total		128 sf	0 sf		266 sf	-44 sf		44 sf		0 lb	**4461 lb**

Seismic Loading

Location	Tributary Area	Add'l Area	Story Force	Add'l Load	**Total Seismic**	125% Seismic	Seismic Collector
To Rear	680 sf	77 sf	1476 lb	0 lb	1476 lb	1845 lb	2398 lb
To Front	0 sf	0 sf	0 lb	0 lb	0 lb	0 lb	0 lb
Total	680 sf	77 sf	1476 lb	0 lb	**1476 lb**	1845 lb	2398 lb

Shear Wall Calculations

Summary of Inputs (See Below)

# of Walls	2
Total Net Length	10.00 ft
Adjusted Length	8.33 ft

Worst Case Design Values

Wind Shear	446 plf
Seismic Shear	148 plf

Shearwall Summary

Type Required	4
Override	N/A
SW TYPE USED	**4**

Shear Wall & Holdown Calculations

Net Length / Wall Height	H:W Ratio	Total Load (W/E)	Roof Trib / Floor Trib	Additional Uplifts Wind	Seismic	Location	Total Uplifts Wind	Seismic	Anchorage Spec	Holdown Spec	Add'l Reinf KN
5.00 ft.	2.40	2231 lb	2.0 ft	682 lb	0 lb	0.0 ft	6320 lb	1666 lb	Corner	19	
12.00 ft.		738 lb		0 lb	0 lb		5638 lb	1666 lb	Typical	19	
5.00 ft.	2.40	2231 lb	2.0 ft	0 lb	0 lb		5638 lb	1666 lb	Typical	19	
12.00 ft.		738 lb		682 lb	0 lb	5.0 ft	6320 lb	1666 lb	Corner	19	

Vertical Lateral Elements Above Plate

P2 - 1st Floor; Front; Left to Right

Shear Panels in Roof

Length
Height
Trib Roof
Shear (W)
Shear (E)
Uplift (W)
Uplift (E)

Truss Overturning

Truss Overturning Calculations							
	Drag Load (lbs)				Trib	Wind	Seismic
Length	Wind	Seismic	Pitch	Config	Roof (ft)	Uplift (#)	Uplift (#)
24.5	4461	2398	12	Gable	2	682 lb	0 lb
			12				
			12				

Shearwall Deflection Calculations

P2 - 1st Floor; Front; Left to Right

$$\delta_{ex} = \frac{8vh^3}{EAb} + \frac{vh}{1000\,G_a} + \Delta_a \frac{h}{b}$$

Shearwall Construction

Typical Wall Width	2x6
Sheathing Type	Ply

Shearwall Deflection

Deflection, δ_{ex}	0.21 in
Deflection, δ_x	0.85 in
Allowable Drift	2.88 in

Diaphragm Calculations

Diaphragm Shear								
	Diaph Length	Floor/ Roof	Diaph Case	Add'l Load (W)	Add'l Load (E)	Wind Shear	Seismic Shear	Blkg and Nailing
To Rear	32 ft	Roof	3	0 lb	0 lb	139 plf	75 plf	Unblocked
To Front	32 ft	Roof	3	0 lb	0 lb	0 plf	0 plf	Unblocked
Total	32 ft					Material		Ply

Summary of Inputs

Location	To Rear	To Front
Type	Simple	Simple
Width	42.5 ft.	0.0 ft.
Depth	32.0 ft.	32.0 ft.

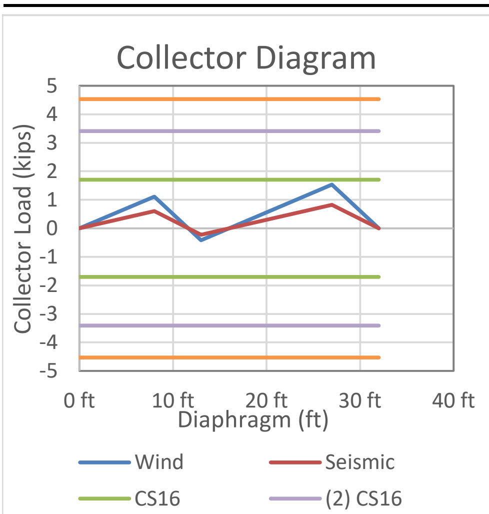
Chord Forces

Location	To Rear	To Front
w_{wind}	210 lb	
$w_{seismic}$	113 lb	
T/C Load	1481 lb	0 lb

Diaphragm Deflections

Location	To Rear	To Front
Top Plates	(2) 2x4	(2) 2x4
Deflection, δ_{ex} (in)	0.67 in	
Deflection, δ_x (in)	2.69 in	

Collector Calculations



Shear Wall & Diaphragm Data							
Shear Data (W/E)		At Level		0		0	
Design SW Shear		446 plf	240 plf				
Diaph:	To Rear	139 plf	75 plf				
	To Front	0 plf	0 plf				
Wall		Start	End	Start	End	Start	End
1		8.0 ft	13.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
2		27.0 ft	32.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
3		0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
4		0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
5		0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
6		0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
7		0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
8		0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
Diaph:	To Rear	0.0 ft	32.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
	To Front	0.0 ft	32.0 ft				

Lateral Analysis Calculation: P3 - 1st Floor; Left; Front to Back

Wall Location

Level	1st Floor
Location of Line	Left
Direction of Load	Front to Back

Building Data

Plate Height Above	0.00 ft
Plate Height Below	12.00 ft
Rho (Front to Back)	1.0

Diaphragm Geometry

Location	To Left	To Right
Diaphragm Type	Cantilever	Simple
Diaphragm Width	5 ft	32 ft
Diaphragm Depth	9 ft	51 ft
Structure Above	Parapet	Parapet
Avg Height Above	3.58 ft	3.58 ft

Additional Loads

Source		
% of Total		
Wind	0 lb	0 lb
Seismic	0 lb	0 lb
% To Left		
% To Right		
% Direct	100%	100%

Wind & Seismic Loads

Wind Loading

Location	Loading Condition	Wall (including gable) Avg Area	Add'l Area	Pressure	Pitched Roof Avg Area	Add'l Area	Pressure	Parapet Area	Pressure	Add'l Load	**Total Wind**
To Left	Two-Sided	30 sf	0 sf	11.5 psf	0 sf	0 sf	9.0 psf	0 sf	21.7 psf	0 lb	344 lb
To Right	Two-Sided	96 sf	36 sf	11.5 psf	0 sf	0 sf	9.0 psf	-24 sf	21.7 psf	0 lb	992 lb
Total		126 sf	36 sf		0 sf	0 sf		-24 sf		0 lb	**1336 lb**

Seismic Loading

Location	Tributary Area	Add'l Area	Story Force	Add'l Load	**Total Seismic**	125% Seismic	Seismic Collector
To Left	45 sf	0 sf	88 lb	0 lb	88 lb	110 lb	143 lb
To Right	808 sf	-117 sf	1347 lb	0 lb	1347 lb	1683 lb	2188 lb
Total	853 sf	-117 sf	1434 lb	0 lb	**1434 lb**	1793 lb	2331 lb

Shear Wall Calculations

Summary of Inputs (See Below)

# of Walls	1
Total Net Length	10.00 ft
Adjusted Length	10.00 ft

Worst Case Design Values

Wind Shear	134 plf
Seismic Shear	143 plf

Shearwall Summary

Type Required	2
Override	N/A
SW TYPE USED	**2**

Shear Wall & Holdown Calculations

Net Length / Wall Height	H:W Ratio	Total Load (W/E)	Roof Trib / Floor Trib	Additional Uplifts Wind	Seismic	Location	Total Uplifts Wind	Seismic	Anchorage Spec	Holdown Spec	Add'l Reinf KN
10.00 ft.	0.90	1336 lb	16.8 ft	0 lb	0 lb		464 lb	81 lb	Typical	NONE	
9.00 ft.		1434 lb		0 lb	0 lb		464 lb	81 lb	Corner	19	

Shearwall Deflection Calculations

P3 - 1st Floor; Left; Front to Back

$$\delta_{ex} = \frac{8vh^3}{EAb} + \frac{vh}{1000\,G_a} + \Delta_a \frac{h}{b}$$

Shearwall Construction

Typical Wall Width	2x6
Sheathing Type	Ply

Shearwall Deflection

Deflection, δ_{ex}	0.16 in
Deflection, δ_x	0.64 in
Allowable Drift	2.88 in

Diaphragm Calculations

Diaphragm Shear								
	Diaph Length	Floor/ Roof	Diaph Case	Add'l Load (W)	Add'l Load (E)	Wind Shear	Seismic Shear	Blkg and Nailing
To Left	9 ft	Roof	3	0 lb	0 lb	38 plf	16 plf	Unblocked
To Right	51 ft	Roof	3	0 lb	0 lb	20 plf	43 plf	Unblocked
Total	51 ft					Material		Ply

Summary of Inputs

Location	To Left	To Right
Type	Cantilever	Simple
Width	5.0 ft.	32.0 ft.
Depth	9.0 ft.	50.5 ft.

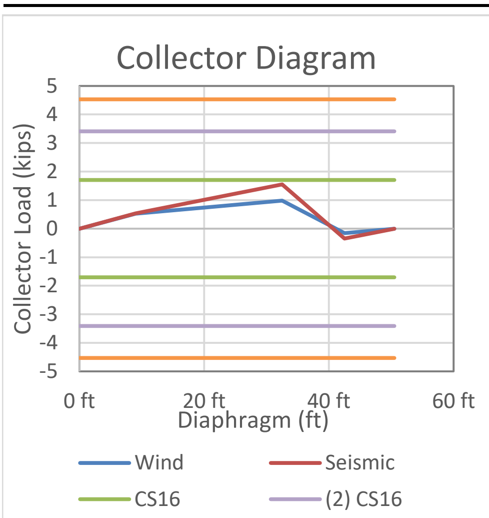
Chord Forces

Location	To Left	To Right
w_{wind}	138 lb	62 lb
$w_{seismic}$	57 lb	137 lb
T/C Load	191 lb	347 lb

Diaphragm Deflections

Location	To Left	To Right
Top Plates	(2) 2x4	(2) 2x4
Deflection, δ_{ex} (in)	0.03 in	0.13 in
Deflection, δ_x (in)	0.13 in	0.52 in

Collector Calculations



Shear Wall & Diaphragm Data						
Shear Data (W/E)	At Level		0		0	
Design SW Shear	134 plf	233 plf				
Diaph: To Left	38 plf	16 plf				
To Right	20 plf	43 plf				
Wall	Start	End	Start	End	Start	End
1	32.5 ft	42.5 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
2	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
3	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
4	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
5	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
6	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
7	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
8	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
Diaph: To Left	0.0 ft	9.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
To Right	0.0 ft	50.5 ft				

Lateral Analysis Calculation: P4 - 1st Floor; Right; Front to Back

Wall Location

Level	1st Floor
Location of Line	Right
Direction of Load	Front to Back

Building Data

Plate Height Above	0.00 ft
Plate Height Below	12.00 ft
Rho (Front to Back)	1.0

Diaphragm Geometry

Location	To Left	To Right
Diaphragm Type	Simple	Simple
Diaphragm Width	32 ft	0 ft
Diaphragm Depth	51 ft	51 ft
Structure Above	Gable Roof	Pitched Roof
Avg Height Above	7.50 ft	4.25 ft

Additional Loads

Source		
% of Total		
Wind	0 lb	0 lb
Seismic	0 lb	0 lb
% To Left		
% To Right		
% Direct	100%	100%

Wind & Seismic Loads

Wind Loading											
Location	Loading Condition	Wall (including gable)			Pitched Roof			Parapet		Add'l Load	**Total Wind**
		Avg Area	Add'l Area	Pressure	Avg Area	Add'l Area	Pressure	Area	Pressure		
To Left	Two-Sided	216 sf	0 sf	11.5 psf	0 sf	0 sf	9.0 psf	0 sf	21.9 psf	0 lb	2477 lb
To Right	Two-Sided	0 sf	0 sf	11.5 psf	0 sf	0 sf	9.0 psf	0 sf	21.7 psf	0 lb	0 lb
Total		216 sf	0 sf		0 sf	0 sf		0 sf		0 lb	**2477 lb**

Seismic Loading							
Location	Tributary Area	Add'l Area	Story Force	Add'l Load	**Total Seismic**	125% Seismic	Seismic Collector
To Left	808 sf	0 sf	1575 lb	0 lb	1575 lb	1968 lb	2559 lb
To Right	0 sf	0 sf	0 lb	0 lb	0 lb	0 lb	0 lb
Total	808 sf	0 sf	1575 lb	0 lb	**1575 lb**	1968 lb	2559 lb

Shear Wall Calculations

Summary of Inputs (See Below)

# of Walls	2
Total Net Length	8.00 ft
Adjusted Length	5.33 ft

Worst Case Design Values

Wind Shear	310 plf
Seismic Shear	197 plf

Shearwall Summary

Type Required	4
Override	N/A
SW TYPE USED	**4**

Shear Wall & Holdown Calculations												
Net Length / Wall Height	H:W Ratio	Total Load (W/E)	Roof Trib / Floor Trib	Additional Uplifts			Total Uplifts		Anchorage Spec	Holdown Spec	Add'l Reinf KN
				Wind	Seismic	Location	Wind	Seismic			
4.00 ft.	3.00	1239 lb	12.0 ft	0 lb	0 lb		3911 lb	2242 lb	Typical	17	
12.00 ft.		787 lb		0 lb	0 lb		3911 lb	2242 lb	Typical	17	
4.00 ft.	3.00	1239 lb	12.0 ft	0 lb	0 lb		3911 lb	2242 lb	Typical	17	
12.00 ft.		787 lb		0 lb	0 lb		3911 lb	2242 lb	Corner	19	

Shearwall Deflection Calculations

P4 - 1st Floor; Right; Front to Back

$$\delta_{ex} = \frac{8vh^3}{EAb} + \frac{vh}{1000\,G_a} + \Delta_a \frac{h}{b}$$

Shearwall Construction

Typical Wall Width	2x6
Sheathing Type	Ply

Shearwall Deflection

Deflection, δ_{ex}	0.43 in
Deflection, δ_x	1.73 in
Allowable Drift	2.88 in

Diaphragm Calculations

Diaphragm Shear								
	Diaph Length	Floor/ Roof	Diaph Case	Add'l Load (W)	Add'l Load (E)	Wind Shear	Seismic Shear	Blkg and Nailing
To Left	51 ft	Roof	3	0 lb	0 lb	49 plf	51 plf	Unblocked
To Right	51 ft	Roof	3	0 lb	0 lb	0 plf	0 plf	Unblocked
Total	51 ft					Material		Ply

Summary of Inputs

Location	To Left	To Right
Type	Simple	Simple
Width	32.0 ft.	0.0 ft.
Depth	50.5 ft.	51.0 ft.

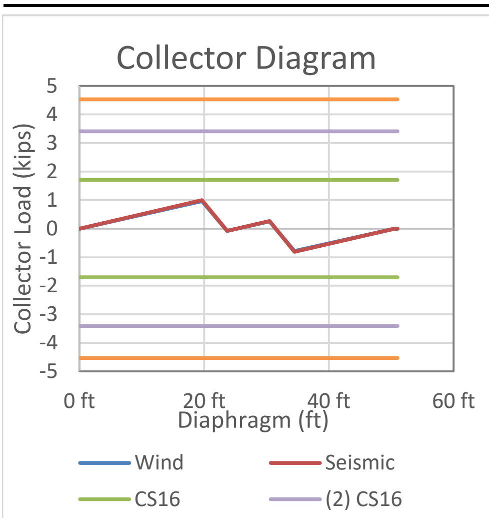
Chord Forces

Location	To Left	To Right
w_{wind}	155 lb	
$w_{seismic}$	160 lb	
T/C Load	405 lb	0 lb

Diaphragm Deflections

Location	To Left	To Right
Top Plates	(2) 2x4	(2) 2x4
Deflection, δ_{ex} (in)	0.17 in	
Deflection, δ_x (in)	0.68 in	

Collector Calculations



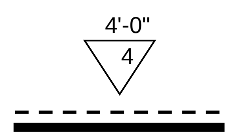
Shear Wall & Diaphragm Data						
Shear Data (W/E)	At Level		0		0	
Design SW Shear	310 plf	320 plf				
Diaph: To Left	49 plf	51 plf				
To Right	0 plf	0 plf				
Wall	Start	End	Start	End	Start	End
1	19.7 ft	23.7 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
2	30.5 ft	34.5 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
3	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
4	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
5	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
6	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
7	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
8	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
Diaph: To Left	0.0 ft	50.5 ft	0.0 ft	0.0 ft	0.0 ft	0.0 ft
To Right	0.0 ft	51.0 ft				

Shearwall Table

Shearwall Capacities			
Type	Wind	Seismic	Description of Wall Construction
4	750	640	3/8" APA RATED SHEATHING ONE FACE WITH 8d COMMON NAILS AT 2" O.C. EDGE AND 12" O.C. FIELD. INSTALL MINIMUM 3X NOMINAL FRAMING MEMBERS AT ADJOINING PANEL EDGES WITH STAGGERED NAILING. MAX. HOLDOWNS AS SPECIFIED IN CALCULATIONS.
2	350	350	3/8" APA RATED SHEATHING ONE FACE WITH 8d COMMON NAILS AT 4" O.C. EDGE AND 12" O.C. FIELD. HOLDOWNS AS SPECIFIED IN CALCULATIONS.

NOTES:

1. Shearwalls are designated on the plans by a triangle symbol surrounding the shearwall type.
2. Shearwall length is indicated above the shearwall callout and is shown graphically with shading & a dashed line.
3. See anchor bolt calculations for required anchor spacing.



Holdown Table

Upper Level Holdown Capacities							
Type	To Post / Beam		To Rim		To Header		Description of Holdown Note: Holdowns to rim are designated on the plans and within the calculations w/ a "C" after the holdown number type. Holdowns to headers are designated w/ an "A" after the holdown number type. The holdown schedule on the plans references details which provide additional specifications
	Wind	Seismic	Wind	Seismic	Wind	Seismic	
7	9215	9215	N/A	N/A	N/A	N/A	CMST12 STRAP, CLEAR SPAN VARIES.
6	6475	6475	N/A	N/A	N/A	N/A	CMST14 STRAP, CLEAR SPAN VARIES.
3	4690	4690	N/A	N/A	N/A	N/A	CMSTC16 STRAP, CLEAR SPAN VARIES.
2	3410	3410	3410	3410	3410	3410	(2) CS16 STRAPS, CLEAR SPAN VARIES.
1	1705	1705	1705	1705	1705	1705	CS16 STRAP, CLEAR SPAN VARIES.
NONE	500	500	500	500	500	500	NONE

Foundation Level Holdown Capacities									
Type	Midwall Condition		Corner Condition		End Condition		Interior Condition		Description of Holdown Note: Holdowns w/ an "A" after the holdown number are installed to a larger post than their non-"A" counterparts. Refer to the holdown schedule on plan for additional information
	Wind	Seismic	Wind	Seismic	Wind	Seismic	Wind	Seismic	
21	14390	14390	14390	14390	14390	14390	14390	14390	HDU14 HOLDOWN TO MINIMUM 4X8 POST W/ 1" ANCHOR ROD.
19A	7870	7870	7870	7315	7310	6395	7870	7870	HDU8 HOLDOWN TO MINIMUM 4X6 POST W/ SSTB28/34 ANCHOR.
19	6970	6970	6970	6970	6970	6395	6970	6970	HDU8 HOLDOWN TO MINIMUM 4X POST W/ SSTB28/34 ANCHOR.
17	4565	3740	4295	3325	4295	3325	4565	3740	HDU4 HOLDOWN TO MINIMUM 4X POST W/ SSTB24/24
9	4020	3400	0	0	0	0	NA	NA	AT PT SLAB: STHD10 HOLDOWN TO MINIMUM 4X POST. AT CONVENTIONAL FDN: STHD14 HOLDOWN TO MINIMUM 4X POST.
NONE	500	500	500	500	500	500	500	500	NONE

NOTES:
1. Holdowns are designated on the plans by a diamond symbol surrounding the holdown type, e.g.: 2

Calculations For Anchor Bolts & Mudsill Anchors At Shearwalls

Allowable loads per NDS/hardware values (1.6 load duration factor)							
Load Source	Sill Plate Size	1/2" ϕ A.B.	5/8" ϕ A.B.	Simpson MASA standard	Simpson MASA one leg up	Simpson MASA (1) of (3) up	USP FA4 (1 of 3 up)
Wind	2x	650#	930#	1475#	965#	1305#	1135#
	3x	770#	1180#	1165#	760#	1030#	0#
Seismic	2x	650#	930#	1235#	845#	1105#	1035#
	3x	770#	1180#	1020#	685#	908#	0#

Mudsill Anchor Spacing for All Shearwall Types							
Shearwall Type	Sill Plate Size	Wind Capacity	Seismic Capacity	Anchor Spacing[1] 1/2" ϕ A.B.	Anchor Spacing[1] 5/8" ϕ A.B.	Anchor Spacing[1] MASA	Anchor Spacing[1] FA4
12	3x	970plf (W)	770plf (E)	15.2" oc	23.4" oc	12.7" oc	0.0" oc
				14.0" oc	22.0" oc	12.0" oc	N/A
4	2x	750plf (W)	640plf (E)	16.6" oc	23.8" oc	20.7" oc	18.2" oc
				16.0" oc	22.0" oc	16.0" oc	16.0" oc
2	2x	350plf (W)	350plf (E)	35.7" oc	51.0" oc	37.9" oc	35.5" oc
				34.0" oc	48.0" oc	34.0" oc	34.0" oc
							N/A
							N/A
							N/A
							N/A
							N/A
							N/A
							N/A
							N/A
							N/A
							N/A
							N/A
							N/A
							N/A
							N/A

Notes:
1. Shading indicates spacing used in shearwall schedule