STRUCTURAL CALCUALTIONS
for:

Fields
7520 North Mercer Way
Mercer Island, WA 98040

PROJECT:
Fields Remodel
7520 North Mercer Island
Mercer Island, WA 98040

DANIEL JAY CEDERLUND
STATE OF WASHINGTON
46346
REGISTERED
PROFESSIONAL ENGINEER

Project #: 23-147
Date: 5/22/2023

Puget Sound Structural Engineering

PO Box 5096, Tacoma WA
(253) 298-2554

BASIS OF DESIGN GENERAL

GENERAL: 2018 INTERNATIONAL BUILDING CODE as Amended by local jurisdiction
ASCE 7-16 Minimum Design loads for Buildings and other Structures

RISK CATEGORY: II

MATERIAL SPECIFIC CODES

WOOD: 2018 NATIONAL DESIGN SPECIFICATION for Wood

CONCRETE: 2014 ACI 318-19

STEEL: 2017 AISC 15th Edition

BASIS OF DESIGN - GRAVITY

SNOW:

Roof Snow Load	25	psf
Importance Factor (I_s)	1.00	Unitless

SOIL BEARING:

Allowable Soil bearing pressure	1500	psf

DEFLECTIONS:

	L	D + L
ROOF	L/240	L/180
FLOOR	L/360	L/240
WEB JOISTS or TILE FLOOR	L/480	L/360

BASIS OF DESIGN - LATERAL

WIND:

Importance Factor (I_w)	1.00	Unitless
Exposure Category	C	
Topographic Factor (K_{zt})	1.00	Unitless
Basic Wind Speed (V)	110	mph

SEISMIC:

Importance Factor (I_e)	1	Unitless			
Site Class	D				
Short Period Spectral Acceleration (S_S)	1.386	g	S_{DS}	1.108	g
One Second Spectral Acceleration (S_1)	0.483	g	S_{1S}	0.483	g
Response Modification Factor (R)	6.5	Unitless			

ALLOWABLE DRIFT: 2.50%

Gravity Analysis

GRAVITY LOADS

DEAD LOADS

Conccrete	150	pcf	
DF lumber	32	pcf	
Steel	490	pcf	
Plywood	36	pcf	

ROOF

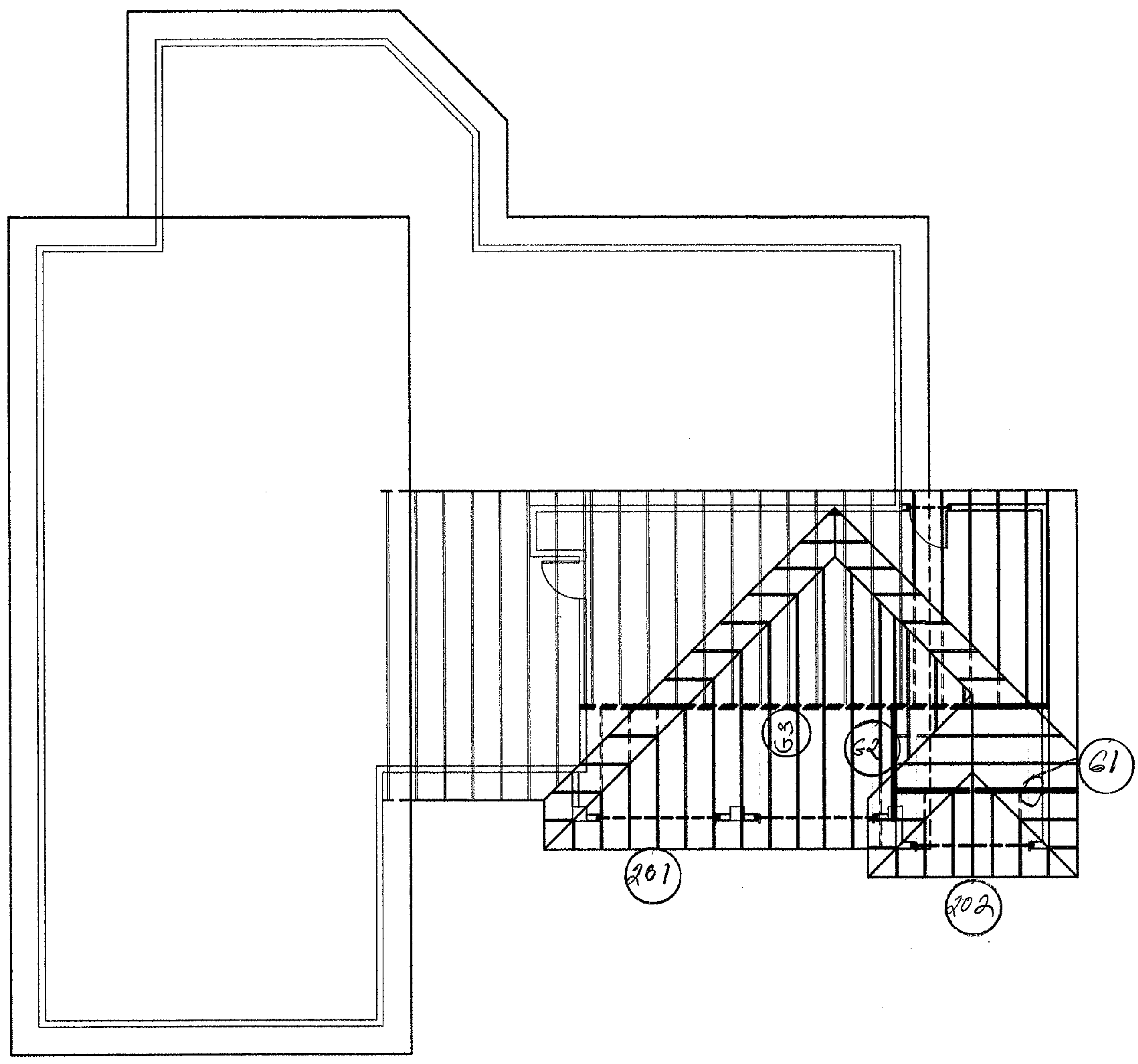
Truss @ 24" OC	2.2	psf	
1/2" Plywood	1.5	psf	
15 Lb Felt	0.4	psf	
Asphalt Shingles	8.6	psf	Assumes (2) Layers shingles
R49 Batt Insulation	1.1	psf	
5/8" GWB	2.8	psf	
Mechanical	0.5	psf	
Total	17.1	psf	
Use	**20**	**psf**	

EXTERIOR WALL

Framing	2.4	psf	Assumes 2x6 at 16" OC with double top plate
Sheathing	1.5	psf	
R 21 Insulation	0.5	psf	
1/2" GWB	2.2	psf	
Siding	2.2	psf	
Mechanical	0.5	psf	
Total	9.3	psf	
Use	**10**	**psf**	

LIVE LOADS

Roof	25	psf	Snow


63
62
61
201
202

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Project: Fields Remodel | **Sheet** | **of**
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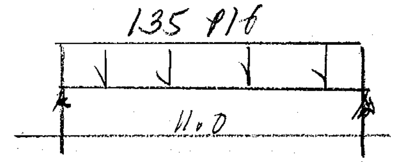
Roof Framing — Gravity

Girder Trusses

G1 $\ell = 11'\text{-}0"$

$w = (20+25)(6/2) = 60 + 75$ plf



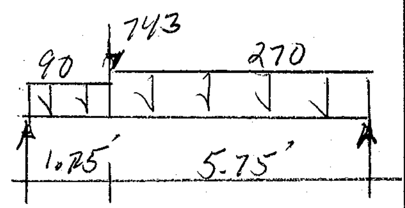
0.330 D
0.413 S

G2 $\ell = 7'\text{-}6"$

0-1.75' $w = (20+25)(4/2) = 40+50$ plf

1.75-7.5 $w = (20+25)(12/2) = 120+150$ plf

Pt load at 1.75' from G1 P_D 330 P_S 413



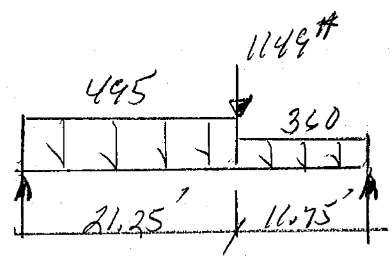
0.579 D — 0.511
0.725 S — 0.638

G3 $\ell = 33'\text{-}0"$

0-21.25 $w = (20+25)(22/2) = 220 + 275$ plf

21.25-33 $w = (20+25)(16/2) = 160+200$ plf

Pt load at 21.25 from Rt end G2 P_D 511 P_S 638



3,686 D — 3,380
4.608 S — 4.224

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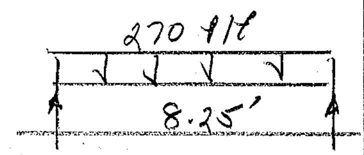
Roof Framing

Gravity

201 $\ell = 8'-3"$

$W = (20+25)(8/2+2) = 120 + 150$ plf

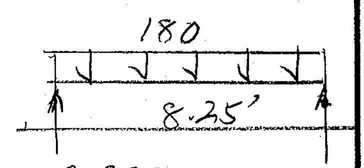


Use 4x8

0.495 D
0.619 S

202 $\ell = 8'-3"$

$W = (20+25)(4/2+2) = 80 + 100$ plf



Use 4x8

Load is less than 201

0.330
0.413

2018 NDS FOR SAWN LUMBER (TABLE 4A)

Project:	Fields Remodel	Span:	8.25	Ft	Enter Loading or M & V		
Calculation #:	**201**	W_D:	20	psf	Moment (M)		k-ft
Section:	4x8 DF	W_L:	25	psf	Shear (V)		kips
Moment:	2.297 k-ft	Trib Width:	6	ft			
Shear:	1.114 kips	Moment	2.30	k-ft			
		Shear	1.11	kips			

SECTION PROPERTYS

Depth (In)	Thick (In)	Rough Cut	Flat Use	Nom Depth	Nom Thick	Area (In^2)	Sx (In^3)	Ix (In4)	Quantity
8.0	4.0	NO	NO	7.5	3.5	26.25	32.81	123.0	1

WOOD PROPERTYS TABLE VALUES

Species / Grade		Fb (psi)	Ft (psi)	Fv (psi)	FcPerp (psi)	Fc (psi)	E (ksi)	Emin (ksi)	
DF #2		900	575	180	625	1350	1,600	580	
	C_D	1.15	1.15	1.15	-	1.15	-	-	Duration Factor
	C_F	1.30	1.20	-	-	1.05	-	-	Size Factor
	C_M	1.00	1.00	1.00	1.00	1.00	1.00	1.00	Moisture Factor
	C_t	1.00	1.00	1.00	1.00	1.00	1.00	1.00	Temperature Factor
	C_{fu}	1.00	-	-	-	-	-	-	Flat Use Factor
	C_i	1.00	1.00	1.00	1.00	1.00	1.00	1.00	Incising Factor
	C_r	1.00	-	-	-	-	-	-	Repetative Member Factor
	C_L	1.00	-	-	-	-	-	-	Beam Stability Factor
Adjusted Values		1346	794	207	625	1630	1600	580	

ADJUSTMENT FACTORS

Duration	2 Months					
Moisture	NO					
Temp > 100 F	NO	T>125 & T<=150	NO			
Incising	NO					
Repetative Member	NO					
Beam Stability	NO	lu	99	Inches	le	184 Inches

STRESS CHECK

Moment Capacity	**3.68**	k-ft	C/D	**1.60**	**OK**
Shear Capacity	**3.623**	kips	C/D	**3.25**	**OK**

DEFLECTION CHECK

Dead + Live Deflection	0.143	inches		Roof	NO
Span/Deflection	**693**	Unitless			
Allowable Span/Deflection	240	Unitless	**OK**	Deflection is less than Allowable Deflection	
Live Deflection	0.079	inches			
Span/Deflection	**1247**	Unitless			
Allowable Span/Deflection	360	Unitless	**OK**	Deflection is less than Allowable Deflection	

Deflection is for a uniformly loaded simply supported member.

Lateral Analysis

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Wind Areas — Lateral

Front - Rear

Worst Case is from rear

Area = 10.5(21) = 221 sf

Azone 2(3)21 = 126 sf
Czone 221-126 95 sf

End - End

No new area use end wall

$\left(\frac{21+11}{2}\right) 24.5 =$ 392 sf

Azone 2(3)21(1/2) = 63 sf
Czone 392-63 = 329 sf

LATERAL WIND DESIGN ASCE 7

Project: **Fields Remodel**

Risk Category	II	Table 1.5-1
Basic Wind Speed	110	mph / Figure 26.5-1A
Exposure Category	C	Section 26.7
Mean Roof Height (h)	16.0	Feet
Adjustment Factor for Building Height & Exposure (λ)	1.23	Unitless / Figure 28.6-1
Topographic factor (K_{zt})	1.00	Unitless / Figure 26.8-1
Least horizontal dimension	25	Feet
Distance (a)	3	Feet
Roof Angle	26.57	Degrees

Simplified Design Wind Pressure, ps30 for V = 110 mph											
	Horizontal Pressures				Vertical Pressures						
Angle	A	B	C	D	E	F	G	H	E_{OH}	G_{OH}	
25.0	24.10	3.90	17.40	4.00	-10.70	-14.60	-7.70	-11.70	-19.90	-17.00	$P_{s\,30}$ from Figure 28.6-1
30.0	21.60	14.80	17.20	11.80	1.70	-13.10	0.60	-11.30	-7.60	-8.70	$P_{s\,30}$ from Figure 28.6-1
26.6	23.32	7.31	17.34	6.44	7.88	14.13	5.48	11.57	16.05	-14.40	Interpolated $P_{s\,30}$ value
26.6	28.59	8.96	21.26	7.90	9.66	17.32	6.72	14.19	19.68	-17.66	Interpolated $P_{s\,30} * K_{zt} * \lambda$

P_{s30} values are converted to Absolute values before interpolating.

Roof Area (SF)

	A	B	C	D	E	F	G	H	E_{OH}	G_{OH}	Roof Kips	Wall Kips	Total Kips
N-S											0.00		0.00
E-W											0.00		0.00

Wall Area (SF)

	A	B	C	D	E	F	G	H	E_{OH}	G_{OH}	Roof Kips	Wall Kips	Total Kips	Base Shear Total Kips
N-S	126.0		95.0									2.81	2.81	2.81
E-W	63.0		329.0									4.40	4.40	4.40

Fields Remodel

Latitude, Longitude: 47.5912, -122.238

Date	5/22/2023, 11:33:25 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	1.386	MCE_R ground motion. (for 0.2 second period)
S_1	0.483	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.663	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.108	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.593	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.711	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.386	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.535	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	3.112	Factored deterministic acceleration value. (0.2 second)
S1RT	0.483	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.539	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.281	Factored deterministic acceleration value. (1.0 second)
PGAd	1.08	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.593	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.903	Mapped value of the risk coefficient at short periods

LATERAL SEISMIC DESIGN ASCE 7

Project: **Fields Remodel**

$V = C_s * W$			12.8-1
Site Class	D		
S_{DS}	1.108	g	
S_{D1}	0.483	g	
Response Modification Factor "R"	6.5	Unitless	Table 12.2-1
Structural Height (h_n)	11.0	Ft	11.2
Fundamental Period (T)	0.12	Seconds	12.8-7
Importance Factor (I_E)	1.00	Unitless	Table 1.5-2
Seismic Response Coefficient (C_s)	0.170	g	12.8-2
C_s Max	0.615	g	12.8-3
C_s Min	0.049	g	12.8-5
C_s	0.170	g	12.8-2
Exponent related to the period (k)	1.0	Unitless	12.8-12

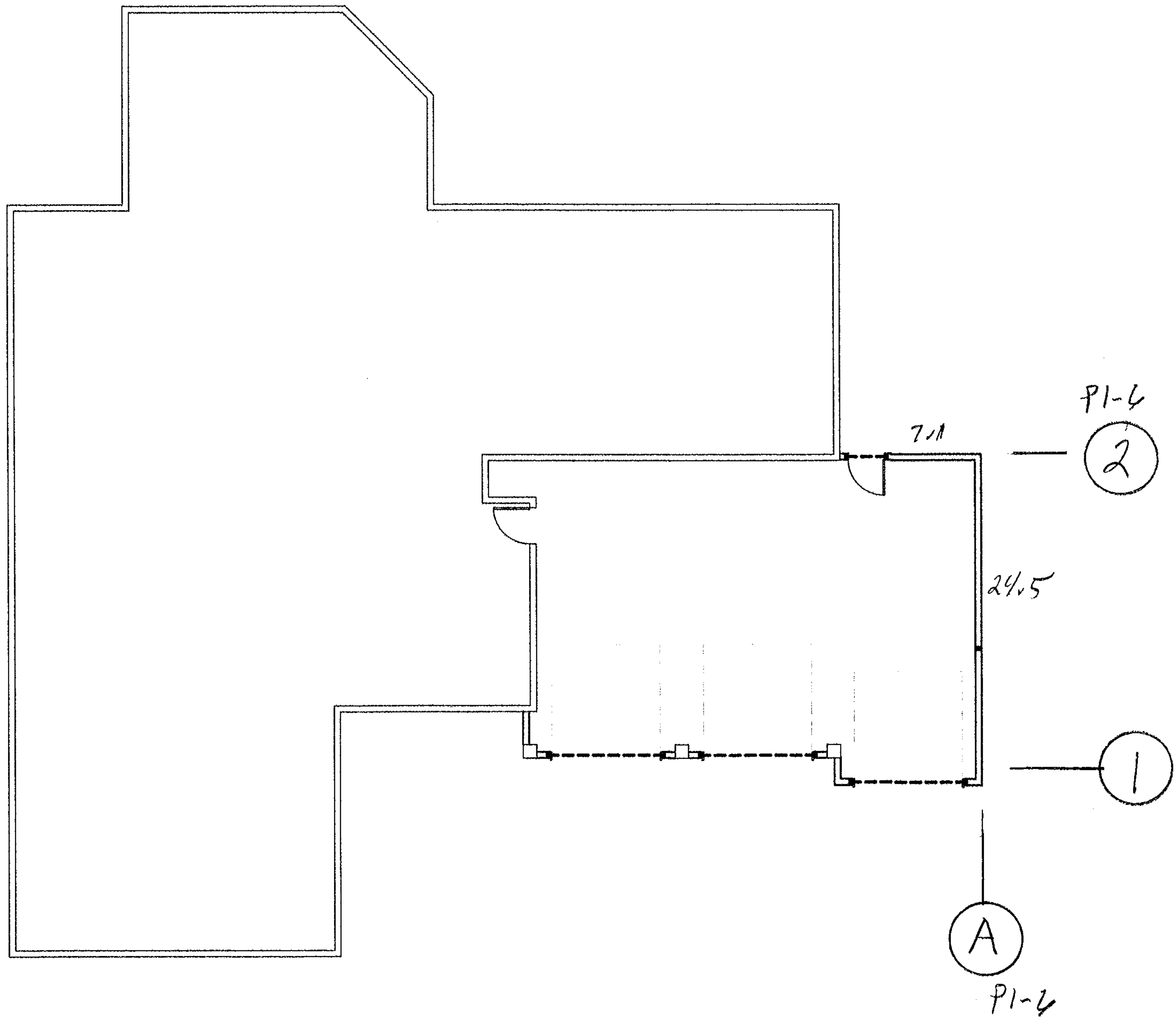
Seismic Weight		
Roof (psf)	Wall (psf)	Floor (psf)
20	10	

Level	Mean Height	Area Roof	Area Wall	Area Floor	Weight	Shear (V)	$W*h^k$	C_{vx} (12.8-12)	F_x (12.8-11)	Base Shear
	Ft	Sq Ft	Sq Ft	Sq Ft	Kips	Kips	k-ft	Unitless	Kips	Kips
Roof	15.0	573			11.45	1.95	172	0.87	2.32	
Walls	6.0		828		4.14	0.71	25	0.13	0.34	2.66
				Total Base Shear (V)		2.66	197	Sum		

Structural Height (h_n) is mean roof, center of wall, or floor.

$C_{vx} = (W*h^k) / \text{Sum}(W*h^k)$ Percentage of total base shear (V) assigned to this level

$F_x = C_{vx} * \text{Total Base Shear (V)}$



P1-4
7.1
2
24.5
1
A
P1-4

Project: **Fields Remodel**

Lateral Force Summary

	Force (kips)								
	N-S Wind			E-W Wind			Seismic		
	N-S Top	N-S Mid	N-S Total	E-W Top	E-W Mid	E-W Total	Top	Mid	Total
Totals	0.00	2.81	2.81	0.00	4.40	4.40	2.32	0.34	2.66

	Wind				Seismic										
Wall Line	% of Wind Force to Wall Line	Wind Force Applied at Top of Wall	Wind Force Applied at Center of Wall	Total Wind Force Applied to Wall Line	% of Seismic Force to Wall Line	Seismic Force Applied at Top of Wall	Seismic Force Applied at Center of Wall	Total Seismic Force Applied to Wall Line	Total Shearwall Length in Wall Line	Wall Height (h)	Minimum Panel Length in Wall Line (b)	Seismic Aspect Ratio 1.25-0.125*(h/b)	Seismic Force Adjusted for Aspect Ratio	Seismic Unit Shear	Wind Unit Shear
	%	Kips	Kips	Kips	%	Kips	Kips	Kips	Ft	Ft	Ft	Unitless	Kips	Lbs/Ft	Lbs/Ft
A	100%	0.00	2.81	2.81	100%	2.32	0.34	2.66	10.00	11.00	5.00	0.98	2.73	273	281
B	0%	0.00	0.00	0.00	0%	0.00	0.00	0.00	10.00	9.00	5.00	1.03	0.00	0	0
C	0%	0.00	0.00	0.00	0%	0.00	0.00	0.00	10.00	9.00	5.00	1.03	0.00	0	0
Sum	100%	0.00	2.81	2.81	100%	2.32	0.34	2.66							
1	35%	0.00	1.54	1.54	35%	0.81	0.12	0.93	10.00	9.00	5.00	1.03	0.93	93	154
2	65%	0.00	2.86	2.86	65%	1.51	0.22	1.73	7.10	19.60	7.10	0.90	1.91	269	403
3	0%	0.00	0.00	0.00	0%	0.00	0.00	0.00	10.00	9.00	5.00	1.03	0.00	0	0
Sum	100%	0.00	4.40	4.40	100%	2.32	0.34	2.66							

b Min: Minimum wall panel length in a wall line used for determining seismic aspect ratio adjustment

Seismic Force Adjusted for Aspect Ratio =
IF Seismic Aspect Ratio < 1.0
Seismic Force Adjusted for Aspect Ratio = Total Seismic Force to Grid Line / Seismic Aspect Ratio
IF Seismic Aspect Ratio => 1.0
Seismic Force Adjusted for Aspect Ratio = Total Seismic Force to Grid Line

(Wind / Seismic) Unit Shear = Total Force to Grid Line / Total Shearwall Length
Lettered Wall lines resist N-S wind forces
Wind & Seismic Unit Shears are "Strength" level forces

SHEAR WALL PANEL NAILING AND HOLD DOWN DESIGN

Project: **Fields Remodel**

Grid Line	**A**		**HF**	**Wood Species**
Wall Height	11.00	ft		
Seismic Unit Shear	273	plf	Strength Level	
Wind Unit Shear	281	plf	Strength Level	

Shearwall Design: Type and Capacity

Shear Wall Designation		**P1-6**	
	Demand	Capacity	
Service Level	plf	plf	
Seismic Unit Shear	191	237	OK
Wind Unit Shear	169	332	OK

Service Level Force to Wall Line			
Wind Force		Seismic Force	
V_T	V_M	V_T	V_M
lbs	lbs	lbs	lbs
0	1686	1625	235
plf	plf	plf	plf
0	69	66	10

Holdown Design: Dead Loads

Panel	Panel	Dead Load Resisting				Wind		Seismic	
#	Length	Roof Wt	Wall Wt	Floor Wt	Total Wt	Wind T	Wind C	EQ T	EQ C
	Ft	plf	plf	plf	plf	lbs	lbs	lbs	lbs
1	24.50	60	120		180	662	2205	834	2694
	24.50	Sum							

Holdown Design: Tension and Compression

Panel	Wind					Seismic				
#	T = C	T-0.6*D	T Above	C + DL	C Above	T = C	$T-(0.6-0.2S_{DS})*D$	T Above	$C+(1.2*S_{DS})*D$	C Above
	lbs	lbs	lbs	lbs	lbs	lbs	lbs	lbs	lbs	lbs
1	379	-283		2584		782	-52		3476	

Holdown Design: Type and Capacity

Panel #	Wind		Seismic		Holddown	Holddown	Holdown
	Tension	Comp	Tension	Comp	Type	Capacity	Post
	lbs	lbs	lbs	lbs		lbs	
1	-283	2584	-52	3476	NA	0	0

Allowable Wind Unit Shear = 0.6 * Strength Level Wind Unit Shear
Allowable Seismic Unit Shear = 0.7 * Strength Level Seismic Unit Shear

Dead Load Resisting:
Total Dead Load (plf)= (Roof Dead + Wall Dead + Floor Dead)
Wind T (Lbs) = 0.6* Total Dead Load * Length / 2, Wind C (Lbs) = Total Dead Load * Length / 2
EQ T (Lbs) = (0.6-0.2*SDS)* Total Dead Load * Length / 2, EQ C (Lbs) = Total Dead Load * (1.2*SDS) * Length / 2
(If Length > 12 then Length / 3, If Length > 24 then Length / 4)

T = C = vt (top) * Height + vt (mid) * Height / 2
Wind Tension = Panel Tension - 0.6*DL + Tension Above
Wind Compresion = Panel Compresion + DL + Compresion Above

V_T (Lbs): Total shear force (Allowable) this level only in the wall line acting at the Top of Wall.
V_M (Lbs): Total shear force (Allowable) this level only in the wall line acting at the Middle of Wall.
vt (plf) = Vt / Sum of Panel Length in wall line

Uplift of less than 1500 lbs not considered to require Holdowns

SHEAR WALL PANEL NAILING AND HOLD DOWN DESIGN

Project: **Fields Remodel**

Grid Line	**2**		**HF**	**Wood Species**
Wall Height	19.60	ft		
Seismic Unit Shear	269	plf	Strength Level	
Wind Unit Shear	403	plf	Strength Level	

Shearwall Design: Type and Capacity

Shear Wall Designation		**P1-6**	
	Demand	Capacity	
Service Level	plf	plf	
Seismic Unit Shear	188	237	OK
Wind Unit Shear	242	332	OK

Service Level Force to Wall Line			
Wind Force		Seismic Force	
V_T	V_M	V_T	V_M
lbs	lbs	lbs	lbs
0	1715	1056	153
plf	plf	plf	plf
0	242	149	22

Holdown Design: Dead Loads

Panel	Panel	Dead Load Resisting				Wind		Seismic	
#	Length	Roof Wt	Wall Wt	Floor Wt	Total Wt	Wind T	Wind C	EQ T	EQ C
	Ft	plf	plf	plf	plf	lbs	lbs	lbs	lbs
1	7.10	175	190		365	777	1296	490	1583
	7.10	Sum							

Holdown Design: Tension and Compression

Panel	Wind					Seismic				
#	T = C	T-0.6*D	T Above	C + DL	C Above	T = C	T-(0.6-0.2S_{DS})*D	T Above	C+(1.2*S_{DS})*D	C Above
	lbs	lbs	lbs	lbs	lbs	lbs	lbs	lbs	lbs	lbs
1	2367	1590		3663		3127	2637		4710	

Holdown Design: Type and Capacity

Panel #	Wind		Seismic		Holddown	Holddown	Holdown	
	Tension	Comp	Tension	Comp	Type	Capacity	Post	
	lbs	lbs	lbs	lbs		lbs		
1	1590	3663	2637	4710	6-STHD10-Corner	2640	3"	OK

Allowable Wind Unit Shear = 0.6 * Strength Level Wind Unit Shear
Allowable Seismic Unit Shear = 0.7 * Strength Level Seismic Unit Shear

Dead Load Resisting:
Total Dead Load (plf)= (Roof Dead + Wall Dead + Floor Dead)
Wind T (Lbs) = 0.6* Total Dead Load * Length / 2, Wind C (Lbs) = Total Dead Load * Length / 2
EQ T (Lbs) = (0.6-0.2*SDS)* Total Dead Load * Length / 2, EQ C (Lbs) = Total Dead Load * (1.2*SDS) * Length / 2
(If Length > 12 then Length / 3, If Length > 24 then Length / 4)

T = C = vt (top) * Height + vt (mid) * Height / 2
Wind Tension = Panel Tension - 0.6*DL + Tension Above
Wind Compresion = Panel Compresion + DL + Compresion Above

V_T (Lbs): Total shear force (Allowable) this level only in the wall line acting at the Top of Wall.
V_M (Lbs): Total shear force (Allowable) this level only in the wall line acting at the Middle of Wall.
vt (plf) = Vt / Sum of Panel Length in wall line

Uplift of less than 1500 lbs not considered to require Holdowns

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Lateral 1

Wall Line 1

Forces

Wind 1.54 K (0.6) = 0.924 Allowable ← CONTROLS

EQ 0.93 K (0.7) = 0.651 Allowable

Use APA Portal Frames

At Popout (2) 16" Panels 10' wall

Capacity = 2(625) = 1250# > 924

Use 2 additional portal frames at 1st garage door

(2) 2 ft × 8' frames Capacity = 2(1675) = 3350#

Total Capacity 1250 + 3350 = 4600# > 924#

Table 1. Recommended Allowable Design Values for APA Portal Frame Used on a Rigid-Base

Minimum Width (in.)	Maximum Height (ft)	Allowable Design (ASD) Values per Frame Segment		
		Shear[e,f] (lbf)	Deflection (in.)	Load Factor
16	8	850	0.33	3.09
	10	625	0.44	2.97
24	8	1,675	0.38	2.88
	10	1,125	0.51	3.42

Foundation for Wind or Seismic Loading[a,b,c,d]

(a) Design values are based on the use of Douglas-fir or Southern pine framing. For other species of framing, multiply the above shear design value by the specific gravity adjustment factor = (1 – (0.5 – SG)), where SG = specific gravity of the actual framing. This adjustment shall not be greater than 1.0.

(b) For construction as shown in Figure 1.

(c) Values are for a single portal-frame segment (one vertical leg and a portion of the header). For multiple portal-frame segments, the allowable shear design values are permitted to be multiplied by the number of frame segments (e.g., two = 2x, three = 3x, etc.).

(d) Interpolation of design values for heights between 8 and 10 feet, and for portal widths between 16 and 24 inches, is permitted.

(e) The allowable shear design value is permitted to be multiplied by a factor of 1.4 for wind design.

(f) If story drift is not a design consideration, the tabulated design shear values are permitted to be multiplied by a factor of 1.15. This factor is permitted to be used cumulatively with the wind-design adjustment factor in Footnote (e) above.

Figure 1. Construction Details for APA Portal-Frame Design with Hold Downs