Structural Calculations For:

Fukano Residence

Structural Calculations

6600 82nd Ave SE

Mercer Island, WA

DAN J. SAY
STATE OF WASHINGTON
23000
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER

10-5-20

Prepared for: Suyama Peterson Deguchi

Job #: 00043-2018-09

Date: October 5, 2020

Criteria Sheet

Codes:

Structural:	IBC 2015
Loading:	ASCE 7-10
Wood:	NDS 2015
Steel:	AISC 360-10
Concrete:	ACI 318-14
Masonry:	TMS 402/602-13

Project Location:

Street & Number:	6600 82nd Ave SE		
City:	Mercer Islanc	State:	WA
ZIP:			
Latitude:	47.5434 N		
Longitude:	-122.2290 W		

Occupancy Category

Risk Category: II — ASCE 7 Table 1.5-1

Seismic Load Summary:

Analysis Procedure: Equivalent Lateral Force Procedure

Lateral System: Light-frame (wood) Walls Sheathed with Wood Structural Panels Rated for Shear Resistance

R:	6.50	C_d=	4
Base Shear V =	7 kips	Ω_o=	2.5
S_S=	1.46	S_1=	0.559
S_{DS}=	0.97	S_{D1}=	0.56
C_s=	0.150	I_E=	1.0

Wind Load Summary:

V=	110	K_{ZT}=	1.00
Exposure =	B		

Dead Loads:

Roof		
Roofing	2.5	psf
1/2" Sheathing	1.8	psf
Trusses @ 24" oc	2.5	psf
Misc./Mech.	1.5	psf
Ceiling Finish	2.8	psf
Solar Panels	4	
	15	psf
Use	15	psf
Floor		
Finish Floor	1	psf
3/4" Sheathing	2.7	psf
Joists @ 16" oc	2.2	psf
Misc./Mech.	2	psf
Ceiling Finish	2.8	
	10.7	psf
Use	12	psf

Live Loads:

Snow	25	psf
Floor	40	psf

Soils:

Allowable Bearing	1500	psf

STRUCTURAL ENGINEERING

Fukano		DATE	7/1/2020
Criteria		PROJ. #	
		DESIGN	RJA
		SHEET	1

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Seismic Design

ASCE 7-10 Seismic Analysis — Equivalent Lateral Force Procedure

Seismic Force Resisting System: Per Table 12.2-1	System:	Bearing Wall Systems
	Type:	Light-frame (wood) Walls Sheathed with Wood Structural Panels Rated for Shear Resistance

Seismic Design Cat.	D	
Risk Category	II	I, II, or III, or IV per Table 1.5-1
Site Class	D	per soils report (D assumed, without soils report)
Diaphragm Flexibility	Flexible	

Ω_o	2.5	
S_S	1.46 g	2% in 50 yr, Latitude & Longitude lookup
S_1	0.559 g	2% in 50 yr, Latitude & Longitude lookup
h_n	11.0 ft	
R	6.50	
I_e	1.0	Table 1.5-2
C_d	4	
Ct	0.02	Table 12.8-2
x	0.75	Table 12.8-2
T	0.12 sec	Eq. 12.8-7
T_0	0.11 sec	
T_S	0.57 sec	
k	1.000	
Fa	1.00	Table 11.4-1
Fv	1.50	Table 11.4-2
S_{MS}	1.46 g	Eq. 11.4-1
S_{M1}	0.84 g	Eq. 11.4-2
S_{DS}	0.973 g	Eq. 11.4-3
S_{D1}	0.559 g	Eq. 11.4-4
Cs	***0.150 Controls***	Eq. 12.8-2
	0.712	Eq. 12.8-3 need not exceed, $T < T_L$
	0.010	Eq. 12.8-5 or 12.8-6 minimum
Cs, design	0.150	
Bldg. Weight	49.0 k	
$V = C_S W$	7.3 k	Eq. 12.8-1, Strength Level Base Shear
$V = C_{Sasd} W$	5.1 k	Eq. 12.8-1 ASD Base Shear

$T_a = C_t h_n^x$	Eq. 12.8.7
$S_{MS} = F_a S_S$	Eq. 11.4-1
$S_{M1} = F_v S_1$	Eq. 11.4-2
$S_{DS} = {}^2/_3 S_{MS}$	Eq. 11.4-3
$S_{D1} = {}^2/_3 S_{M1}$	Eq. 11.4-4
$C_S = \frac{S_{DS}}{(R/I_e)}$	Eq. 12.8-2
$C_S = \frac{S_{D1}}{T(R/I_e)}$	Eq. 12.8-3
$C_S = \frac{S_{D1}T_L}{T^2(R/I_e)}$	Eq. 12.8-4
$C_S \geq 0.044 S_{DS} I_e$	Eq. 12.8-5
$C_S \geq 0.01$	Eq. 12.8-5

$C_{VX} = w_x h_x^k / \sum_{i=1}^n w_x h_i^k$	Eq. 12.8-12
$F_{px} = \sum_{i=x}^n F_i / \sum_{i=x}^n w_i \, w_{px}$	Eq. 12.10-1
$F_{px} \geq 0.2 S_{DS} I_e w_{px}$	Eq. 12.10-2
$F_{px} \leq 0.4 S_{DS} I_e w_{px}$	Eq. 12.10-3

Vertical Distribution ASD ρ= 1.3

Level	h_x (ft)	W_x (k)	h_x^k (ft)	$W_x h_x^k$	Story Shear ASD			Diaphragm Force (ρ not included)				
					C_{vx} (%)	F_x (k)	SV (k)	$F_{px,calc}$	$F_{px,min}$	$F_{px,max}$	$F_{px,design}$	$\gamma = F_{px}/F_x$
Roof	11.0	49	11.0	539	1.00	6.68	**6.68**	5.1	6.7	13.4	**6.7**	1.00
Σ		49.0		539		**6.68**						

STRUCTURAL ENGINEERING	Fukano	DATE	7/1/2020
	Seismic Criteria	PROJ. #	
		DESIGN	RJA
		SHEET	2

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Wind Design - MWFRS

ASCE 7-10 Chapter 27 - Directional Procedure

Design Method	ASD

Wind Coefficients

Exposure	B	
V=	110	mph
K_d=	0.85	Table 26.6-1
K_h=	0.57	Table 27.3-1
G=	0.85	26.9.4

Location and Building Dimensions

Calculate Kzt?	No	
Kzt	1.00	
Roof Type	Gable	
Roof Angle - Transverse Dir	22	degrees
Roof Angle - Long Dir	0	degrees
Ground to top of roof	17	ft
Bot of roof to top of roof	8.5	ft
Mean Roof Height, h	12.75	ft
Short Plan Dimension	44	ft
Long Plan Dimension	64	ft
Parapet ?	No	
Ground to top of parapet		ft
Average Parapet Height		ft
Ht of 2nd Level Above Grade	0	ft

Velocity Pressure at Mean Roof Height, q_h =	15.1	psf

Transverse Wind Pressures

L/B = 0.69 h/L = 0.29

Pressure Coefficients from Figure 27.4-1:

Bldg Face	C_p
Windward Wall	0.8
Leeward Wall	-0.50
Windward Roof	-0.28 / 0.21
Leeward Roof	-0.60

Wall Pressures (Unfactored): ASD

Ht	K_z	q_z	$P_{ww\ walls}$	$P_{lwwalls}$	P_{walls} (psf)
0-15	0.58	15.27	10.38	6.43	**10.09**
15-20	0.62	16.32	11.10	6.43	**10.52**
20-25	0.66	17.38	11.82	6.43	**10.95**
25-30	0.7	18.43	12.53	6.43	**11.38**
30-40	0.76	20.01	13.61	6.43	**12.02**
41-50	0.81	21.33	14.50	6.43	**12.56**
51-60	0.85	22.38	15.22	6.43	**12.99**
61-70	0.89	23.43	15.93	6.43	**13.42**
71-80	0.93	24.49	16.65	6.43	**13.85**
81-90	0.96	25.28	17.19	6.43	**14.17**
91-100	0.99	26.07	17.73	6.43	**14.49**

Roof Pressures (Unfactored) ASD

Windward		Leeward	Horiz Proj (psf)
Max	Min		
2.8	-3.5	-7.7	**4.80**

Longitudinal Wind Pressures

L/B = 1.45 h/L = 0.20

Pressure Coefficients from Figure 27.4-1:

Bldg Face	C_p
Windward Wall	0.8
Leeward Wall	-0.41
Windward Roof	-0.9 / -0.18
Leeward Roof	-0.30

Wall Pressures (Unfactored): ASD

Ht	K_z	q_z	$P_{ww\ walls}$	$P_{lwwalls}$	P_{walls} (psf)
0-15	0.58	15.27	10.38	5.26	**9.60**
15-20	0.62	16.32	11.10	5.26	**9.82**
20-25	0.66	17.38	11.82	5.26	**10.25**
25-30	0.7	18.43	12.53	5.26	**10.68**
30-40	0.76	20.01	13.61	5.26	**11.32**
41-50	0.81	21.33	14.50	5.26	**11.86**
51-60	0.85	22.38	15.22	5.26	**12.29**
61-70	0.89	23.43	15.93	5.26	**12.72**
71-80	0.93	24.49	16.65	5.26	**13.15**
81-90	0.96	25.28	17.19	5.26	**13.47**
91-100	0.99	26.07	17.73	5.26	**13.79**

Roof Pressures (Unfactored) ASD

Windward		Leeward	Horiz Proj (psf)
Max	Min		
-2.3	-11.6	-3.9	**4.80**

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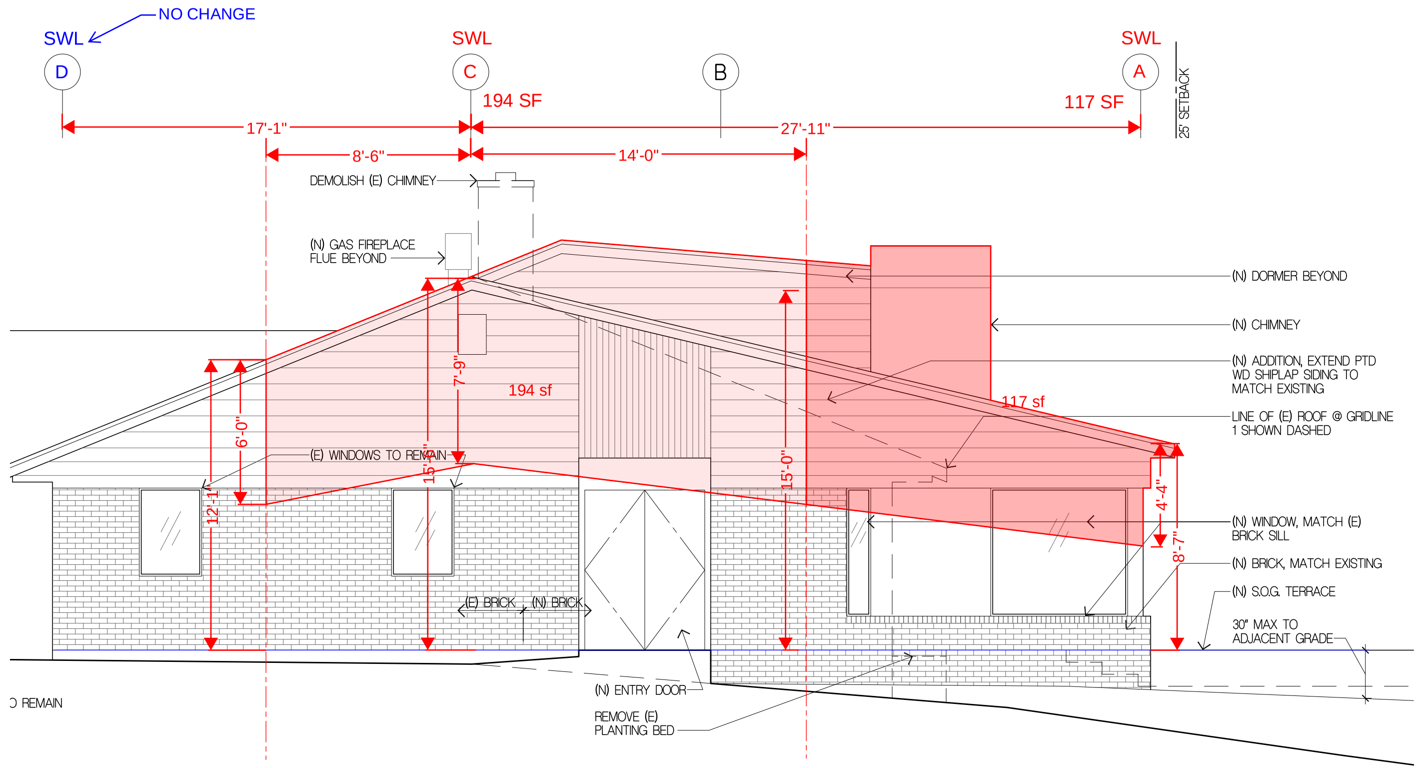
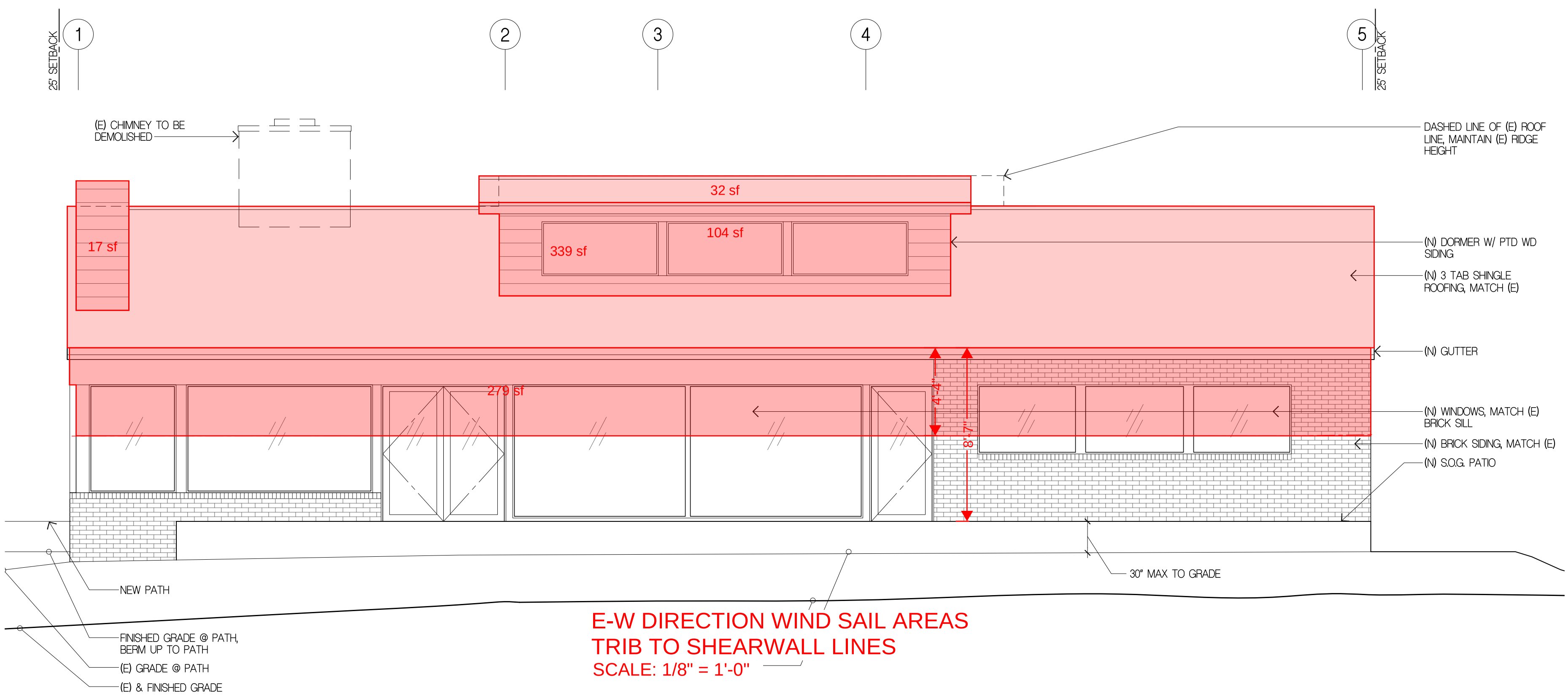
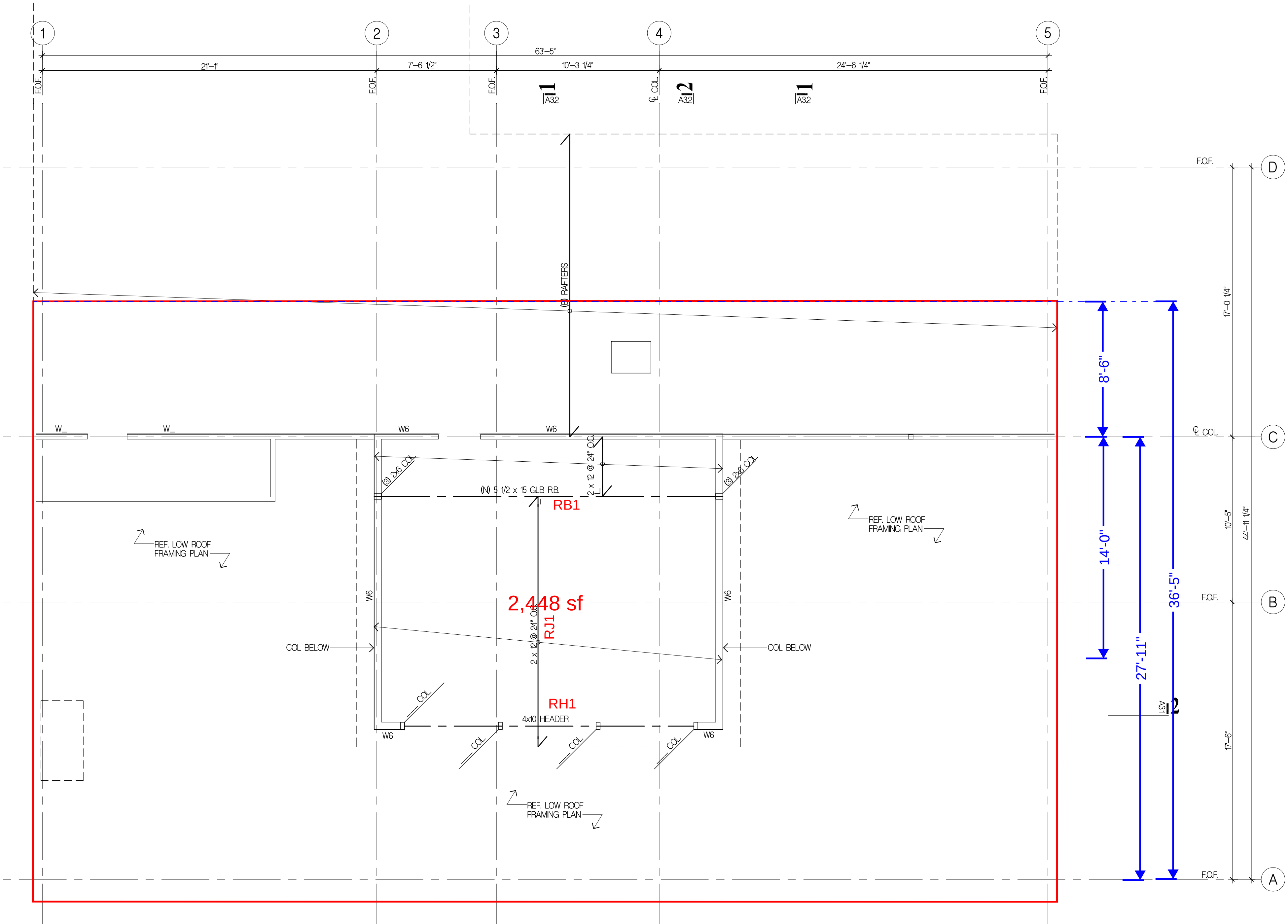


N-S DIRECTION WIND SAIL AREAS
TRIB TO SHEARWALL LINES
SCALE: 1/8" = 1'-0"

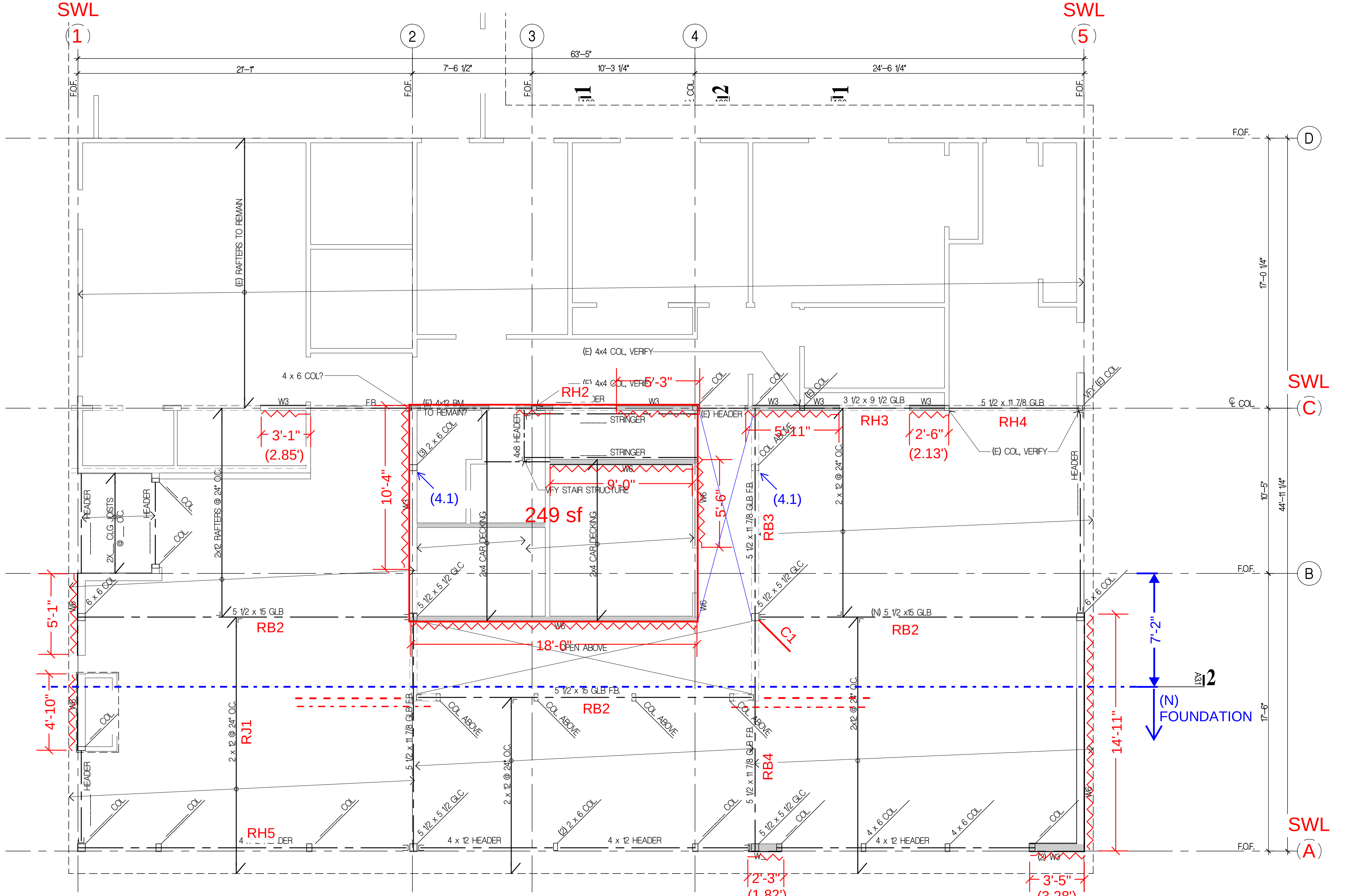


E-W DIRECTION WIND SAIL AREAS
TRIB TO SHEARWALL LINES
SCALE: 1/8" = 1'-0"



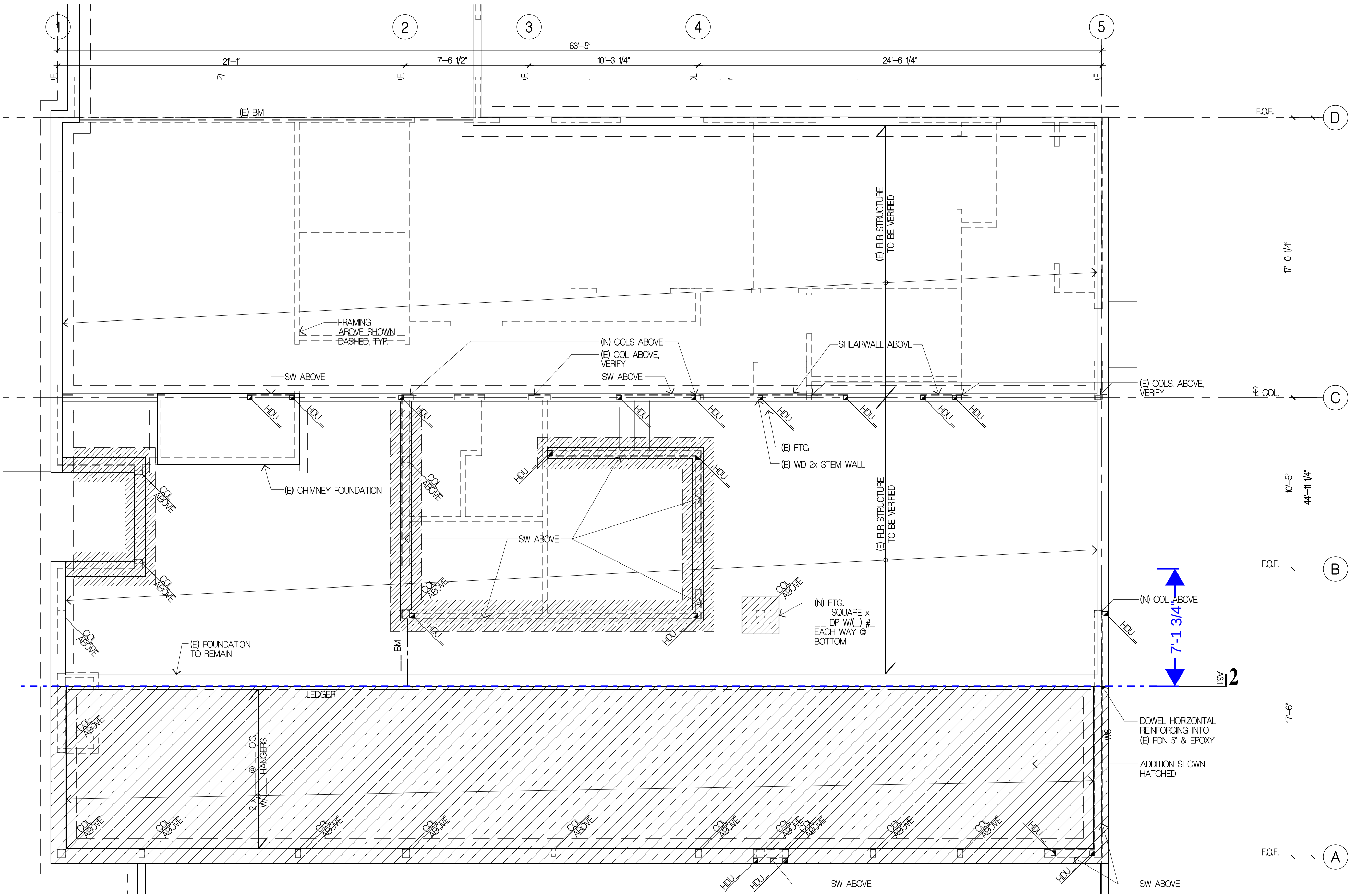
HIGH ROOF FRAMING KEY PLAN

SCALE: 1/8" = 1'-0"



MEZZ. & MAIN RF FRAMING KEY PLAN

SCALE: 1/8" = 1'-0"



FLOOR/FOUNDATION KEY PLAN
SCALE: 1/8" = 1'-0"

LATERAL ANALYSIS

SEISMIC: RF DL = 15 PSF
FLR DL = 12 PSF

MAIN RF: 2450(15+5) = 49.0^k (N-S DIRECTION)
MEZZ: 250 (12+5) = 4.3^k
↳ GRIDLINES A TO D, EAST OF D NO CHANGE TO LFRS

$C_s = 0.150$
$F_{x\,RF} = 0.7(0.150)(49)(1.3) = 6.69^k$
$F_{x\,MEZZ} = 0.7(0.150)(4.3)(1.3) = 0.59^k$ | CONTROLS MEZZ. LAT.

N-S | TRIB TO SWL-A: 6.69(14/36.4) = 2.57^k | CONTROLS
TRIB TO SWL-C: 6.69 - 2.57 = 4.12^k

E-W | TRIB TO SWL-1: 6.69(28/34.4)/2 = 2.72^k
TRIB TO SWL-5: 2.72^k

WIND: 110 MPH EXP. B $K_{zt} = 1.6$

N-S | TRIB TO SWL-A: 117(15.2) = 1.78^k
TRIB TO SWL-C: 194(15.3) = 2.97^k

E-W | 279(15) + (104+17)(15.2) + 4.8(339+32) = 7.8^k
TRIB TO SWL-1: 7.8/2 = 3.9^k | CONTROLS
TRIB TO SWL-5: 3.9^k

PROJECT: FURLANO
DATE
PROJ. #
DESIGN: RJA
SHEET

LATERAL DESIGN

N-S DIRECTION: SEISMIC CONTROLLED

SWL	D	C	A
V	NO CHANGE	4.12k	2.57k
l_w		16.1'	5.1'
v		255 plf	503 plf
SW		W4	W2
UP		2.0k	4.0k
NET UP		1.7k	3.7k
H.D.		HDU2	HDU5

E-W DIRECTION: WIND CONTROLLED

↳ NOTE: CONSERVATIVELY NEGLECTS CONTRIBUTION OF (E) SW EAST OF GRIDLINE C (FOR HD INTO (N) CONC.)

SWL	1	5	
V	3.9k	3.9k	
l_w	9.91'	14.92'	10'
v	393 plf	261 plf	390
SW	W4	W4	W4
UP	4.4k	2.9k	4.2k
NET UP	4.4k	2.9k	4.2k
H.D.	HDU8	HDU4 →	HDU8

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LATERAL DESIGN

MEZZ. LATERAL:

N-S DIRECTION: SEISMIC CONTROLLED

SWL	A.8	B.7
V	0.3k	0.3k
lw	18'	9'
v	17plf	33plf
SW	W6	W6
UP	0.1k	0.2k
NET UP	ϕ	ϕ
H.D.	NONE	NONE

E-W DIRECTION: SEISMIC CONTROLLED

SWL	2	4
V	0.3k	0.3k
lw	10'	5.5'
v	30plf	54plf
SW	W6	W6
UP	0.21k	0.38k
NET UP	ϕ	ϕ
H.D.	NONE	NONE

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SHEAR WALL SCHEDULE CALCULATIONS

A35 CAPACITY = 600# | SIMPSON C-2011
LTP4 CAPACITY = 575#

5/8" φ A.B. w/ 2x SILL PL = 1.6 (860) = 1376# | 2005 NDS
5/8" φ A.B. w/ 3x SILL PL = 1.6 (1070) = 1712#

16d x 0.135" φ x 3 1/2" w/ 2x SIDE MEMBER = $1.6(89)\left(\frac{1.25}{1.35}\right) = 132\#$ | 2005 NDS
16d x 0.131" φ x 3 1/4" w/ 2x SIDE MEMBER = $1.6(84)\left(\frac{1.00}{1.31}\right) = 103\#$

SHEAR WALL W6 - 242 PLF CAPACITY

15/32" CDX SHEATHING w/ 8d @ 6" o/c:

SEISMIC: 520 (1/2) (0.93) = 242 PLF ← CONTROLS
WIND: 730 (1/2) (0.93) = 339 PLF

A35 CLIP @ 24" o/c = 600 (12/24) = 300 PLF
BASE PL NAILING: 16d @ 6" o/c = 132 (12/6) = 264 PLF
SILL PL ANCHORAGE: 5/8" φ @ 48" o/c = 1376 (12/48) = 344 PLF

SHEAR WALL W4 - 353 PLF CAPACITY

15/32" CDX SHEATHING w/ 8d @ 4" o/c:

SEISMIC: 760 (1/2) (0.93) = 353 PLF ← CONTROLS
WIND: 1065 (1/2) (0.93) = 495 PLF

A35 CLIP @ 16" o/c = 600 (12/16) = 450 PLF
BASE PL NAILING: 16d @ 4" o/c = 132 (12/4) = 396 PLF
SILL PL ANCHORAGE: 5/8" φ @ 32" o/c = 1376 (12/32) = 516 PLF

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SHEAR WALL SCHEDULE CALCULATIONS

SHEAR WALL W3 – 456 PLF CAPACITY

15/32" CDX SHEATHING w/ 8d @ 3" o/c :

SEISMIC : 980 (1/2)(0.93) = 456 PLF ← CONTROLS

WIND : 1370 (1/2)(0.93) = 637 PLF

A35 CLIP @ 12" o/c = 600(1) = 600 PLF

BASE PL NAILING : 16d @ 3" o/c = 132 (12/3) = 528 PLF

SILL PL ANCHORAGE : 5/8" φ @ 16" o/c = 1376 (12/16) = 1032 PLF

SHEAR WALL W2 – 595 PLF CAPACITY

15/32" CDX SHEATHING w/ 8d @ 2" o/c :

SEISMIC : 1280 (1/2)(0.93) = 595 PLF ← CONTROLS

WIND : 1790 (1/2)(0.93) = 832 PLF

A35 CLIPS @ 9" o/c = 600 (12/9) = 800 PLF

BASE PL NAILING : 16d @ 2" o/c = 132 (12/2) = 792 PLF

SILL PL ANCHORAGE : 5/8" φ @ 12" o/c = 1376 (1) = 1376 PLF

SHEAR WALL 2W3 – 911 PLF CAPACITY

15/32" CDX SHEATHING w/ 8d @ 3" o/c EA SIDE

SEISMIC : 980 (2)(1/2)(0.93) = 911 PLF ← CONTROLS

WIND : 1370 (2)(1/2)(0.93) = 1274 PLF

A35 CLIP @ 6" o/c = 600 (12/6) = 1200 PLF

BASE PL NAILING : (2)-16d @ 3" o/c = 132 (2)(12/3) = 1056 PLF

SILL PL ANCHORAGE : 5/8" φ @ 16" o/c = 1712 (12/16) = 1284 PLF

→ w/ 3x SILL PL

SHEAR WALL 2W2 – 1190 PLF CAPACITY

15/32" CDX SHEATHING w/ 8d @ 2" o/c EA SIDE

SEISMIC : 1280 (2)(1/2)(0.93) = 1190 PLF ← CONTROLS

WIND : 1790 (2)(1/2)(0.93) = 1665 PLF

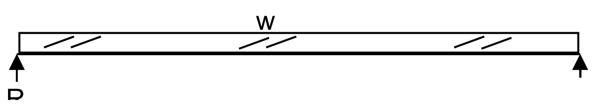
HGA10 CLIPS @ 8" o/c = 840 (12/8) = 1260 PLF

BASE PL NAILING : (2)-16d @ 2" o/c = 132 (2)(12/2) = 1584 PLF

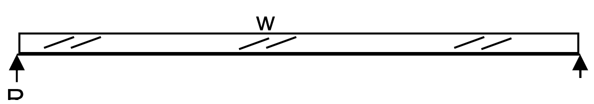
SILL PL ANCHORAGE : 5/8" φ @ 12" o/c = 1712 (1) = 1712 PLF

→ w/ 3x SILL PL

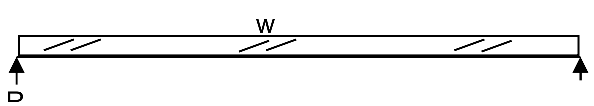
Rf Joist	RJ1		HF	2 x 12	
w=	80	plf	R=	580	lbs
L=	14.5	ft	M=	2,103	ft-lbs
b=	1.50	in	Fb=	797	psi
d=	11.25	in	Fv=	45	psi
E=	1300	ksi	Δ=	0.34	in
Cv=	1.00	≤1.0	l /	506	



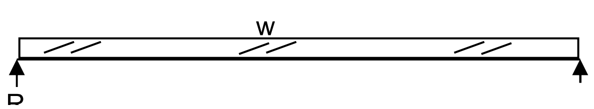
Ridge Beam	RB1		GL	5 1/2 x 15	
w=	380	plf	R=	4,085	lbs
L=	21.5	ft	M=	21,957	ft-lbs
b=	5.50	in	Fb=	1,277	psi
d=	15.00	in	Fv=	66	psi
E=	1800	ksi	Δ=	0.66	in
Cv=	0.97	≤1.0	l /	393	



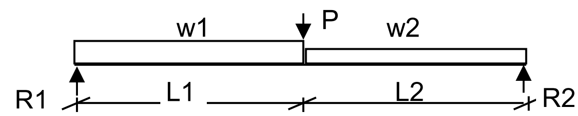
Beam	RB2		GL	5 1/2 x 15	
w=	560	plf	R=	5,880	lbs
L=	21	ft	M=	30,870	ft-lbs
b=	5.50	in	Fb=	1,796	psi
d=	15.00	in	Fv=	94	psi
E=	1800	ksi	Δ=	0.88	in
Cv=	0.97	≤1.0	l /	286	



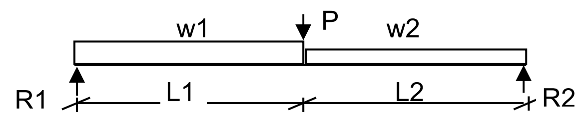
Header	RH1		DF-L	4 x 8	
w=	350	plf	R=	1,050	lbs
L=	6	ft	M=	1,575	ft-lbs
b=	3.50	in	Fb=	616	psi
d=	7.25	in	Fv=	50	psi
E=	1700	ksi	Δ=	0.05	in
Cv=	1.00	≤1.0	l /	1333	



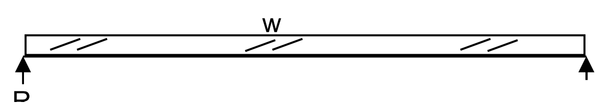
Beam	RB3		GL	5 1/2 x 11 1/4	
w1=	190	plf	R1 =	4,220	lbs
w2=	190	plf	R2 =	2,335	lbs
L1=	3.5	ft	M =	13,607	lb-ft
L2=	9.5	ft	Fb =	1,407	psi
X=	3.5	ft	Fv =	98	psi
P=	4,085	lbs	Δ=	0.25	in
b=	5.50	in	l /	627	
d=	11.25	in	Cv=	1.00	
E=	1,800	ksi			



Beam	RB4		GL	5 1/2 x 11 1/4	
w1=	80	plf	R1 =	4,295	lbs
w2=	80	plf	R2 =	2,721	lbs
L1=	5.2	ft	M =	21,251	lb-ft
L2=	9.0	ft	Fb =	2,198	psi
X=	5.2	ft	Fv =	102	psi
P=	5,880	lbs	Δ=	0.50	in
b=	5.50	in	l /	340	
d=	11.25	in	Cv=	1.00	
E=	1,800	ksi			



Beam	RH2		HF	3 x 10	
w=	400	plf	R=	800	lbs
L=	4	ft	M=	800	ft-lbs
b=	3.00	in	Fb=	224	psi
d=	9.25	in	Fv=	27	psi
E=	1300	ksi	Δ=	0.01	in
Cv=	1.00	≤1.0	l /	5359	



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Project: Fukano

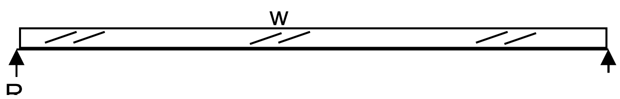
Date: 10/05/20

Project #:

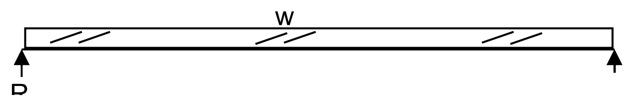
Design: RJA

Sheet:

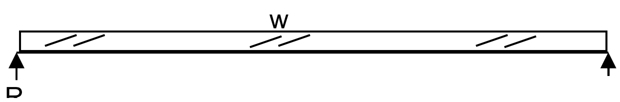
Beam	RH3		HF	4	x 10
w=	600	plf	R=	1,500	lbs
L=	5	ft	M=	1,875	ft-lbs
b=	3.50	in	Fb=	451	psi
d=	9.25	in	Fv=	48	psi
E=	1300	ksi	Δ=	0.03	in
Cv=	1.00	≤1.0	l /	2134	



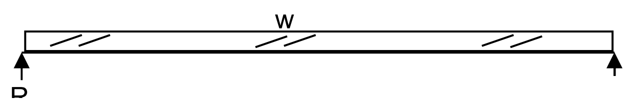
Mezz Bm	MB1		HF	4	x 6
w=	212	plf	R=	318	lbs
L=	3	ft	M=	239	ft-lbs
b=	3.50	in	Fb=	162	psi
d=	5.50	in	Fv=	17	psi
E=	1300	ksi	Δ=	0.01	in
Cv=	1.00	≤1.0	l /	5878	



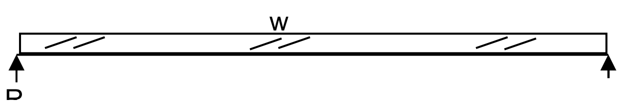
Beam	RH4		GL	3 1/2	x 9
w=	600	plf	R=	2,550	lbs
L=	8.5	ft	M=	5,419	ft-lbs
b=	3.50	in	Fb=	1,376	psi
d=	9.00	in	Fv=	100	psi
E=	1800	ksi	Δ=	0.18	in
Cv=	1.00	≤1.0	l /	554	



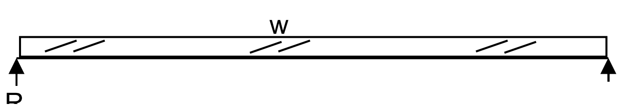
Mezz Bm	MB2		HF	4	x 8
w=	100	plf	R=	300	lbs
L=	6	ft	M=	450	ft-lbs
b=	3.50	in	Fb=	176	psi
d=	7.25	in	Fv=	14	psi
E=	1300	ksi	Δ=	0.02	in
Cv=	1.00	≤1.0	l /	3568	



Beam	RH5		DF-L	6	x 10
w=	360	plf	R=	1,620	lbs
L=	9	ft	M=	3,645	ft-lbs
b=	5.50	in	Fb=	529	psi
d=	9.50	in	Fv=	38	psi
E=	1700	ksi	Δ=	0.08	in
Cv=	1.00	≤1.0	l /	1358	



Floor Joist	FJ1		HF	2	x 10
w=	70	plf	R=	350	lbs
L=	10	ft	M=	875	ft-lbs
b=	1.50	in	Fb=	491	psi
d=	9.25	in	Fv=	32	psi
E=	1300	ksi	Δ=	0.12	in
Cv=	1.00	≤1.0	l /	980	



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Project: Fukano

Date: 10/05/20

Project #:

Design: RJA

Sheet:

Column Buckling Calculations

NDS 2015

Column Geometry Data

Glue-Lam		
-N/A-		
b	5.5	in
d	5.5	in
Le_1	13.00	ft
Le_2	13.00	ft
$le_{bending}$		ft

Bracing

No Brace
No Brace

Column Design Values

F_b	2400	psi
F_c	1650	psi
E'min	930	ksi
Fcperp	0	psi
cb	1.00	

Bearing Area Increase
No

Column Loading

P	6650	lbs
W_1	5	plf
M1	106	ft-lbs
W_2	0	plf
M2	0	ft-lbs

Flexural Stress Adjustment Factors

Roof/EQ / Wind - C_D	1.15
Size Factor - C_F	1.00
Repetitive - C_r	1.00

Compressive Parallel Adjustment Factors

Roof/EQ / Wind - C_D	1.15
Size Factor - C_F	1.00

Other Factors

Visually Graded Lumber	
c	0.8
Solid Column	
Kf	1
Column: Pinned Pinned	
Ke	1

Column Stability Factor Calculation

Strong Axis

Fce_1	950	psi
Fc^*_1	1898	psi
Fce_1/Fc^*_1	0.501	
Cp_1	0.434	

Weak Axis

Fce_2	950	psi
Fc^*_2	1898	psi
Fce_2/Fc^*_2	0.501	
Cp_2	0.434	

Beam Stability Factor Calculation

Strong Axis

Fbe_1	22668	psi
Fb'_1	2760	psi
Fbe_1/Fb'_1	8.2	
le	22.6	ft
CL_1	1.00	

Weak Axis

Fbe_2	19,509	psi
Fb'_2	2760	psi
Fbe_2/Fb'_2	7	

Adjusted Allowable Stresses

Strong Axis

Fc'_1	824	psi
Fb'_1	2760	psi

Weak Axis

Fc'_2	824	psi
Fb'_2	2760	psi

Imposed Column Stresses

Strong Axis

fc_1	220	psi
fb_1	46	psi

Weak Axis

fc_2	220	psi
fb_2	0	psi

Perpendicular to Grain Stress Check fcp/Fcp =	**-N/A-**	**OK**
Slenderness Check le/d	**28**	**OK**
Slenderness Check le/b	**28**	**OK**

(1) $$\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_{b1}}{F_{b1}'\left[1 - f_c/F_{cE1}\right]} + \frac{f_{b2}}{F_{b2}'\left[1 - f_c/F_{cE2} - \left(f_{b1}/F_{bE}\right)\right]} \leq 1.0$$

(2) $$\frac{f_c}{F_{cE2}} + \left(\frac{f_{b1}}{F_{bE}}\right)^2 < 1.0$$

(3) $$\frac{f_c}{F_{c'}}, \frac{f_{b1}}{F_{b1}'}, \frac{f_{b2}}{F_{b2}'} < 1.0$$

Allowable Stress Interaction Formula	**0.27**	**OK**

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Project:	Fukano	*Date:*	*7/2/2020*
		Project #:	
		Design:	RJA
		Sheet:	

Spread Footing Soil Bearing Design

Service Loads Loading

Dead Load =	2.5 kips
Live Load =	4.2 kips
Wind/EQ Load =	0.0 kips
Wind/EQ Moment (M_Y) =	0 ft-kips
Gravity Load Eccentricity (±X)=	0.00 ft.
Footing Weight =	0.6 kips
Total Load =	7.3 kips
Total Moment =	0 ft-kips

Service Load Factors

DL	1
LL	1
EQ/Wind	1

Soil Properties

Allowable Soil Brg. (Qa) =	1.50 ksf
Overburdan Density (γs) =	125 psf
Net Ftg Wt? (γ_c-γ_s)	No

Column Dimensions and Location

Column Xc Dimension (Dx) =	5.50 in.
Column Yc Dimension (Dy) =	5.50 in.
Column Face from right (Cr) =	1.02 ft.
Column Face from left (Cl) =	1.02 ft.

Soil Bearing Check (Allowable)

Eccentricity =	0.00 ft.	
Leng. Soil Brg. Under Ftg.=	2.50 ft.	
qmax =	1.16 ksf	OK
qmin =	1.16 ksf	

Footing Dimensions

L Dimension (X) =	2.50 ft.
B Dimension (Y) =	2.50 ft.
Footing Thickness (t) =	8.00 in.
Ftg Overburden (Ot) =	0.00 ft.

Soil Pressure Equations:

$e \leq L/6$

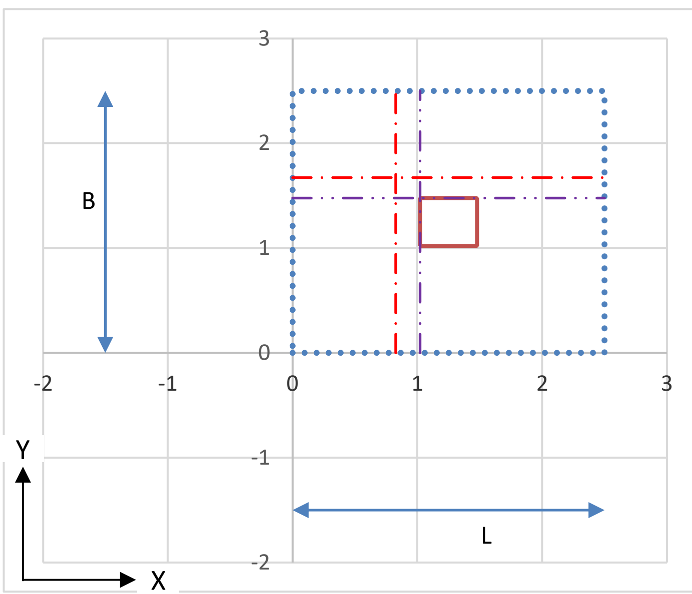
$$qmax = \frac{Q}{LB}\left(1 + \frac{6e}{L}\right)$$

$$qmin = \frac{Q}{LB}\left(1 - \frac{6e}{L}\right)$$

$e > L/6$

$$qmax = \frac{4Q}{3L(L-2e)}$$

$$qmin = 0$$



Spread Footing Concrete Design - ACI 318-14

Footing Properties

Concrete Strength (f'c) =	3000 psi
Rebar Yield Strength (f_y) =	60000 psi
Reinforcing Clear Cover (cvr) =	3.00 in.
Reinforcing Depth (d) =	4.69 in.

Strength Load Factors

DL	1.2
LL	1.6
EQ/Wind	1

Factored Loads

Factored Total Load =	10.4 kips
Factored Total moment =	0 ft-kips

Factored Moments and Shears

	Mu k-ft	Vu kips
X Right Side	2	3
X Left Side	2	3
Y Both Sides	2	3

Factored Bearing

Eccentricity =	0.00 ft.
Length of Soil Brg. Under Ftg. =	2.50 ft.
qmax =	1.66 ksf
qcolr =	1.66 ksf
qcoll =	1.66 ksf
qmin =	1.66 ksf

Flexural Design - X Direction

Bar Size =	#5	
Bars =	3	
Mu =	2 ft-kips	
ØMn =	18 ft-kips	OK
ρ_{min} =	0.0018	*Controls*
ρ_{req} =	0.0007	
A_s Required =	0.25 sq. in.	
A_s Provided =	0.93 sq. in.	OK

Flexural Design - Y Direction

Bar Size =	#5	
Bars =	3	
Mu =	2 ft-kips	
ØMn =	18 ft-kips	OK
ρ_{min} =	0.0018	*Controls*
ρ_{req} =	0.0007	
A_s Required =	0.25 sq. in.	
A_s Provided =	0.93 sq. in.	OK

One-Way Shear Design - X Direction

Vu =	3 kips	
ØVn =	12 kips	OK

One-Way Shear Design - Y Direction

Vu =	3 kips	
ØVn =	12 kips	OK

β =	1.000
$\gamma_s = 2/(\beta+1)$ =	1.00
Provide $A_{s,req}\gamma_s$ =	0.25 sq. in.

Provide evenly distributed bars in each direction.

Two-Way (Punching) Shear Design

b_o =	41 in	
vu =	9 kips	
Øvn =	29 kips	OK

Concrete Capacity Equations:

$$Mn = A_s F_y \left[d - \frac{1}{2}\left(\frac{A_s F_y}{0.85 f'c\, b}\right)\right]$$

$$vn = min\begin{pmatrix} 4\sqrt{f'c} \\ \left(2+\frac{4}{\beta}\right)\sqrt{f'c} \\ \left(2+\frac{\alpha_s d}{b_o}\right)\sqrt{f'c} \end{pmatrix} b_o d$$

$$Vn = 2\,\gamma\sqrt{f'c}\, b_w d$$

$$b_o = 2(Dx+d) + 2(Dy+d)$$

$$\beta = \max(Dx, Dy)/\min(Dx, Dy)$$

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ASCE 7-10 Wind Loads - Components and Cladding
Flat, Gable and Hip Roofs

Part 1: Low-Rise Buildings (h ≤ 60 feet)
Section 30.4

Wind Coefficients

Exposure	B	
V=	110	mph
K_d=	0.85	Table 26.6-1
GC_p =	(Calculated from Ch. 30 Tables)	
GC_{pi} =	0.18	Table 26.11-1

Location and Building Dimensions

K_{zt} =	1.60	
Mean Roof Height, h	13	ft
K_z =	0.70	Table 30.3-1
Roof Angle	22	degrees

Velocity Pressure, $q_h = 0.00256 K_d K_{zt} K_z V^2$ 17.7 psf (30.3-1)

Design Wind Pressure, $p = q_h[(GC_P)-(GC_{Pi})]$ (30.4-1)

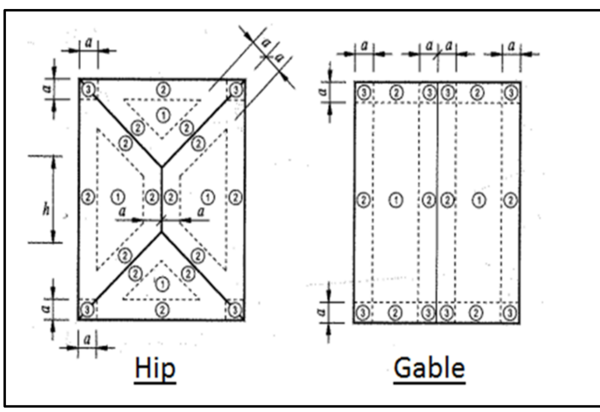
Design	ASD

Roof Type	Gable

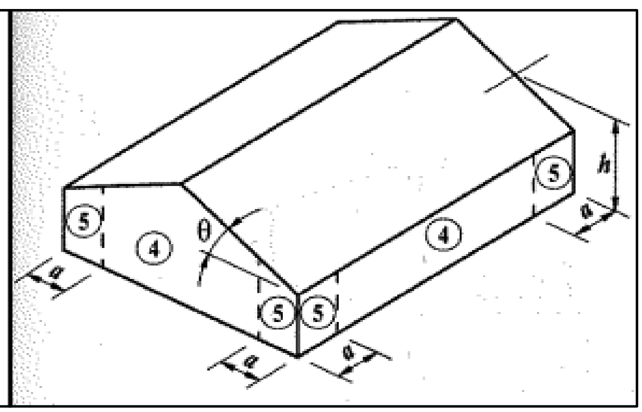
Design Wind Pressure (psf)

Component	Zone		Effective Wind Area (sq ft) ≤10	20	50	100	200	≥500
Gable and Hip Roofs 7 to 27 deg	1	+	12.0*	11.2*	9.6*	8.5*	8.5*	8.5*
		-	-19.1	-18.6	-17.7	-17.4	-17.4	-17.4
	2	+	12.0*	11.2*	9.6*	8.5*	8.5*	8.5*
		-	-33.3	-30.6	-26.9	-24.4	-24.4	-24.4
		OH	-42.1	-42.1	-42.1	-42.1	-42.1	-42.1
	3	+	12.0*	11.2*	9.6*	8.5*	8.5*	8.5*
		-	-49.2	-45.7	-41.8	-38.6	-38.6	-38.6
		OH	-68.7	-61.6	-52.8	-47.5	-47.5	-47.5
Wall	4	+	20.9	20.0	18.8	17.7	16.8	15.6*
		-	-22.7	-21.6	-20.5	-19.3	-18.6	-17.4
	5	+	20.9	20.0	18.8	17.7	16.8	15.6*
		-	-28.0	-26.2	-23.7	-21.8	-19.7	-17.4
Parapet (Fig. 30.7-1)	Typ - LC A	+	54.2	50.6	45.7	42.1	41.3	40.0
	Typ - LC B	-	-43.6	-41.6	-39.3	-37.0	-35.4	-32.9
	Corner - LC A	+	70.1	65.7	60.6	56.3	55.4	54.2
	Corner - LC B	-	-48.9	-46.2	-42.5	-39.5	-36.5	-32.9

Note: * Indicates 10psf minimum lateral load per 30.2.2 controls this load case for most buildings.



Gable/Hip Roofs 7 -27 degree - Figure 30.4-2B



Wall Zones - Figure 30.4-1

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

h: Mean roof height, in feet (meters), except that eave height shall be used for $\theta \leq 10°$.

Fukano		Date:	7/1/2020
		Project #:	
		Design:	RJA

Column Buckling Calculations

NDS 2015

Column Geometry Data

Hem-Fir #2 Studs		
Hem-Fir Plates		
b	1.5	in
d	5.5	in
Le_1	14.50	ft
Le_2	14.50	ft
$le_{bending}$		ft

Bracing

No Brace
Braced

Column Stability Factor Calculation

Strong Axis

Fce_1	324633	psi
Fc^*_1	2288	psi
Fce_1/Fc^*_1	141.885	
Cp_1	1.000	

Weak Axis

Fce_2	29	psi
Fc^*_2	2288	psi
Fce_2/Fc^*_2	0.013	
Cp_2	0.013	

Beam Stability Factor Calculation

Strong Axis

Fbe_1	769	psi
Fb'_1	2033	psi
Fbe_1/Fb'_1	1.0	
le	25.0	ft
CL_1	0.82	

Weak Axis

Fbe_2	36,315	psi
Fb'_2	2033	psi
Fbe_2/Fb'_2	18	

Column Design Values

F_b	850	psi
F_c	1300	psi
E'min	470	ksi
Fcperp	405	psi
cb	1.00	

Bearing Area Increase
No

Adjusted Allowable Stresses

Strong Axis

Fc'_1	2288	psi
Fb'_1	1662	psi

Weak Axis

Fc'_2	29	psi
Fb'_2	2033	psi

Column Loading

P	160	lbs
W_1	30	plf
M1 (Braced)	0	ft-lbs
W_2	0	plf
M2	0	ft-lbs

Imposed Column Stresses

Strong Axis

fc_1	19	psi
fb_1	0	psi

Weak Axis

fc_2	19	psi
fb_2	0	psi

Flexural Stress Adjustment Factors

Roof/EQ / Wind - C_D	1.60
Size Factor - C_F	1.30
Repetitive - C_r	1.15

Perpendicular to Grain Stress Check fcp/Fcp =	***19 / 405***	**OK**
Slenderness Check le/d	**32**	**OK**
Slenderness Check le/b	**116**	**NG**

Compressive Parallel Adjustment Factors

Roof/EQ / Wind - C_D	1.60
Size Factor - C_F	1.10

(1) $$\left(\frac{f_c}{F_c'}\right)^2+\frac{f_{b1}}{F_{b1}'\left[1-f_c/F_{cE1}\right]}+\frac{f_{b2}}{F_{b2}'\left[1-f_c/F_{cE2}-\left(f_{b1}/F_{bE}\right)\right]}\le 1.0$$

Other Factors

Visually Graded Lumber	
c	0.8
Solid Column	
Kf	1
Column: Pinned Pinned	
Ke	1

(2) $$\frac{f_c}{F_{cE2}}+\left(\frac{f_{b1}}{F_{bE}}\right)^2<1.0$$

(3) $$\frac{f_c}{F_{c'}},\frac{f_{b1}}{F_{b1}'},\frac{f_{b2}}{F_{b2}'}<1.0$$

Allowable Stress Interaction Formula	**0.68**	**OK**

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Project:	Fukano	*Date:*	*7/2/2020*
		Project #:	
		Design:	RJA
		Sheet:	

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 0.875
Effective Embedment depth, h_{ef} (inch): 24.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 26.38
C_{min} (inch): 1.75
S_{min} (inch): 3.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 32.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: No

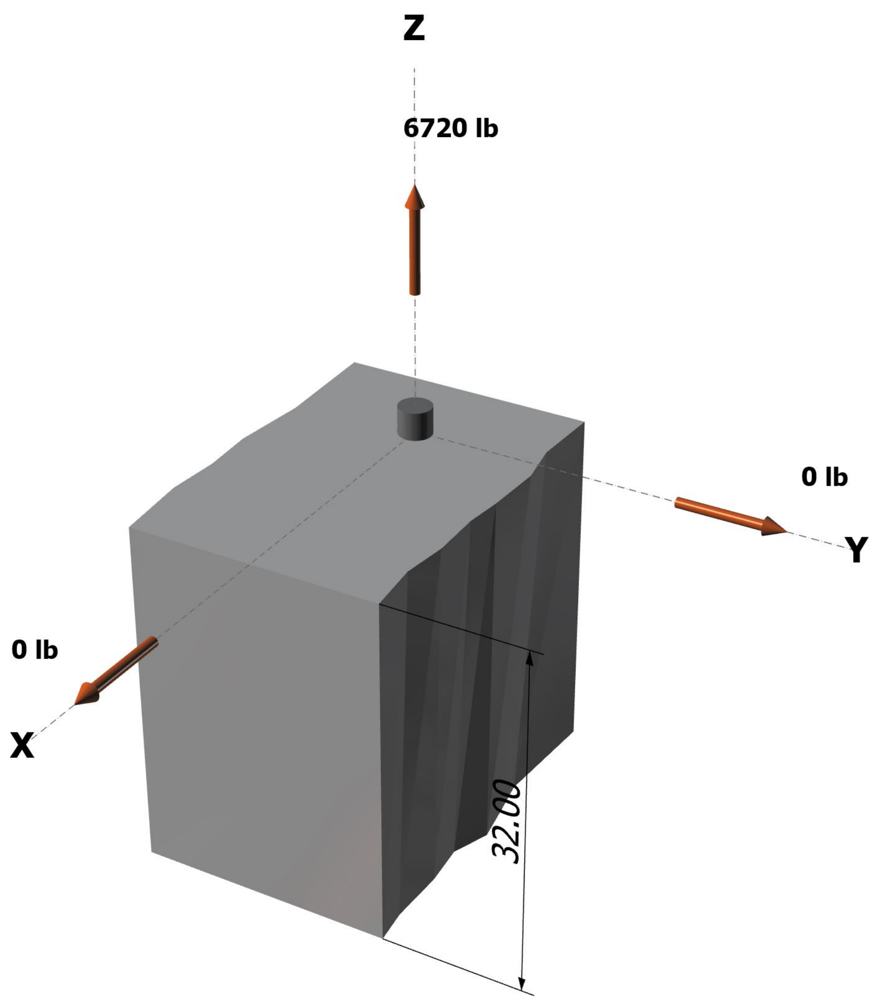
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB7 (7/8"Ø)

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.3 (c) is satisfied
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

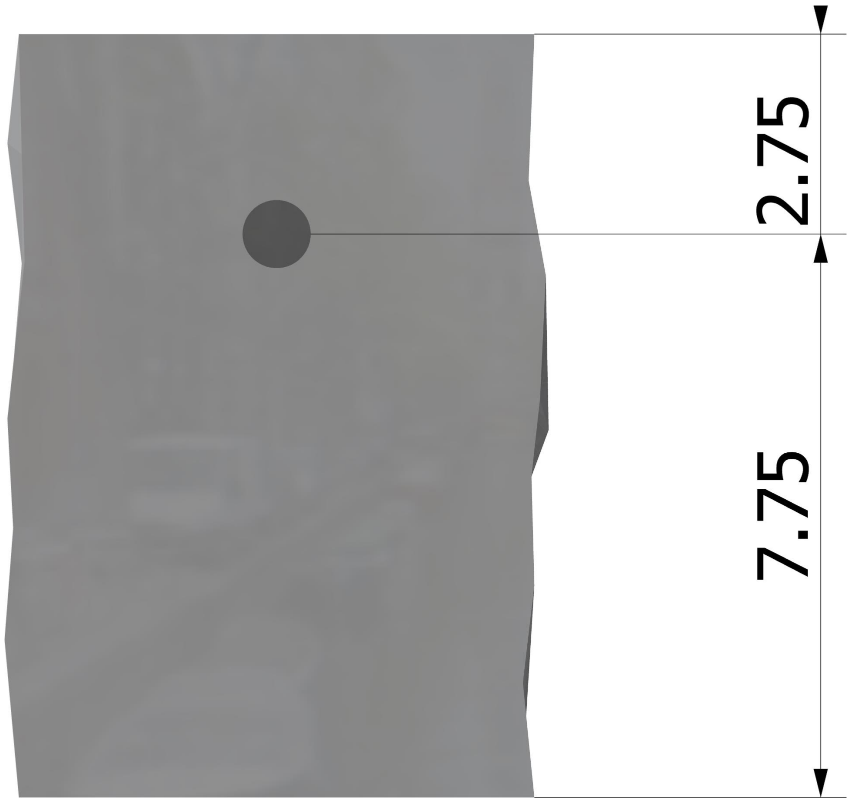
Strength level loads:

N_{ua} [lb]: 6720
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	6720.0	0.0	0.0	0.0
Sum	6720.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 6720
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
26795	0.75	20096

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = 16\lambda_a\sqrt{f'_c}h_{ef}^{5/3}$ (Eq. 17.4.2.2b)

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	2500	24.000	159750

$0.75\phi N_{cb} = 0.75\phi\,(A_{Nc}\,/\,A_{Nco})\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
781.59	5184.00	2.75	0.723	1.00	1.000	159750	0.70	9141

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}N_p = 0.75\phi\Psi_{c,P}8A_{brg}f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	4.07	2500	0.70	42683

7. Side-Face Blowout Strength of Anchor in Tension (Sec. 17.4.4)

$0.75\phi N_{sb} = 0.75\phi\{(1+c_{a2}/c_{a1})/4\}(160c_{a1}\sqrt{A_{brg}})\lambda\sqrt{f'_c}$ (Sec. 17.3.1 & Eq. 17.4.4.1)

c_{a1} (in)	c_{a2} (in)	A_{brg} (in^2)	λ_a	f'_c (psi)	ϕ	$0.75\phi N_{sbg}$ (lb)
2.75	99999.00	4.07	1.00	2500	0.70	23287

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	6720	20096	0.33	Pass
Concrete breakout	**6720**	**9141**	**0.74**	**Pass (Governs)**
Pullout	6720	42683	0.16	Pass
Side-face blowout	6720	23287	0.29	Pass

PAB7 (7/8"Ø) with hef = 24.000 inch meets the selected design criteria.

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 12.000
Code report: IAPMO UES ER-263
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 13.25
c_{ac} (inch): 21.40
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 27.00
State: Cracked
Compressive strength, f'_c (psi): 3000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 110/75°F
Ignore 6do requirement: Not applicable
Build-up grout pad: No

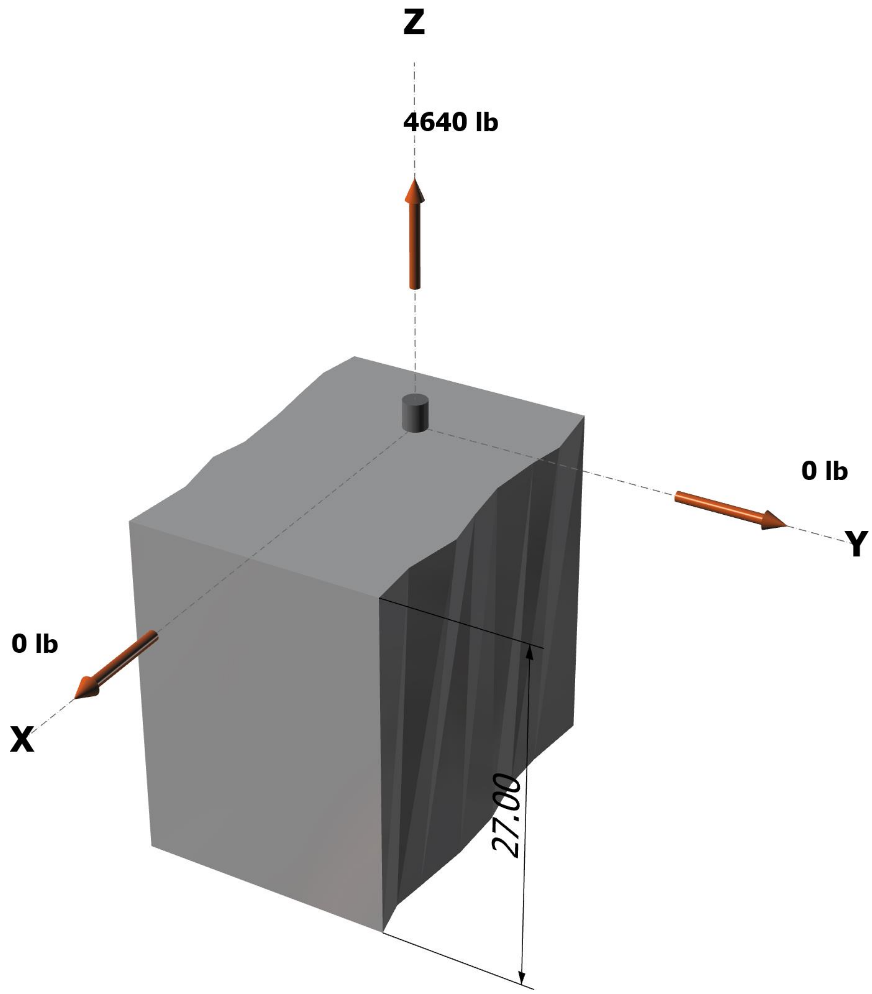
Recommended Anchor
Anchor Name: AT-XP® - AT-XP w/ 5/8"Ø F1554 Gr. 36
Code Report: IAPMO UES ER-263

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: No
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

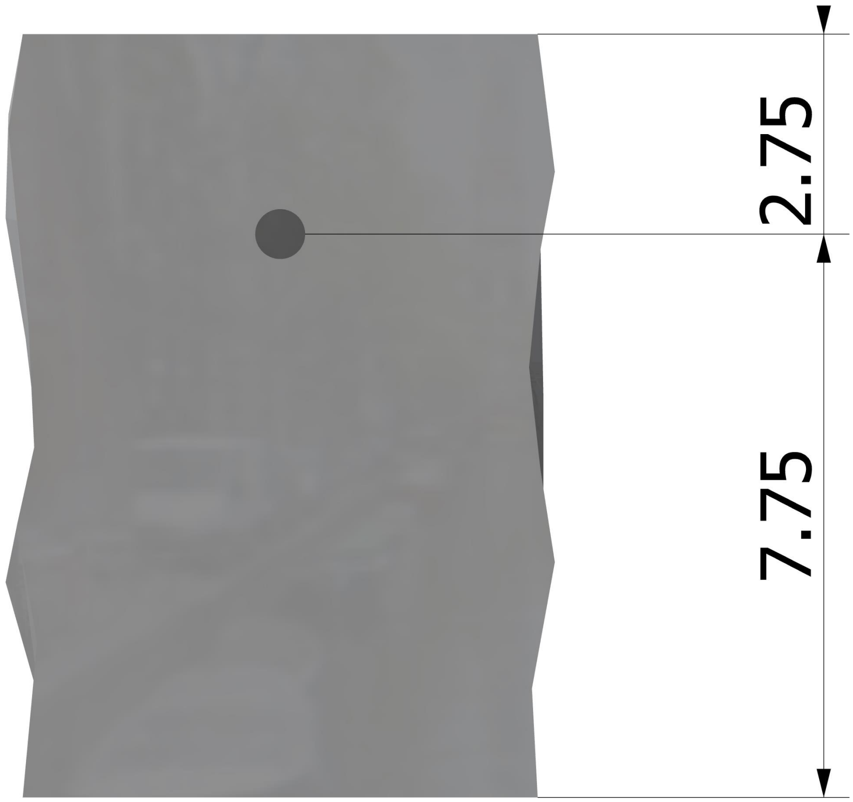
Strength level loads:

N_{ua} [lb]: 4640
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	4640.0	0.0	0.0	0.0
Sum	4640.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 4640
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13110	0.75	9833

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	3000	12.000	38706

$\phi N_{cb} = \phi\,(A_{Nc} / A_{Nco})\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
378.00	1296.00	2.75	0.746	1.00	1.000	38706	0.65	5473

6. Adhesive Strength of Anchor in Tension (Sec. 17.4.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\tau_{k,cr}$ (psi)
980	1.00	1.00	980

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.4.5.2)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	980	0.63	12.000	23091

$\phi N_a = \phi\,(A_{Na} / A_{Na0})\,\Psi_{ed,Na}\,\Psi_{cp,Na} N_{ba}$ (Sec. 17.3.1 & Eq. 17.4.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	ϕ	ϕN_a (lb)
163.88	243.61	7.80	2.75	0.806	1.000	23091	0.65	8135

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	4640	9833	0.47	Pass
Concrete breakout	**4640**	**5473**	**0.85**	**Pass (Governs)**
Adhesive	4640	8135	0.57	Pass

AT-XP w/ 5/8"Ø F1554 Gr. 36 with hef = 12.000 inch meets the selected design criteria.

12. Warnings

- This temperature range is currently outside the scope of ACI 318-14/-11 and ACI 355.4. Designer must exercise judgement to determine if this design is suitable.

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer’s product literature for hole cleaning and installation instructions.