Structural Calculations For:

Grima-Fahrer Residence

4008 90th Avenue SE

Mercer Island, WA 98040

6-9-23

Prepared for:	Board & Vellum
Job #:	10539-2023-03
Date:	June 9, 2023

Criteria Sheet

Codes

Structural	IBC 2018
Loading	ASCE 7-16
Wood:	NDS 2018
Steel:	AISC 360-16
Concrete:	ACI 318-14
Masonry:	TMS 402/602-16

Project Location

Street & Number	4008 90th Ave SE
City:	Mercer Island
State:	WA
ZIP:	98040
Latitude:	47.5743 N
Longitude:	-122.2185 W
Ground Elevation	313 ft

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 =	23 kips	Ω_o=	2.5
S_S=	1.409	S_1=	0.49
S_{DS}=	1.00	S_{D1}=	0.59
C_s=	0.154	I_E=	1.0

Story Information

# Stories Above Grade (Including Mezzanine Levels)	2

Horizontal and Vertical Irregularities:

Is the building a "Regular Structure"? (No horizontal or vertical irregularities): Yes

Wind Load Summary:

V=	98	K_{zt}=	1.00
Exposure =	B		

Dead Loads:

Patio Roof			Floor		
Roofing	1	psf	Finish Floor	2	psf
1/2" Sheathing	1.8	psf	3/4" Sheathing	2.7	psf
2x Rafters	2.5	psf	Joists @ 16" oc	2.2	psf
2x Ceiling Joists	2.5	psf	Misc./Mech.	2	psf
Misc./Mech.	1.5	psf	Ceiling Finish	2.8	
Ceiling Finish	2.8	psf		11.7	psf
Solar Panels	5	psf	Use	12	psf
	17	psf			
Use	17	psf			

Live Loads:

Roof	20	psf
Floor	40	psf

Snow Loading Criteria:

Ground Snow, p_g	25 psf	Flat Roof Snow Load, p_f	25.0 psf
Exposure Factor, C_e	1.00	Sloped Roof Snow Load, p_s	25.0 psf
Thermal Factor, C_t	1.20		
Importance Factor, I_s	1.00		
Slope Factor, C_s	0.73		

Soils:

Soils Report Provided? No — To be approved by the authority having jurisdiction, per 11.8.2 exception.

Allowable Bearing: 2000 psf

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Seismic Design

ASCE 7-16 Seismic Analysis | Equivalent Lateral Force Procedure | Apply Section 12.8.1.3 (Where Applicable)? Yes

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 (Default)	Assumed default soil properties, per 11.4.3.
Diaphragm Flexibility	Flexible	

Section 12.8.1.3

1. Regular Structure	Yes
2. ≤ 5 Stories above grade	Yes
3. T ≤ 0.5s	Yes
4. ρ = 1.0	Yes
5. Not Site Class E or F	Yes
6. Risk Category I or II	Yes

If all items above are met, S_{DS} may be taken as 1, but not less than 0.7*(Calculated S_{DS})

S_S	1.409 g	2% in 50 yr, Latitude & Longitude lookup
S_1	0.49 g	2% in 50 yr, Latitude & Longitude lookup
R	6.50	
C_d	4.0	
Ω_o	2.5	
I_e	1.00	Table 1.5-2
h_n	25.0 ft	
Ct	0.02	Table 12.8-2
x	0.75	Table 12.8-2
T_a	0.22 sec	
T	0.22 sec	Eq. 12.8-7
T_0	0.12 sec	
T_S	0.59 sec	
T_L	6.00 sec	
F_a	1.20	Table 11.4-1
F_v	1.81	Table 11.4-2
S_{MS}	1.69 g	Eq. 11.4-1
S_{M1}	0.89 g	Eq. 11.4-2
S_{DS}	1.000 g	Eq. 11.4-3
S_{D1}	0.591 g	Eq. 11.4-4
C_s	***0.154 Controls***	Eq. 12.8-2
	0.407	Eq. 12.8-3 need not exceed, T < T_L
	0.010	Eq. 12.8-5 or 12.8-6 minimum
C_s, design	0.154	Section 11.4.8 Exception 2 Applied
Bldg. Weight	152.1 k	
V = C_SW	23.4 k	Eq. 12.8-1, Strength Level Base Shear
V = C_{Sasd}W	16.4 k	Eq. 12.8-1 ASD Base Shear

Building Period Per Alternate Analysis

T (sec)	

Per Geotech Report

F_a	
F_v	

$$T_a = C_t h_n^x \quad \text{Eq. 12.8.7}$$

$$S_{MS} = F_a S_S \quad \text{Eq. 11.4-1}$$
$$S_{M1} = F_v S_1 \quad \text{Eq. 11.4-2}$$
$$S_{DS} = {}^2/_3 S_{MS} \quad \text{Eq. 11.4-3}$$
$$S_{D1} = {}^2/_3 S_{M1} \quad \text{Eq. 11.4-4}$$

$$C_S = \frac{S_{DS}}{(R/I_e)} \quad \text{Eq. 12.8-2}$$
$$C_S = \frac{S_{D1}}{T(R/I_e)} \quad \text{Eq. 12.8-3}$$
$$C_S = \frac{S_{D1} T_L}{T^2(R/I_e)} \quad \text{Eq. 12.8-4}$$
$$C_S \geq 0.044 S_{DS} I_e \quad \text{Eq. 12.8-5}$$
$$C_S \geq 0.01 \quad \text{Eq. 12.8-5}$$

$$C_{VX} = w_x h_x^k / \sum_{i=1}^{n} w_x h_i^k \quad \text{Eq. 12.8-12}$$

$$F_{px} = {}^{\sum_{i=x}^{n} F_i}/_{\sum_{i=x}^{n} w_i} w_{px} \quad \text{Eq. 12.10-1}$$
$$F_{px} \geq 0.2 S_{DS} I_e w_{px} \quad \text{Eq. 12.10-2}$$
$$F_{px} \leq 0.4 S_{DS} I_e w_{px} \quad \text{Eq. 12.10-3}$$

Vertical Distribution ASD ρ= 1 k= 1.000

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)	$F_{px,calc}$	$F_{px,min}$	$F_{px,max}$	$F_{px,design}$	γ=F_{px}/F_x
ROOF	25.0	64	25.0	1600	0.623	10.2	**10.2**	10.2	9.0	17.9	**10.2**	1.00
2 FLOOR	11.0	67.5	11.0	743	0.289	4.7	**14.9**	7.7	9.5	18.9	**9.5**	2.00
PATIO	11.0	21	11.0	227	0.088	1.4	**16.4**	2.2	2.9	5.8	**2.9**	2.00
Σ		152.1		2569		**16.4**						

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PROPOSED SEISMIC:

DEAD LOADS:

ROOF = 15 PSF + 5 PSF WALLS = 20 PSF

FLOOR = 12 PSF + 10 PSF WALLS = 22 PSF

PATIO ROOF = 20 PSF (INCL. CEILING FRAMING)

AREAS:

ROOF = 3200 SF

SECOND FLOOR = 3070 SF

PATIO ROOF = 1030 SF

SEISMIC WEIGHTS:

ROOF = (3200 SF)(20 PSF) = 64.0 k } EXISTING

SECOND FLOOR = (3070 SF)(22 PSF) = 67.5 k } EXISTING

PATIO ROOF = (1030 SF)(20 PSF) = 20.6 k } NEW

BASE SHEAR: $C_S = 0.154$

EXISTING $V_{ASD} = 0.7(64.0 + 67.5\,k)(0.154) = 14.45\,k$

NEW $V_{ASD} = 0.7(20.5\,k)(0.154) = 2.21\,k$

AREA BASE SHEAR:

EXISTING $V_A = \frac{14.45\,k}{3070\,SF} = 4.7\,PSF$

NEW $V_A = \frac{2.21\,k}{1030\,SF} = 2.1\,PSF$

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SHEARWALL DESIGN — NORTH-SOUTH LOADING

WEST EXTERIOR WALL:

- $A_{TRIB,EXISTING} = 615$ SF $A_{TRIB,NEW} = 1030$ SF (ALL)
- $V = (615\ SF)(4.7\ PSF) + 2.21k = 5.10k$
- CAPACITY OF EXISTING SHEARWALLS = 242 PLF MIN.
  - ⇒ FROM ORIGINAL STRUCTURAL SET, 7/16 OSB OR 15/32 PLY w/ 8d @ 6"oc EDGE NAILING

$L_{PROPOSED} = 21'11"$

$V = 5.10k / 21'11" = 233$ PLF

$OT = 2.33k$

⇒ OT_R : (7'6" FLOOR) + (20' ROOF) = 390 PLF

FOR LOAD CASE 10: $E_v = 0.2(1.0)(D) = 0.2D$

⇒ $0.6D - 0.7E_v = 0.46D$

FOR 4'1" NORTHERNMOST SEGMENT: (HDR END REACTION)

$OT_{NET} = 2.33k - 0.46\left[(390\ PLF)(2'0") + 4.7k\right]$

$= -200$ LBF ⇒ NO NET UPLIFT (SHORTEST SEGMENT)

- EXISTING LATERAL CAPACITY IS ADEQUATE FOR NEW DEMAND
- NO NET UPLIFT AT WALL SEGMENTS SHORTENED BY NEW OPENINGS

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SHEARWALL DESIGN: EAST-WEST LOADING

NORTH WALL SEGMENT: h = 8'6"

A_{TRIB} = 409 SF

V = (409 SF)(2.1 PSF) = 860 LBF

L = 2'7"

v = 333 PLF

OT = 2.83k

SHEAR WALL: W4
HOLD DOWN: HDU4

SOUTH INT. WALL SEGMENTS: h = 11'0"

A_{TRIB} = 621 SF

V = (621 SF)(2.1 PSF) = 1.30 k

⇒ PORTAL FRAME PER APA TT-100

⇒ AT 9'2" x 16", PIER CAPACITY = 667 LBF
(INTERPOLATED, HF FRAMING)

2(667 LBF) = 1.33 k ⇒ OKAY

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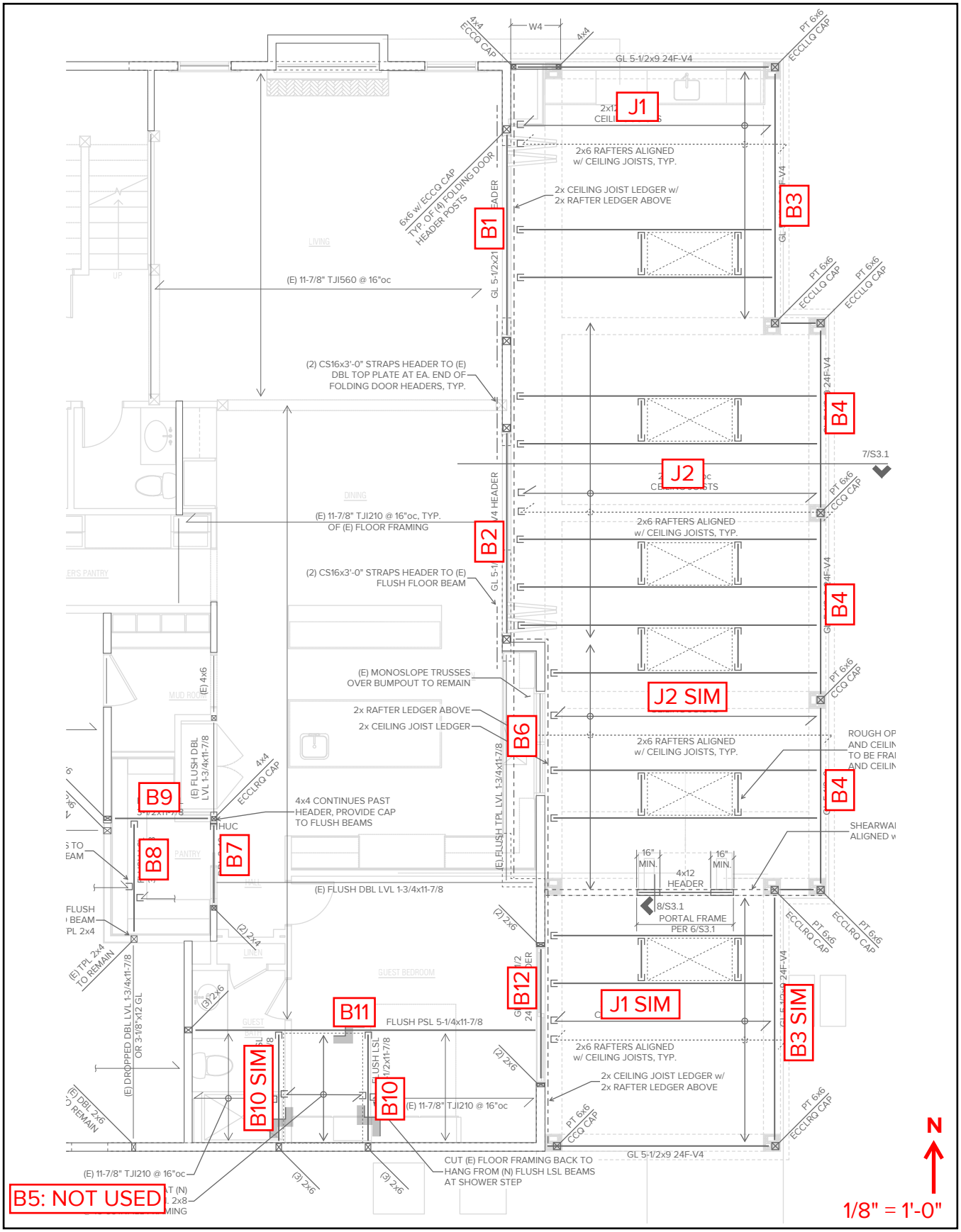

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Gravity Key Plan

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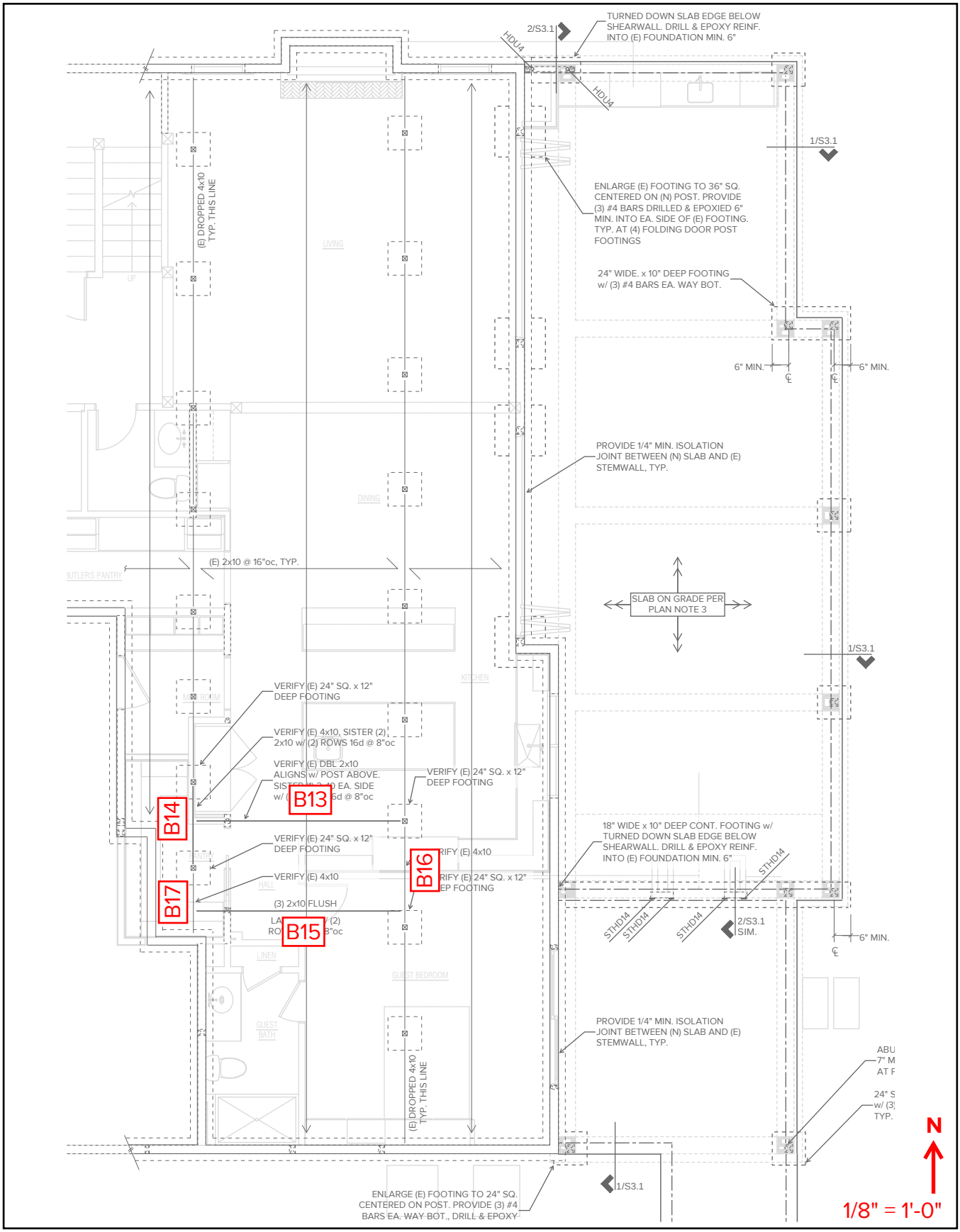

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Beam Analysis

B1

Beam:	BiFold 101 Header						
Load		Dead	Live	Snow	Seismic	Factored	Location
Distributed (k/ft)	w_1	**0.360**		**0.600**		0.810	Roof
	w_2	**0.132**	**0.440**			0.462	2nd Flr
	w_3	**0.136**		**0.200**		0.286	Patio R
	w_4	**0.123**				0.123	Wall
	w_5					0.000	
	w_6					0.000	
	w_7					0.000	
	w_8					0.000	
	w_9					0.000	
	w_{10}	**0.03128**				0.031	S.W.
Trapezoidal (k/ft/ft)	t_1					0.000	
	t_2					0.000	
	t_3					0.000	
	t_4					0.000	
	t_5					0.000	
	t_6					0.000	
Point (k)	P_1					0.000	
	P_2					0.000	
	P_3					0.000	
	P_4					0.000	
	P_5					0.000	
	P_6					0.000	
	P_7					0.000	
	P_8					0.000	
	P_9					0.000	
	P_{10}					0.000	

Support Locations and Reactions		
# of Supports		2
Total Beam Length		**12.25**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	10.485	**0.00**
R_2	10.485	**12.25**
R_3	0.000	**12.25**
R_4	0.000	**12.25**
R_5	0.000	**12.25**
R_6	0.000	**12.25**
R_7	0.000	**12.25**
R_8	0.000	**12.25**
R_9	0.000	**12.25**
R_{10}	0.000	**12.25**

Demand Output		
Location, ft		**6.13**
Shear, k	V	0.00
Moment, k-ft	M =	32.11
Deflection, in	D =	-0.114
Δ/Span		L/1295

Load Factors	
Dead	**1.00**
Live	**0.75**
Snow	**0.75**
Seismic	**1.00**

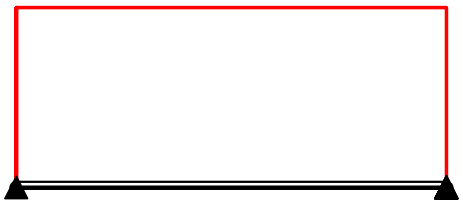
Stresses @ Input Location	
f_v (psi)	0
f_b (psi)	953

Max/Min Stresses	
f_{v_MAX} (psi)	136
f_{v_MIN} (psi)	-136
f_{b_MAX} (psi)	953
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	**1800**
b (in)	**5.5**
d (in)	**21**
I (in^4)	4244.6
S (in^3)	404.25
A (in^2)	115.5
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	NONE

Beam Loading Diagram



Span	V_L (kips)	V_R (kips)	M(-) (k-ft)	M(+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	10.5	-10.5	-	32.1	-0.134 (↓)	6.1	L/1097	-0.029 (↓)	6.1	L/5069

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Beam Analysis

B2

Beam:		BiFold 102 Header						
Load		Dead	Live	Snow	Seismic	Factored	Location	
Distributed (k/ft)	w_1	**0.360**		**0.600**		0.810		Roof
	w_2	**0.120**	**0.400**			0.420		2nd Flr
	w_3	**0.157**		**0.231**		0.331		Patio R
	w_4	**0.123**				0.123		Wall
	w_5					0.000		
	w_6					0.000		
	w_7					0.000		
	w_8					0.000		
	w_9					0.000		
	w_{10}	**0.03128**				0.031		S.W.
Trapezoidal (k/ft/ft)	t_1					0.000		
	t_2					0.000		
	t_3					0.000		
	t_4					0.000		
	t_5					0.000		
	t_6					0.000		
Point (k)	P_1					0.000		
	P_2					0.000		
	P_3					0.000		
	P_4					0.000		
	P_5					0.000		
	P_6					0.000		
	P_7					0.000		
	P_8					0.000		
	P_9					0.000		
	P_{10}					0.000		

Support Locations and Reactions		
# of Supports		2
Total Beam Length		**12.25**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	10.501	**0.00**
R_2	10.501	**12.25**
R_3	0.000	**12.25**
R_4	0.000	**12.25**
R_5	0.000	**12.25**
R_6	0.000	**12.25**
R_7	0.000	**12.25**
R_8	0.000	**12.25**
R_9	0.000	**12.25**
R_{10}	0.000	**12.25**

Demand Output		
Location, ft		**6.13**
Shear, k	V	0.00
Moment, k-ft	M =	32.16
Deflection, in	D =	-0.114
Δ/Span		L/1293

Load Factors	
Dead	**1.00**
Live	**0.75**
Snow	**0.75**
Seismic	**1.00**

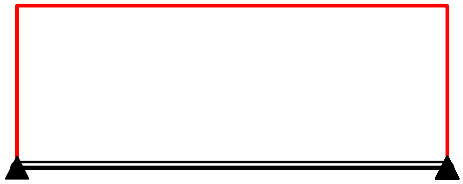
Stresses @ Input Location	
f_v (psi)	0
f_b (psi)	955

Max/Min Stresses	
f_{v_MAX} (psi)	136
f_{v_MIN} (psi)	-136
f_{b_MAX} (psi)	956
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	**1800**
b (in)	**5.5**
d (in)	**21**
I (in^4)	4244.6
S (in^3)	404.25
A (in^2)	115.5
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	NONE

Beam Loading Diagram



Span	V_L (kips)	V_R (kips)	M(-) (k-ft)	M(+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	10.5	-10.5	-	32.2	-0.134 (↓)	6.1	L/1097	-0.027 (↓)	6.1	L/5444

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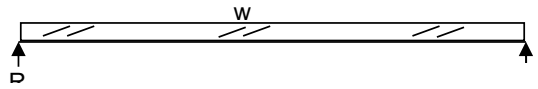
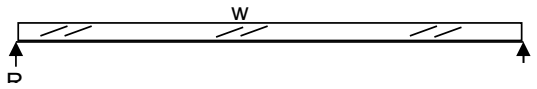
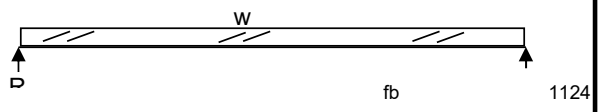
Ceiling Joist	J1		HF	2	x 12
w=	84	plf	R=	665	lbs
L=	15.83	ft	M=	2,631	ft-lbs
b=	1.50	in	Fb=	998	psi
d=	11.25	in	Fv=	52	psi
E=	1300	ksi	Δ=	0.51	in
Cv=	1.00	≤1.0	I /	370	



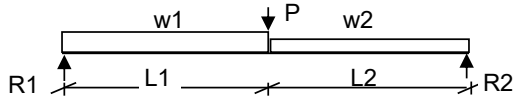
(E) Kitchen Header	B6		HF	4	x 8
w=	347	plf	R=	1,083	lbs
L=	6.25	ft	M=	1,692	ft-lbs
b=	3.50	in	Fb=	662	psi
d=	7.25	in	Fv=	52	psi
E=	1300	ksi	Δ=	0.08	in
Cv=	1.00	≤1.0	I /	911	



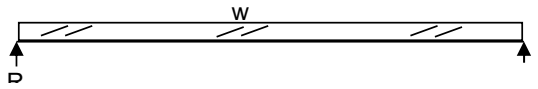
Ceiling Joist	J2		HF	2	x 12
w=	56	plf	R=	518	lbs
L=	18.5	ft	M=	2,396	ft-lbs
b=	1.50	in	Fb=	909	psi
d=	11.25	in	Fv=	41	psi
E=	1300	ksi	Δ=	0.64	in
Cv=	1.00	≤1.0	I /	348	



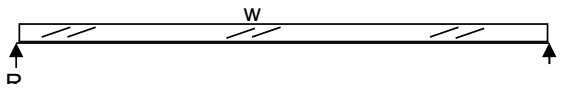
Pantry Pocket Header	B7		HF	3	x 10
w1=	624	plf	R1 =	2,233	lbs
w2=	624	plf	R2 =	1,876	lbs
L1=	2	ft	M =	2,709	lb-ft
L2=	4	ft	Fb =	760	psi
X=	1.5	ft	Fv =	95	psi
P=	833	lbs	Δ=	0.04	in
b=	3.00	in	I /	1,437	
d=	9.25	in	Cv=	1.00	
E=	1,300	ksi			



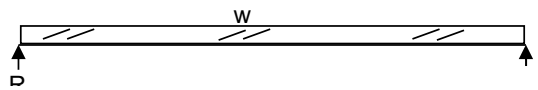
Perimeter Beam	B3		GL	5 1/2	x 9
w=	347	plf	R=	2,656	lbs
L=	15.33	ft	M=	10,179	ft-lbs
b=	5.50	in	Fb=	1,645	psi
d=	9.00	in	Fv=	73	psi
E=	1800	ksi	Δ=	0.72	in
Cv=	1.00	≤1.0	I /	257	



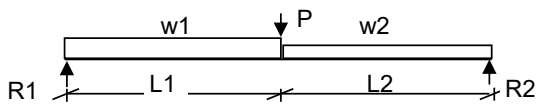
Pantry Flush Header	B8		LSL	3 1/2	x 11 7/8
w=	650	plf	R=	2,194	lbs
L=	6.75	ft	M=	3,702	ft-lbs
b=	3.50	in	Fb=	540	psi
d=	11.88	in	Fv=	56	psi
E=	1550	ksi	Δ=	0.04	in
Cv=	1.00	≤1.0	I /	2020	



Perimeter Beam	B4		GL	5 1/2	x 9
w=	399	plf	R=	2,195	lbs
L=	11	ft	M=	6,035	ft-lbs
b=	5.50	in	Fb=	975	psi
d=	9.00	in	Fv=	57	psi
E=	1800	ksi	Δ=	0.22	in
Cv=	1.00	≤1.0	I /	604	



Pantry Flush Transfer	B9		LSL	3 1/2	x 11 7/8
w1=	69	plf	R1 =	1,855	lbs
w2=	69	plf	R2 =	772	lbs
L1=	2	ft	M =	2,850	lb-ft
L2=	5	ft	Fb =	416	psi
X=	1.6	ft	Fv =	64	psi
P=	2,194	lbs	Δ=	0.02	in
b=	3.50	in	I /	4,452	
d=	11.88	in	Cv=	1.00	
E=	1,550	ksi			



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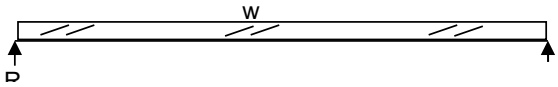
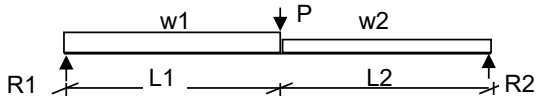
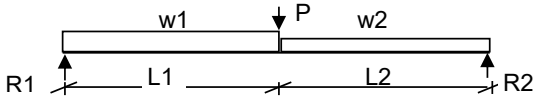
Shower Step Beam	B10		LSL	3 1/2 x 11 7/8	
w=	397	plf	R=	1,338	lbs
L=	6.75	ft	M=	2,258	ft-lbs
b=	3.50	in	Fb=	329	psi
d=	11.88	in	Fv=	34	psi
E=	1550	ksi	Δ=	0.02	in
Cv=	1.00	≤1.0	l /	3311	



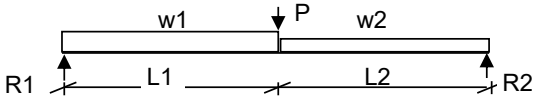
Shower Floor Beam	B11		PSL	5 1/4 x 11 7/8	
w1=	69	plf	R1 =	1,810	lbs
w2=	69	plf	R2 =	1,373	lbs
L1=	8	ft	M =	11,949	lb-ft
L2=	13	ft	Fb =	1,162	psi
X=	7.8	ft	Fv =	42	psi
P=	1,750	lbs	Δ=	0.47	in
b=	5.25	in	l /	531	
d=	11.88	in	Cv=	1.00	
E=	2,200	ksi			



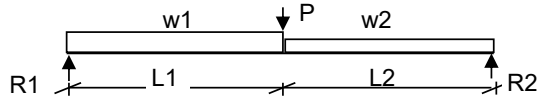
Upgraded Header	B12		GL	3 1/2 x 10 1/2	
w1=	540	plf	R1 =	3,510	lbs
w2=	813	plf	R2 =	3,750	lbs
L1=	3	ft	M =	8,557	lb-ft
L2=	5	ft	Fb =	1,597	psi
X=	3.3	ft	Fv =	124	psi
P=	1,373	lbs	Δ=	0.16	in
b=	3.50	in	l /	612	
d=	10.50	in	Cv=	1.00	
E=	1,800	ksi			



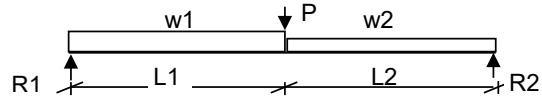
(E) Transfer Joist	B13		HF	6 x 10	
w1=	69	plf	R1 =	4,227	lbs
w2=	69	plf	R2 =	1,159	lbs
L1=	2	ft	M =	8,315	lb-ft
L2=	10	ft	Fb =	1,166	psi
X=	2.0	ft	Fv =	113	psi
P=	4,525	lbs	Δ=	0.21	in
b=	6.00	in	l /	701	
d=	9.25	in	Cv=	1.00	
E=	1,300	ksi			



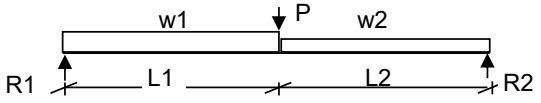
(E) Floor Beam	B14		HF	7 x 10	
w1=	442	plf	R1 =	3,418	lbs
w2=	390	plf	R2 =	2,942	lbs
L1=	2	ft	M =	6,764	lb-ft
L2=	3	ft	Fb =	876	psi
X=	2.3	ft	Fv =	77	psi
P=	4,227	lbs	Δ=	0.05	in
b=	6.50	in	l /	1,277	
d=	9.25	in	Cv=	1.00	
E=	1,300	ksi			



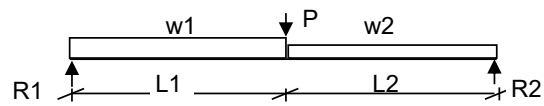
Transfer Joist	B15		HF	5 x 10	
w1=	69	plf	R1 =	2,304	lbs
w2=	69	plf	R2 =	790	lbs
L1=	2	ft	M =	4,469	lb-ft
L2=	10	ft	Fb =	836	psi
X=	2.0	ft	Fv =	81	psi
P=	2,233	lbs	Δ=	0.16	in
b=	4.50	in	l /	910	
d=	9.25	in	Cv=	1.00	
E=	1,300	ksi			



(E) Floor Beam	B16		HF	4 x 10	
w1=	546	plf	R1 =	1,853	lbs
w2=	546	plf	R2 =	2,393	lbs
L1=	5	ft	M =	3,118	lb-ft
L2=	1	ft	Fb =	750	psi
X=	5.3	ft	Fv =	91	psi
P=	790	lbs	Δ=	0.04	in
b=	3.50	in	l /	1,973	
d=	9.25	in	Cv=	1.00	
E=	1,300	ksi			



(E) Floor Beam	B17		HF	4 x 10	
w1=	390	plf	R1 =	1,777	lbs
w2=	390	plf	R2 =	2,184	lbs
L1=	3	ft	M =	3,225	lb-ft
L2=	2	ft	Fb =	775	psi
X=	2.5	ft	Fv =	87	psi
P=	2,304	lbs	Δ=	0.03	in
b=	3.50	in	l /	1,753	
d=	9.25	in	Cv=	1.00	
E=	1,300	ksi			



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Service Loads Loading

Dead Load =	1.8 kips
Live Load =	2.6 kips
Wind/EQ Load =	0.0 kips
Wind/EQ Moment (M_Y) =	0 ft-kips
Gravity Load Eccentricity (±X)=	0.00 ft.
Footing Weight =	0.5 kips
Total Load =	4.9 kips
Total Moment =	0 ft-kips

Service Load Factors

DL	1
LL	1
EQ/Wind	1

Soil Properties

Allowable Soil Brg. (Qa) =	2.00 ksf
Overburdan Density (γs) =	120 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) =	0.77 ft.
Column Face from left (Cl) =	0.77 ft.

Soil Bearing Check (Allowable)

Eccentricity =	0.00 ft.	
Leng. Soil Brg. Under Ftg.=	2.00 ft.	
qmax =	1.23 ksf	OK
qmin =	1.23 ksf	

Footing Dimensions

L Dimension (X) =	2.00 ft.
B Dimension (Y) =	2.00 ft.
Footing Thickness (t) =	10.00 in.
Ftg Overburden (Ot) =	0.00 ft.

Soil Pressure Equations:

$e \le L/6$

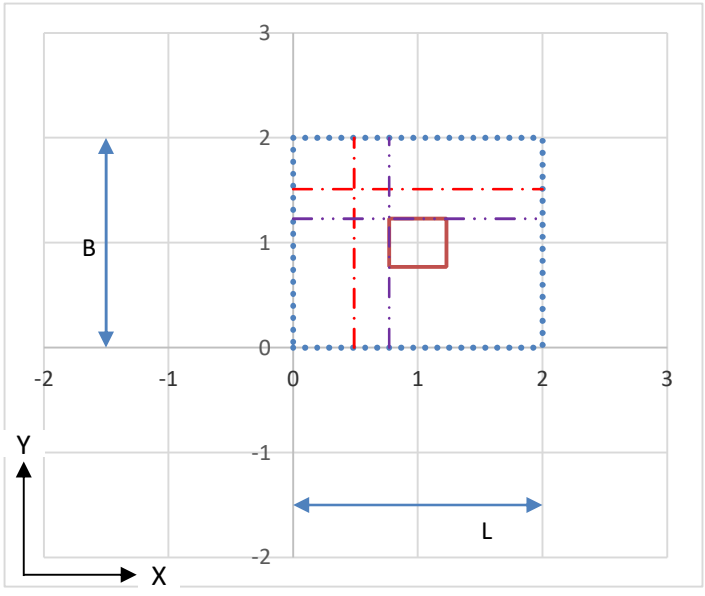
$qmax = \frac{Q}{LB}\left(1 + \frac{6e}{L}\right)$

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

$e > L/6$

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

$qmin = 0$



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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) =	6.75 in.

Strength Load Factors

DL	1.2
LL	1.6
EQ/Wind	1

Factored Loads

Factored Total Load =	6.9 kips
Factored Total moment =	0 ft-kips

Factored Moments and Shears

	Mu k-ft	Vu kips
X Right Side	1	2
X Left Side	1	2
Y Both Sides	1	2

Factored Bearing

Eccentricity =	0.00 ft.
Length of Soil Brg. Under Ftg. =	2.00 ft.
qmax =	1.73 ksf
qcolr =	1.73 ksf
qcoll =	1.73 ksf
qmin =	1.73 ksf

Flexural Design - X Direction (About Y-Axis)

Bar Size =	#4	
Bars =	3	
Mu =	1 ft-kips	
ØMn =	17 ft-kips	OK
ρ_{min} =	0.0018	*Controls*
ρ_{req} =	0.0002	
A_s Required =	0.29 sq. in.	
A_s Provided =	0.60 sq. in.	OK

Flexural Design - Y Direction (About X-Axis)

Bar Size =	#4	
Bars =	3	
Mu =	1 ft-kips	
ØMn =	17 ft-kips	OK
ρ_{min} =	0.0018	*Controls*
ρ_{req} =	0.0002	
A_s Required =	0.29 sq. in.	
A_s Provided =	0.60 sq. in.	OK

One-Way Shear Design - X Direction

Vu =	2 kips	
ØVn =	13 kips	OK

One-Way Shear Design - Y Direction

Vu =	2 kips	
ØVn =	13 kips	OK

β =	1.000
$\gamma_s = 2/(\beta+1)$ =	1.00
Provide $A_{s,req}\gamma_s$ =	0.29 sq. in.

Provide evenly distributed bars in each direction.

Two-Way (Punching) Shear Design

b_o =	49 in	
vu =	5 kips	
Øvn =	52 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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Typical Footing at Patio Post

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SIMPSON Strong-Tie® Anchor Designer™ Software
Version 3.0.7947.0

Company:		Date:	6/8/2023
Engineer:	ETC	Page:	1/5
Project:	Grima-Fahrer		
Address:			
Phone:			
E-mail:			

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: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 8.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 9.38
C_{min} (inch): 3.75
S_{min} (inch): 3.75

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 10.00
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: No
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 3.00 x 3.00 x 0.25

Recommended Anchor
Anchor Name: Heavy Hex Bolt - 5/8"Ø Heavy Hex Bolt, F1554 Gr. 36

Grima-Fahrer Residence
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HDU Anchorage

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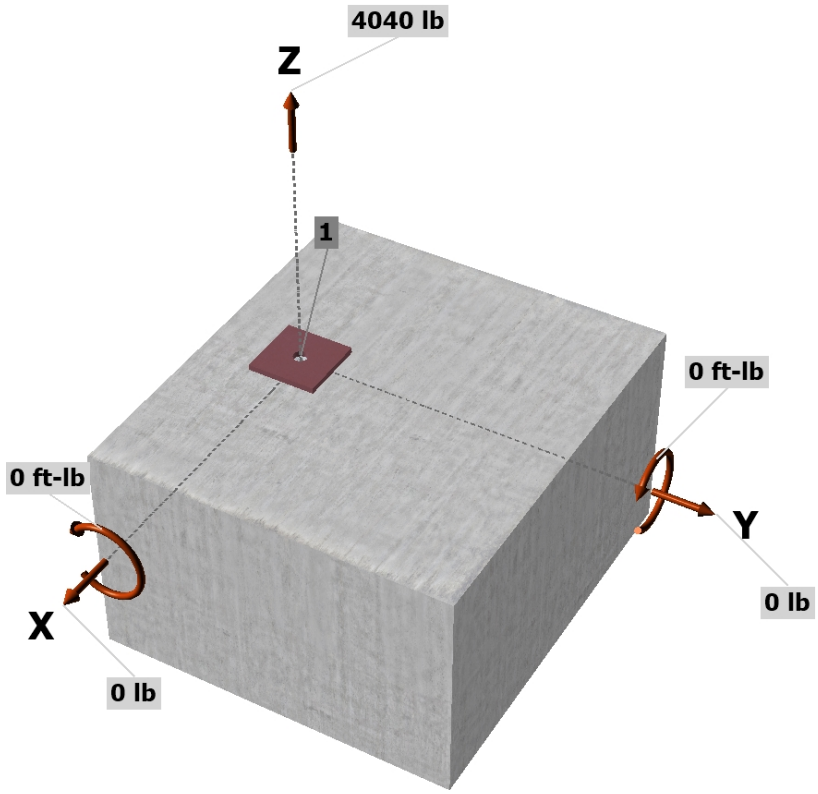
E-mail:			

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: 17.2.3.4.2 not applicable
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: No

Strength level loads:

N_{ua} [lb]: 4040
V_{uax} [lb]: 0
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0

<Figure 1>



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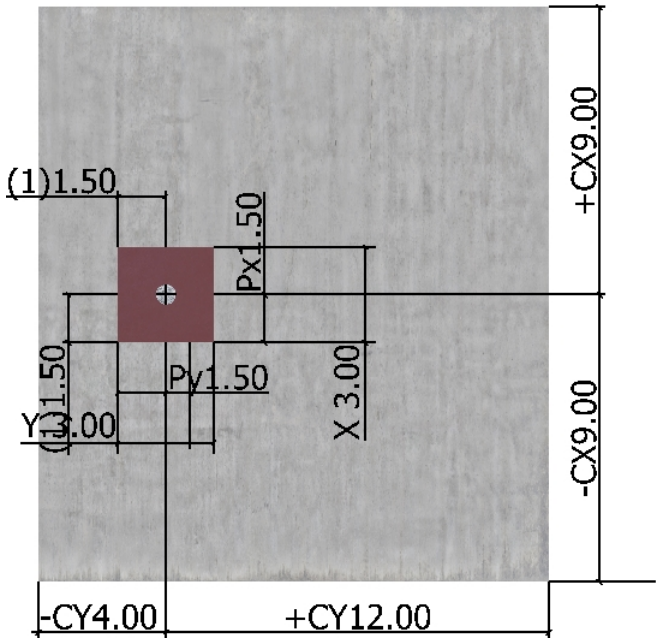
Phone:			
E-mail:			

<Figure 2>



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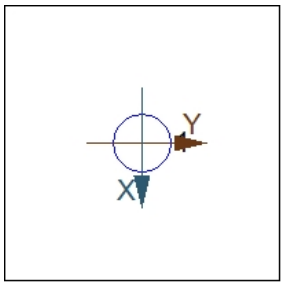
3. Resulting Anchor Forces

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	4040.0	0.0	0.0	0.0
Sum	4040.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 4040
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

<Figure 3>



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

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13100	0.75	9825

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)
24.0	1.00	2500	6.000	17636

$0.75\phi N_{cb} = 0.75\phi \, (A_{Nc} / A_{Nco}) \, \Psi_{ed,N} \, \Psi_{c,N} \, \Psi_{cp,N} N_b$ (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)
234.00	324.00	4.00	0.833	1.00	1.000	17636	0.70	5573

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

$0.75\phi N_{pn} = 0.75\phi \Psi_{c,P} N_p = 0.75\phi \Psi_{c,P} 8 A_{brg} f'_c$ (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	0.67	2500	0.70	7046

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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	4040	9825	0.41	Pass
Concrete breakout	**4040**	**5573**	**0.72**	**Pass (Governs)**
Pullout	4040	7046	0.57	Pass

5/8"Ø Heavy Hex Bolt, F1554 Gr. 36 with hef = 8.000 inch meets the selected design criteria.

12. Warnings

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.4.2 for tension need not 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.

Grima-Fahrer Residence
PROJECT

HDU Anchorage

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DATE
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PROJ. #
ETC
DESIGN
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SHEET