

CALCULATIONS:

Voluntary Substructural and Soft Story Strengthening consisting of shear strengthening though a plywood paneling anchored to the foundation and/or universal retrofit foundation plates anchored to the foundation when crawl spaces is less that 2'. Steel columns or engineered panels may be used.

The proposed engineering solution aligns with CEBC Chapter A3, incorporating FEMA P-1100 compliance as required by project specifications
The engineered design is intended to provide a basic protection from cripple wall collapse during a reasonably foreseeable earthquake.

This project shall comply with the following codes as adopted and amended by the City of Berkeley: 2025 California Building Code (CBC); 2025 California Residential Code (CRC), where applicable; 2025 California Existing Building Code (CEBC); 2025 California Mechanical Code (CMC); 2025 California Electrical Code (CEC); 2025 California Plumbing Code (CPC); 2025 California Energy Code (CEnC); 2025 California Green Building Standards Code (CALGreen); City of Berkeley Municipal Code Amendments.

Seismic Design Category E (SDC E)
Occupancy Group R-2

LOADING:

Roof & Framing	20.00 lb/ft ²	Exterior Walls (Wood & Shake & Plaster)	16.00 lb/ft ²	Cracked concrete Light Wood Framing	58.00 lb/ft ²	CMU Wall Weight
Floor & Framing	11.00 lb/ft ²	Interior Walls (Plaster)	8.00 lb/ft ²	Wood Exterior Composit Roof Wood Floors		

Two-story single-family home built over a crawl space.

SF over front two floors	1744	SF			
Roof: 1744 ft ² X 19			34.9 kips	Roof Fx	Vr= 13.0 kips
Upper Ext Walls: (186' X 10') X 16			29.8 kips		
Upper Int Walls: (110' X 10') X 8			8.8 kips		
Upper Floor: 1744 ft ² X 11			19.2 kips	2nd FL	Vfl= 34.6 kips
Lower Ext Walls: (186' X 8') x 16			23.8 kips		
Lower Int Walls: (125' X 8') X 8			8.0 kips		
Floor: 1744 ft ² X 11			19.2 kips		
Garage: (34' X 8') x 58			15.8 kips		
Crawl Space (Walls): (64' X 8') x 16			8.2 kips		
		W _D =	167.6 kips		

Seismic Design Criteria

Soil Class	D		
Response Spectra (0.2s) S _s	2.690	g	
Response Spectra (1s) S ₁	1.050	g	
Site Coefficient F _a	1.200		Table 11.4-1
Site Coefficient F _v	null		Table 11.4-2
Max Considered Earthquake Acc S _{MS}	2.810	g	SEA Design Maps Summary Report (Attached)
Max Considered Earthquake Acc S _{M1}	2.21	g	SEA Design Maps Summary Report (Attached)
At 5% Damped Design S _{DS}	1.87	g	SEA Design Maps Summary Report (Attached)
S _{D1}	1.47	g	SEA Design Maps Summary Report (Attached)
Building Occupancy	II		Table 1604.5
Seismic Design Category	E		SDC
Response Modification Coef. R	6.5		Table 12.2-1
Response Modification Coef. R	2.5		
Importance Factor I	1.0		Table 11.5-1

Equivalent Lateral Force Procedure 12.8

C _t =	0.02	Table 12.8-2
x=	0.75	Table 12.8-2
h _n [^] =	20.00	Height in ft above the Base
C _u =	1.4	Table 12.8-1

Seismic Base Shear

V= C _s X W X 75% X 1.3		Section 12.3.4.2
C _s = S _{DS} / (R/I)	0.374	12.8-2
	0.972	
V=	47.01 kips	R=6.5
V _{col} =	122.22 kips	R=2.5 col

SHEAR DISTRIBUTION			Min Shear Wall length per (lf)										Shear Wall Design								Hold Downs (min req)		
Grid Line	Tributary Area		Shear	edge nailing = Shear/Type			URFP frnd plate (Shear/Plate Capacity)	Bolting = Shear/Bolt Capacity		Transfer Ties =Shear/Tie Capacity		cw ht	Lateral Bracing Element	#	# of Universal Retrofit Foundation Plate (URFP)	kips	Type A		Type D		Design shear	uplift	Anchors (min)
	L	W	L X W (kips)	Type A (785 lbs)	Type C (1540 lbs)	Type D (2045 lbs)		1/2" bolts	5/8" bolts	L90	L70						SW length	n/e	SW length	n/e			
1	14	12	4.53	6	3	3	4	6	4	5	7	4					6	n			0.755	3.019	HDU2
2	12	11	3.56	5	3	2	4	5	4	4	5	3					6	n			0.593	1.779	HDU2
3	26	10	7.01	9	5	4	6	9	6	8	10	1			6	7.08							
4	8	14	3.02	4	2	2	3	4	3	4	5	4					4	n			0.755	3.019	HDU2
5	16	14	6.04	8	4	3	6	8	6	7	9	1			6	7.08							
6	30	13	10.51	14	7	6	9	14	9	12	15	6							10	n	1.051	6.307	HDU8
7	8	12	2.59	4	2	2	3	4	3	3	4	1			3	3.54							
8	30	14	11.32	15	8	6	10	15	10	13	16	8							2 - 3' panels	n	3.774	5.187	HDU5
A	16	15	16.82	22	11	9	15	22	15	19	23	8	HSS 10X10X3/8	1									
B	14	18	6.79	9	5	4	6	9	6	8	10	4					10	n			0.679	2.717	HDU2
C	16	15	6.47	9	5	4	6	9	6	7	9	8							2 - 3' panels	n	1.078	1.953	HDU2
D	16	15	6.47	9	5	4	6	9	6	7	9	1			6	7.08							
E	32	18	15.53	20	11	8	14	20	14	17	21	3			6	7.08	10	n			1.553	4.658	HDU5
F	14	16	6.04	8	4	3	6	8	6	7	9	1			6	7.08							

SW 0.0270 kips/ft2
Column 0.0701 kips/ft2

Diaphragms: 15/32" CDX 15 Ga X 1-3/4" staples @ 2"

Shearwall Types Table 4.3A AWC - Wood Structural Panels Structural I

Anchor:	Capacity (lbs.)	Collectors:	Capacity (lbs.)	
A23	485	KC Metals	MTS37	4030
L70	740	KC Metals	MTS38	4800
L90	925			
URFP	1510	1180 for Redwood		
HDU2	3075			
HDU4	4565			
HDU5	5645			
HDU8	6970			
A	15/32" Struct I, w/ 8d @ 6" CC all edges, V _{allow} =785 plf			
B	15/32" Struct I, w/ 8d @ 4" CC all edges, V _{allow} = 1205 plf			
C	15/32" Struct I, w/ 8d @ 3" CC all edges, V _{allow} = 1540 plf			
D	15/32" Struct I, w/ 8d @ 2" CC all edges, V _{allow} = 2045 plf			
	6" Field			
	8d Nail - 2-1/2" X 0.131" X 0.281"			

Hand calculations example for Shear Distribution Table:

A			B				C	D		E											F	G	
SHEAR DISTRIBUTION			Min Shear Wall length per (lf)															Hold Downs (min req)					
	Tributary Area		Shear	edge nailing = Shear/Type			URFP fnd plate (Shear/Plate Capacity)	Bolting = Shear/Bolt Capacity		Transfer Ties =Shear/Tie Capacity			Lateral Bracing Element	#	# of Universal Retrofit Foundation Plate (URFP)	kips	Type A		Type D		Design shear	uplift	Anchors (min)
	L	W	L X W (kips)	Type A (785 lbs)	Type C (1540 lbs)	Type D (2045 lbs)		1/2" bolts	5/8" bolts	L90	L70						SW length	n/e	SW length	n/e			
Grid Line	L	W	L X W (kips)	Type A (785 lbs)	Type C (1540 lbs)	Type D (2045 lbs)		1/2" bolts	5/8" bolts	L90	L70	cw ht					SW length	n/e	SW length	n/e	Design shear	uplift	Anchors (min)
1	14	12	4.53	6	3	3	4	6	4	5	7	4					6	n			0.755	3.019	H DU2

Shear/Grid Line 1:

- A.)

Tributary Area= L X W

14' X 12'=168 SF

Muliply by Shear force:

Shear Force= V/A

=47.01kips/1744 SF

0.027 kips/SF
- Shear @ Line 1: =168SF X 0.027kips

4.53 kips

B.) Shear Wall Nailing

- Shearwall Nailing Types :
- A

15/32" Struct I, w/ 8d @ 6" CC all edges, V_{allow} =785 plf
- B

15/32" Struct I, w/ 8d @ 4" CC all edges, V_{allow} = 1205 plf
- C

15/32" Struct I, w/ 8d @ 3" CC all edges, V_{allow} = 1540 plf
- D

15/32" Struct I, w/ 8d @ 2" CC all edges, V_{allow} = 2045 plf
- Type A: =(4.53kips X 1000)/785 plf

5.77 round up to 6'
- Type C: =(4.53kips X 1000)/1540 plf

2.94 round up to 3'
- Type D: =(4.53kips X 1000)/2045 plf

2.21 round up to 3'

C.) Universal Retrofit Foundatin Plates (URFP)

- URFP Capacity 1510lbs use 1180lbs for Redwood
- # of URFPs (if used) = (4.53kips X 1000)/1180lbs

3.8377

Round up to 4

D.) Bolting (either 1/2" or 5/8" diameter bolts)

- 1/2" dia bolt cap = 786lbs

=(4.53kips X 1000)/786lbs

5.76 round up to 6
- 5/8" dia bolt cap = 1177lbs

=(4.53kips X 1000)/1177lbs

3.85 round up to 4

E.) Shear Transfer Ties (STT)

- L70 cap = 740lbs

=(4.53kips X 1000)/740lbs

6.12 round up to 7
- L90 cap = 925lbs

=(4.53kips X 1000)/925lbs

4.90 round up to 5

F.) Design Shear - What is provided

- Design Shear = =(4.53kips X 1000)/(6' of Type A Shear Wall)

=(4.53kips X 1000)/6

754.75 lbs/LF

G.) Uplift

- Uplift = =[(4.53kips X 1000) X Cripple Wall Height]/ Shearl Wall Length

=[(4.53kips X 1000) X 4']/6'

3018.99 lbs
- Use HDU2 Hold Down with a capacity of 3075lbs

New HSS Column

$$S_{\text{required}} = M / 1.33 \times 0.6 \times 50 \text{ ksi}$$

where:

$$P = 16.82 \text{ kips}$$
$$H = 8 \text{ ft}$$
$$M = 1614.6 \text{ in-kips}$$

$$S_{\text{required}} = 40.47 \text{ in}^3$$

HSS 10 X 10 X 3/8 GRADE 50

$$S = 42.9 \text{ in}^3$$
$$I = 214 \text{ in}^4$$
$$42.9 \text{ in}^3 > 40.47 \text{ in}^3$$

Deflections:

Total deflection $\Delta_T = \Delta_C + \Delta_R + \Delta_{\text{slip}}$

Where: Δ_C = Bending of the column itself

Δ_R = Rotation of the foundation grade beam multiplied by the height

Δ_{slip} = Movement due to loose connections between column and wood framing

$$\Delta_C = \frac{PH^3}{3EI_{\text{col}}}$$
$$\Delta_C = 16.82 \text{ kips} \times (8 \text{ ft})^3 \times 1728 \text{ in}^3/\text{ft}^3 / 3 \times 29,000 \text{ kips/in}^2 \times 214 \text{ in}^4$$
$$\Delta_C = 0.80 \text{ in}$$

$$\Delta_R = \Theta H$$

Where: $\Theta = ML / 3EI_{\text{rebar}}$ $I_{\text{rebar}} = 2 \times (A_s d^2)$ D_1 = distance from the Top and Bottom Reinforcement

$L = 12 \text{ ft}$ of length of grade beam $D_1 = (d - 2") / 2 = 6.3125 \text{ in}$

$I_{\text{rebar}} = 342 \text{ in}^4$ $d = 15 \text{ inches}$

$A_s = 0.80 \text{ in}^2$

$$\Delta_R = \Theta H = 621 \text{ in-kips} \times [(12 \text{ ft}) \times 144 \text{ in}^2/\text{ft}^2] / 3 \times 29,000 \text{ kips/in}^2 \times 342 \text{ in}^4 \times 96"$$
$$\Delta_R = 0.001 \text{ in}$$

$\Delta_{\text{slip}} = 3/16 \text{ inch}$ Estimated slip from bolts at top of column to steel channel

TOTAL DEFLECTION: $\Delta_T = \Delta_C + \Delta_R + \Delta_{\text{slip}}$

$\Delta_T = 0.99 \text{ in}$

Allowable deflection for new construction = 0.025 X story height(ft) X 12inches/ft

Allowable Deflection = 0.025 X 8ft X 12in/ft

$\Delta_{\text{allowable}} = 2.40 \text{ in} > 0.99 \text{ in OK}$

Framing connections at top of Column:

New DF 4X12 #1 or better for diaphragm collector.

$$V_{\text{collector}} = 16.82 \text{ kips}$$

Overstrength Factor $\Omega_0 = 2.5$

$$F_{\text{collector}} = \Omega_0 \times V_{\text{collector}} \quad \text{ASCE 7-22 Section 12.10.2.1}$$
$$F_{\text{collector}} = 2.5 \times 16.82$$

Collector Force: $F_{\text{collector}} = 42.05 \text{ kips}$ Any splice, strap, or hardware in the collector line shall be designed for 42.05 kips.

Framing connections at top of Column: Number of 3/4" bolts from Column to Beam Hanger: 3/4" A325 Bolts rated at 18.6K = 11.7K/bolt. "SDS" screws rated at: 0.42k steel-to-wood.

1. Beam hanger–to–HSS 10×10×3/8 column (3/4" A325 bolts)

Required bolts:

$$n_{\text{bolts}} = \frac{F_{\text{collector}}}{11.7} = \frac{42.05}{11.7} = 3.59 \Rightarrow 4 \text{ bolts}$$

Provide (4) 3/4" Ø ASTM A325 bolts at beam hanger–column connection. Bolt group capacity ≥ 42.05 kips → OK for collector overstrength demand.

2. SDS screws – 4×12 DF collector to joists

Required SDS screws (steel-to-wood, 0.42 kips per screw, ASD, 1.33 factor):

$$n_{\text{SDS}} = \frac{F_{\text{collector}}}{0.42 \times 1.33} = \frac{42.05}{0.5586} \approx 75.3 \Rightarrow 76 \text{ SDS screws}$$

Provide (76) SDS screws from 4×12 DF collector into floor joists (steel-to-wood). Total SDS capacity ≥ 42.05 kips → OK for collector overstrength demand.

Voluntary Substructural Strengthening

Attachment - A1

New Foundation for Steel Column

P= 16.82 kips
H= 8 ft
M= 1615 in-kips 135 ft-kips
M_u=MX1.4= 2260 in-kips
188.37 ft-kips
f'_c= 2.5 ksi
f_y= 60 ksi ASTM 615 Steel
D= Grade Beam Depth 24 inches
b= Grade Beam Width 30 inches
d= D-3" + re 21 inches

Required Area of Steel:

As= Mu/φfy(d-a/2)
a= A_{s1}f_y / bX0.85f'_c Assume A_{s_I} = 3 in²
bX0.85Xf'_c= 63.75 in
A_{s1}f_y= 180
a= 3 in² X 60 ksi / 42 in X 0.85 X 2.5 ksi
a= 2.82
d-a/2 = 17.59
therefore,
As= 2.38
If A_{s_I} ≥ A_s then assumption is good

Use: 4 - #8s top and bottom

As = 3.16 in²

Concrete Volume: (1.83" X 2.5' X 12')/27 2.2 CY

Overturning Resistance:

L= Length of Grade Beam 12 ft
λ_{soil}= 115 pcf Soil Weight= (0.115kips/ft³ X d²) / 2 = 0.23 k/ft
λ_{concrete}= 150 pcf Concrete Weight= (0.150kips/ft³) X b X d = 0.75 k/ft
φ= 0.9
Total = 0.98 k/ft
M_R=φ X L² X (soil + conc.) / 2 = 762 in-kips
Factor of Safety = M_R/M = 0.47 Factor of Safety

Bearing Force:

P= 16.82 kips
P_{soil&Conc}= 23.52 kips
P_{tot}= 40.34 kips
M= 64 ft-kips
e= M/P
e= 1.574253 ft
e< L/6 2.000 ft
1.574 < 2.000
q_{max}= [P/(L x B)] x [1 + (6 x e)/L]
q_{max}= 2.40 ksf
q_{ALLOW}= 2000*1.33 for seismic Allowable bearing pressure = 2,000 psf CBC Table 1806.2
q_{ALLOW}= 2.66 ksf
q_{ALLOW} > q_{max}
2.66 > 2.40

Voluntary Substructural Strengthening

Attachment - A1

New Foundation for new Shear Wall - Grid Line 6

P= 10.51 kips
H= 8 ft
M= 1009 in-kips 84 ft-kips
M_u=MX1.4= 1413 in-kips
117.73 ft-kips
f'_c= 2.5 ksi
f_y= 60 ksi ASTM 615 Steel
D= Grade Beam Depth 18 inches
b= Grade Beam Width 24 inches
d'= D-3" + rel 15 inches

Required Area of Steel:

As= Mu/φfy(d-a/2)
a= A_{s1}f_y/ bX0.85f'_c Assume A_{s1} = 2 in²
a= 1 in² X 60 ksi / 30 in X 0.85 X 2.5 ksi
a= 2.35
therefore,
As= 1.79
If A_{s1} ≥ A_s then assumption is good

Use: 4 - #4s top and bottom

As = 0.80 in²

Concrete Volume: (1.5' X 2.0' X 10.0')/27 1.1 CY

Overturning Resistance:

L= Length of Grade Beam 10 ft
λ_{soil}= 115 pcf Soil Weight= (0.115kips/ft³ X d'²) / 2 = 0.13 k/ft
λ_{concrete}= 150 pcf Concrete Weight= (0.150kips/ft³) X b X d = 0.45 k/ft
φ= 0.9
Total = 0.58 k/ft
M_R=φ X L² X (soil + conc.) / 2 = 313 in-kips
Factor of Safety = M_R/M = 0.31 Factor of Safety

Bearing Force:

P= 10.51 kips
P_{soil&Conc}= 11.59 kips
P_{tot}= 22.10 kips
M= 26 ft-kips
e= M/P
e= 1.179752 ft
e< L/6 1.667 ft
1.180 < 1.667
q_{max}= [P/(L x B)] x [1 + (6 x e)/L]
q_{max}= 1.89 ksf
q_{ALLOW}= 2000*1.33 for seismic
q_{ALLOW}= 2.66 ksf
q_{ALLOW} > q_{max}
2.66 > 1.89

- Base plate bearing & punching
- Anchor steel interaction (tension + shear)
- Full ACI 318-19 Chapter 17 summary

Base plate bearing and punching checks

Given:

- Column: HSS 10×10×3/8 (filled w/ concrete)
- Bearing plate: 1" thick, 14"×14"
- Grade beam: 24"×30", $f'_c = 2500$ psi
- Seismic demand (for lateral system): $\Omega_0 F_p = 42.05$ kips

1. Concrete bearing under base plate

Plate bearing area:

$$A_{bear} = 14" \times 14" = 196 \text{ in}^2$$

Concrete bearing strength (ACI 318-19 §22.8):

$$f_{ce} = 0.85f'_c = 0.85(2500) = 2125 \text{ psi}$$

Nominal bearing capacity:

$$N_{bear} = f_{ce}A_{bear} = 2125(196) = 416,500 \text{ lb} = 416.5 \text{ kips}$$

Strength-reduction factor:

$$\phi = 0.65$$

Design bearing strength:

$$\phi N_{bear} = 0.65(416.5) = 270.7 \text{ kips}$$

Check:

$$\phi N_{bear} = 270.7 \text{ kips} \geq 42.05 \text{ kips}$$

Base plate concrete bearing OK

2. Punching shear (around column)

Critical perimeter around HSS 10×10 (simplified):

$$b_0 \approx 4(10") = 40"$$

Effective depth (grade beam):

$$d \approx 24" - \text{cover} \approx 20" \text{ (conservative)}$$

Nominal punching shear strength (no axial load given, lateral system only):

$$V_c \approx 2\sqrt{f'_c} b_0 d$$

$$\sqrt{2500} = 50$$

$$V_c \approx 2(50)(40)(20) = 80,000 \text{ lb} = 80 \text{ kips}$$

Strength-reduction factor:

$$\phi = 0.75$$

$$\phi V_c = 0.75(80) = 60 \text{ kips}$$

Demand here is primarily lateral shear, not column axial; punching is not governing for this voluntary substructure strengthening detail. Punching shear not controlling for the lateral demand considered

Anchor steel interaction (tension + shear)

Anchors:

- (4) 1" Ø ASTM F1554 Gr. 105
- Embedment: 21"
- Spacing: 3" o.c. each way

- Concrete: $f'_c = 2500$ psi
- Seismic demand:
 - Steel limit states $\rightarrow F_p = 16.82$ kips(unamplified)
 - Concrete limit states $\rightarrow \Omega_0 F_p = 42.05$ kips

1. Steel shear (per rod and group)

Per rod (threads in shear plane):

$$V_n = 0.6F_uA_b$$

Take:

$$F_u = 125\text{--}150 \text{ ksi}, A_b = 0.606 \text{ in}^2$$

Lower bound:

$$V_n = 0.6(125)(0.606) = 45.5 \text{ kips}$$

Upper bound:

$$V_n = 0.6(150)(0.606) = 54.5 \text{ kips}$$

Strength-reduction factor:

$$\phi = 0.65$$

Design shear per rod:

$$\phi V_n = 29.5\text{--}35.5 \text{ kips}$$

Group of 4 rods:

$$\phi V_{n,\text{group}} = 4(29.5\text{--}35.5) = 118\text{--}142 \text{ kips}$$

Demand (steel):

$$V_u = F_p = 16.82 \text{ kips}$$

Check:

$$\phi V_{n,\text{group}} \gg 16.82 \text{ kips}$$

Steel shear OK

2. Steel tension (per rod and group)

Per rod:

$$N_n = F_uA_b$$

$$N_n = (125\text{--}150)(0.606) = 75.8\text{--}90.9 \text{ kips}$$

Strength-reduction factor:

$$\phi = 0.75$$

Design tension per rod:

$$\phi N_n = 56.9\text{--}68.2 \text{ kips}$$

Group of 4 rods:

$$\phi N_{n,\text{group}} = 4(56.9\text{--}68.2) = 227.6\text{--}272.8 \text{ kips}$$

Any reasonable uplift demand from this voluntary strengthening will be well below this range.

Steel tension OK (for typical uplift demands)

ACI 318-19 Chapter 17 summary (anchors)

Demand split:

- Steel limit states: $F_p = 16.82$ kips
- Concrete limit states: $\Omega_0 F_p = 42.05$ kips

Anchor configuration:

- (4) 1" Ø F1554 Gr. 105

- Embedment $h_{ef} = 21$ in
- Spacing 3" o.c. each way
- Grade beam 24"×30", $f'_c = 2500$ psi
- Edge distance $c_{a1} = 6$ in

1. Concrete tension breakout

$$N_b = 24\sqrt{f'_c} h_{ef}^{1.5} = 24(50)(21^{1.5}) = 115.6 \text{ kips (per anchor)}$$

Compact 3"×3" group $\rightarrow A_{Nc} \approx A_{Nco}$, not edge-limited.

$$N_{cbg} \approx 4N_b = 462.4 \text{ kips}$$

$$\phi = 0.70 \Rightarrow \phi N_{cbg} = 323.7 \text{ kips}$$

Check:

$$\phi N_{cbg} = 323.7 \text{ kips} \geq 42.05 \text{ kips}$$

Concrete tension breakout OK

2. Concrete shear breakout

Same compact group, deep embedment, and edge geometry (group shear breakout of similar magnitude to tension breakout).

$$\phi V_{n,conc} \gg 42.05 \text{ kips}$$

Concrete shear breakout OK

3. Other concrete limit states

- Pullout: deep embedment, headed/nutted rods $\rightarrow$ not governing.
- Side-face blowout: embedment 21" with 6" edge $\rightarrow$ not governing for this configuration.
- Concrete pryout: with high breakout capacity and modest shear, pryout $<$ breakout $\rightarrow$ not governing.



**Anchor Designer™ for
Concrete Software**
Version 3.4.2512.1

Company:		Date:	4/1/2026
Engineer:	Kerry Morgan, PE	Page:	2
Project:	1449 Grizzly Peak Blvd Berkeley, CA 94708		
Address:			
Phone:			
E-mail:	kerrymorganengineering@gmail.com		

Load and Geometry

Load factor source: ACI 318 Section 9.2

Load combination: not set

Seismic design: Yes

Anchors subjected to sustained tension: No

Strength reduction factor for brittle failure, ϕ_s : not applied

Apply entire shear load at front row: No

Anchors only resisting wind and/or seismic loads: Yes

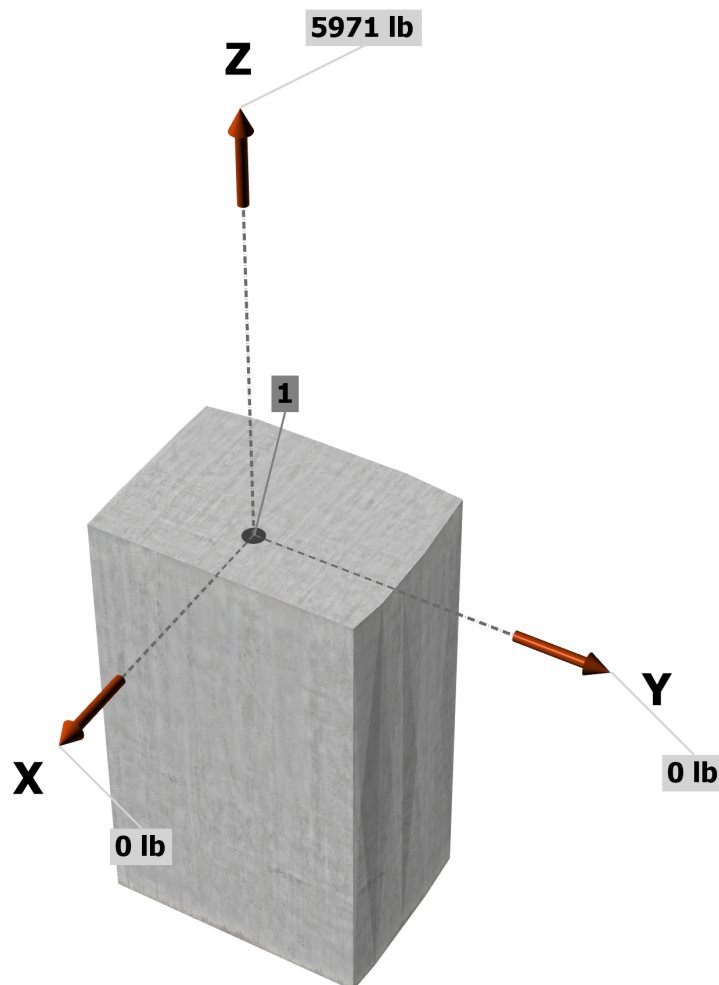
Strength level loads:

N_{ua} [lb]: 5971

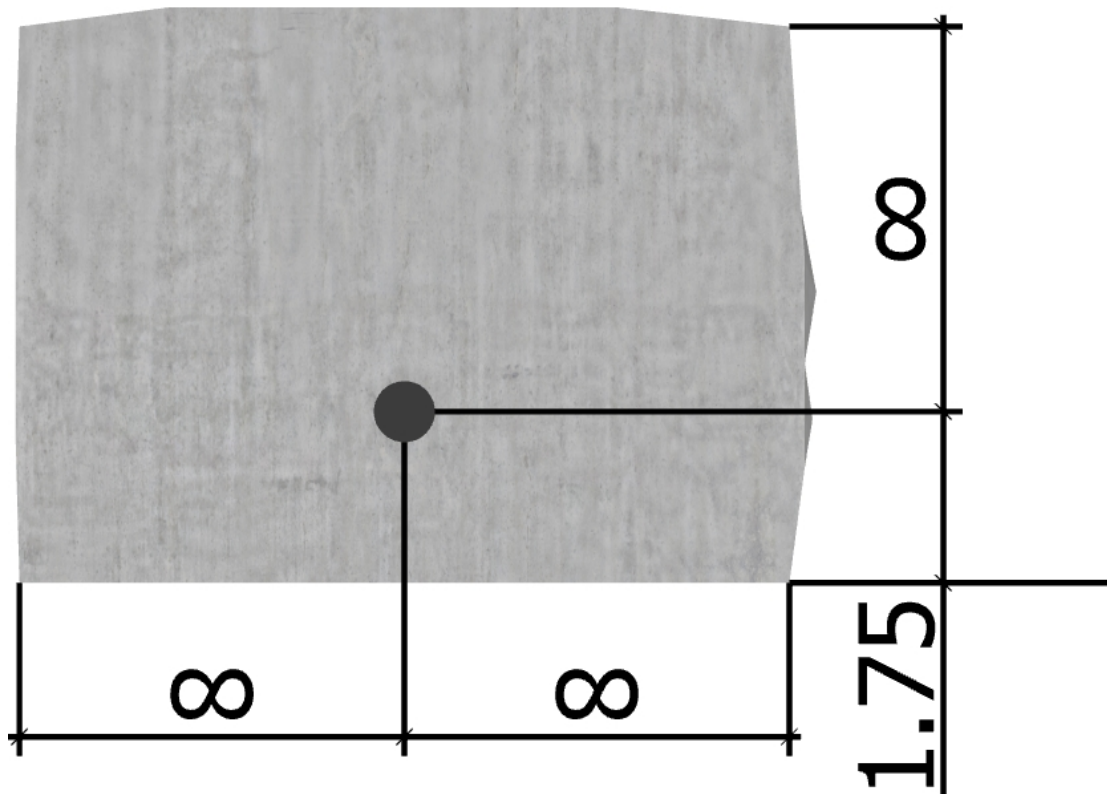
V_{uax} [lb]: 0

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	5971.0	0.0	0.0	0.0
Sum	5971.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 5971
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



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4. Steel Strength of Anchor in Tension (Sec. D.5.1)

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

5. Concrete Breakout Strength of Anchor in Tension (Sec. D.5.2)

$$N_b = k_c \lambda \sqrt{f'_c} h_{ef}^{1.5} \text{ (Eq. D-7)}$$

k_c	λ	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	12.000	35334

$$0.75 \phi_d \phi N_{cb} = 0.75 \phi_d \phi (A_{Nc} / A_{Nco}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \text{ (Sec. D.3.3.3, D.4.1 & Eq. D-4)}$$

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	$0.75 \phi_d \phi N_{cb}$ (lb)
711.00	1296.00	1.75	0.729	1.00	1.000	35334	0.65	6891

6. Adhesive Strength of Anchor in Tension (AC308 Sec. 3.3)

$$\tau_{k,cr} = \tau_{k,cr} f_{short-term} K_{sat} (f'_c / 2,500)^n \alpha_{N,seis}$$

$\tau_{k,cr}$ (psi)	$f_{short-term}$	K_{sat}	$\alpha_{N,seis}$	f'_c (psi)	n	$\tau_{k,cr}$ (psi)
1356	1.00	1.00	1.00	2500	0.24	1356

$$N_{a0} = \tau_{k,cr} \pi d_a h_{ef} \text{ (Eq. D-16f)}$$

$\tau_{k,cr}$ (psi)	d_a (in)	h_{ef} (in)	N_{a0} (lb)
1356	0.63	12.000	31950

$$0.75 \phi_d \phi N_a = 0.75 \phi_d \phi (A_{Na} / A_{Na0}) \Psi_{ed,Na} \Psi_{p,Na} N_{a0} \text{ (Sec. D.3.3.3, D.4.1 & Eq. D-16a)}$$

A_{Na} (in ²)	A_{Na0} (in ²)	$\Psi_{ed,Na}$	$\Psi_{p,Na}$	N_{a0} (lb)	ϕ	$0.75 \phi_d \phi N_a$ (lb)
143.20	232.97	0.769	1.000	31950	0.65	7360

7. Results

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

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	5971	9833	0.61	Pass
Concrete breakout	5971	6891	0.87	Pass (Governs)
Adhesive	5971	7360	0.81	Pass

SET-3G w/ 5/8"Ø F1554 Gr. 36 with hef = 12.000 inch meets the selected design criteria.

8. Warnings

- Brittle failure governs. Governing anchor failure mode is brittle failure.

Per ACI 318-08 Section D.3.3.4, anchors shall be designed to be governed by the steel strength of a ductile steel element in structures assigned to Seismic Design Category C, D, E, or F.

Alternatively it is permitted to take the design strength of the anchors as 0.4 times (or 0.5 times for the anchors of stud bearing walls) determined in accordance with Section D.3.3.3, or the attachment the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a force level corresponding to anchor forces no greater than the design strength of anchors specified in Section D.3.3.3. To include the strength reduction factor in the calculation, select a reduction factor from the Inputs tab and re-calculate.

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

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