Supplemental Structural Calculations For:

8480 Residence

8480 85th Ave SE

Mercer Island, WA 98040

BRETT D. MOZDEN
STATE OF WASHINGTON
45863
REGISTERED
PROFESSIONAL ENGINEER

Prepared for: Brandt Design Group

Job #: 01519-2021-09

Date: May 1, 2025

Entry Roof

Lateral Design

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Entry Roof - Design:

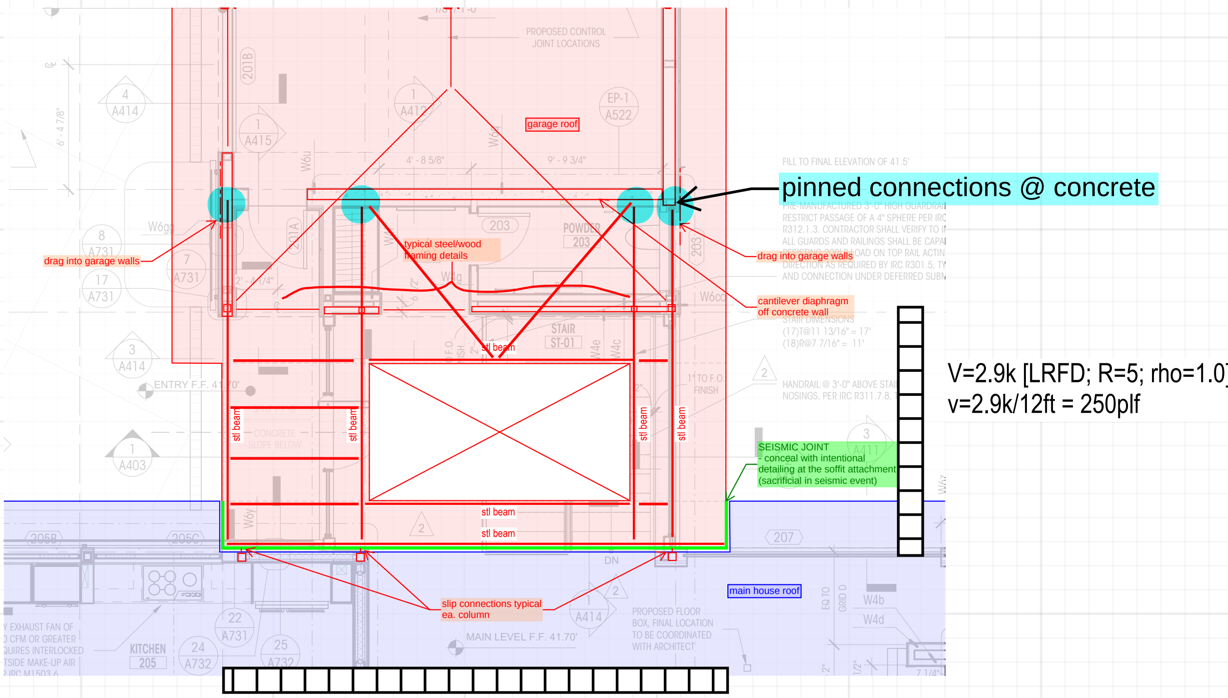
Summary:

- upper garage roof to cantilever from concrete garage wall rather than at light framed shearwall (due to rigidity)
- use steel frame support lateral loads around skylight opening

Design coefficients/factors:

R = 5.0, Ω = 2.5, Cd = 5.0 [Controlled by adjacent system: Special reinforced concrete shear walls (bearing wall systems)]

Key Plan w/ Design Forces:



V=2.9k [LRFD; R=5; rho=1.0]
v=2.9k/24ft= 0.125klf

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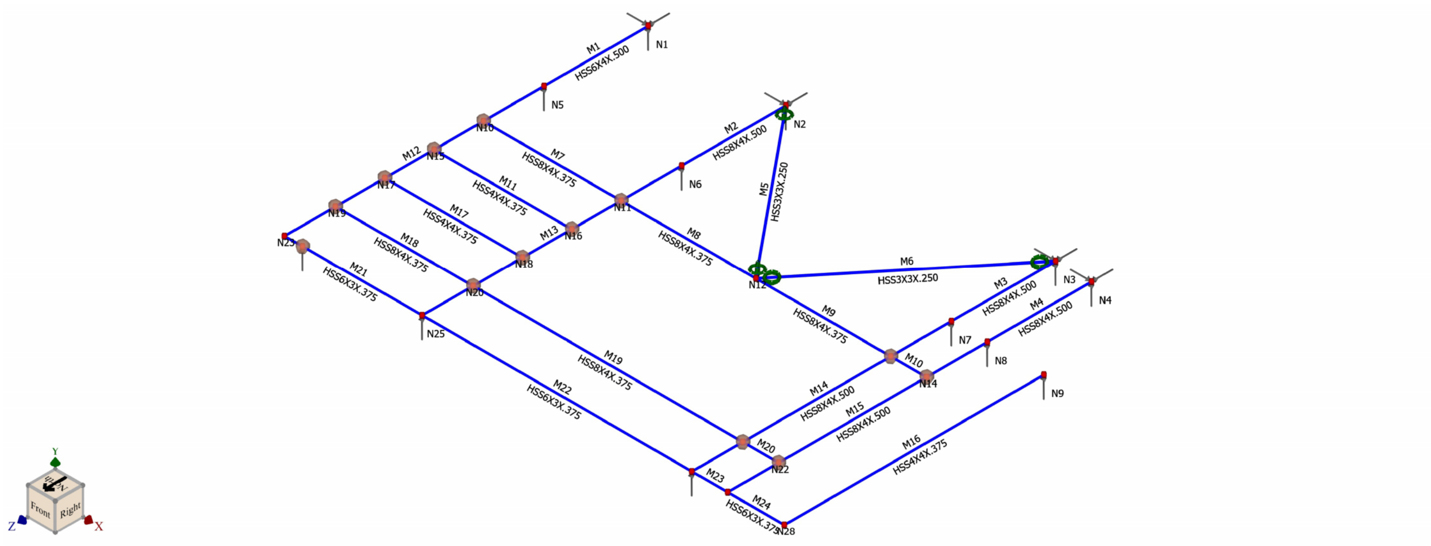
PROJECT	8480 Residence
DATE	10/12/2023
PROJ. #	01519-2021-09
DESIGN	haa
SHEET	

Visual Analysis Model:



Member Loads, Linear

Memb er	Service Case	Direction	Start Magnitude	End Magnitude	Full Length?	Start Offset ft	End Offset ft	Projected?
M1	D	Force Y	0.0000 K/ft	-0.0700 K/ft	Yes	0.0000	5.3330	No
M1	S	Force Y	0.0000 K/ft	-0.0800 K/ft	Yes	0.0000	5.3330	No
M4	D	Force Y	0.0000 K/ft	-0.0700 K/ft	Yes	0.0000	5.3330	No
M4	S	Force Y	0.0000 K/ft	-0.0800 K/ft	Yes	0.0000	5.3330	No

Member Loads, Uniform

Member	Service Case	Direction	Magnitude	Full Length?	Start Offset ft	End Offset ft	Projected?	Predefined Load
M7	D	Force Y	-0.0800 K/ft	Yes	0.0000	7.0417	No	N.A.
M7	S	Force Y	-0.0900 K/ft	Yes	0.0000	7.0417	No	N.A.
M8	D	Force Y	-0.1210 K/ft	Yes	0.0000	6.8958	No	N.A.
M8	D	Force Z	-0.1200 K/ft	Yes	0.0000	6.8958	No	N.A.
M8	S	Force Y	-0.1380 K/ft	Yes	0.0000	6.8958	No	N.A.
M8	S	Force Z	-0.1344 K/ft	Yes	0.0000	6.8958	No	N.A.
M9	D	Force Y	-0.1210 K/ft	Yes	0.0000	6.8958	No	N.A.
M9	D	Force Z	-0.1200 K/ft	Yes	0.0000	6.8958	No	N.A.
M9	S	Force Y	-0.1380 K/ft	Yes	0.0000	6.8958	No	N.A.
M9	S	Force Z	-0.1344 K/ft	Yes	0.0000	6.8958	No	N.A.
M11	D	Force Y	-0.0800 K/ft	Yes	0.0000	7.0417	No	N.A.
M11	S	Force Y	-0.0900 K/ft	Yes	0.0000	7.0417	No	N.A.
M12	E+X	Force X	-0.3000 K/ft	Yes	0.0000	13.2708	No	N.A.
M12	E-X	Force X	0.3000 K/ft	Yes	0.0000	13.2708	No	N.A.
M14	D	Force Y	-0.0300 K/ft	Yes	0.0000	13.2708	No	N.A.
M14	S	Force Y	-0.0380 K/ft	Yes	0.0000	13.2708	No	N.A.

Member Loads, Uniform (continued)

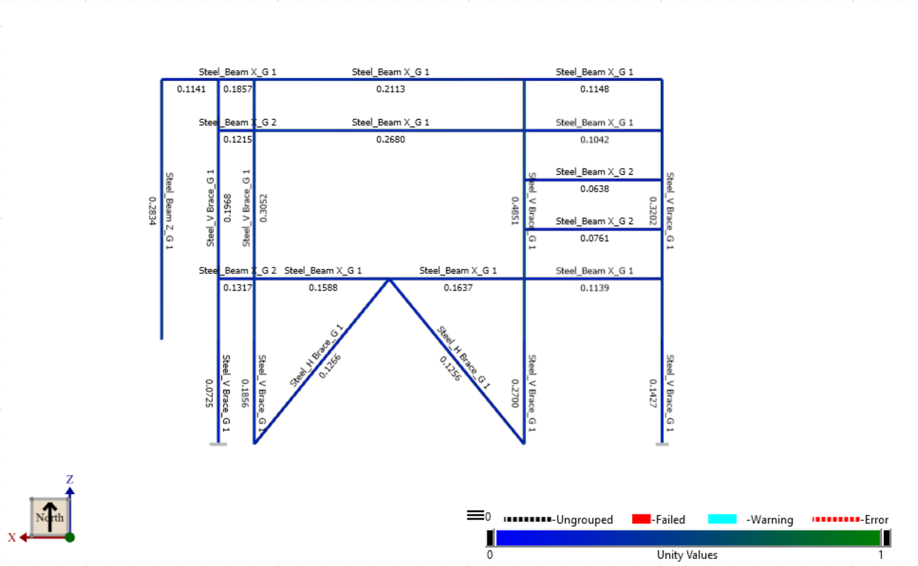
Member	Service Case	Direction	Magnitude	Full Length?	Start Offset ft	End Offset ft	Projected?	Predefined Load
M15	D	Force Y	-0.0600 K/ft	Yes	0.0000	13.2708	No	N.A.
M15	S	Force Y	-0.0630 K/ft	Yes	0.0000	13.2708	No	N.A.
M16	D	Force Y	-0.0300 K/ft	Yes	0.0000	13.2708	No	N.A.
M16	S	Force Y	-0.0380 K/ft	Yes	0.0000	13.2708	No	N.A.
M17	D	Force Y	-0.0800 K/ft	Yes	0.0000	7.0417	No	N.A.
M17	S	Force Y	-0.0900 K/ft	Yes	0.0000	7.0417	No	N.A.
M18	D	Force Y	-0.0800 K/ft	Yes	0.0000	7.0417	No	N.A.
M18	S	Force Y	-0.0900 K/ft	Yes	0.0000	7.0417	No	N.A.
M19	D	Force Y	-0.1210 K/ft	Yes	0.0000	13.7917	No	N.A.
M19	D	Force Z	0.1200 K/ft	Yes	0.0000	13.7917	No	N.A.
M19	S	Force Y	-0.1380 K/ft	Yes	0.0000	13.7917	No	N.A.
M19	S	Force Z	0.1344 K/ft	Yes	0.0000	13.7917	No	N.A.
M21	D	Force Y	-0.0400 K/ft	Yes	0.0000	7.0417	No	N.A.
M21	E+Z	Force Z	0.1400 K/ft	Yes	0.0000	7.0417	No	N.A.
M21	S	Force Y	-0.0450 K/ft	Yes	0.0000	7.0417	No	N.A.
M22	D	Force Y	-0.0330 K/ft	Yes	0.0000	13.7917	No	N.A.
M22	E+Z	Force Z	0.1400 K/ft	Yes	0.0000	13.7917	No	N.A.
M22	S	Force Y	-0.0380 K/ft	Yes	0.0000	13.7917	No	N.A.
M23	E+Z	Force Z	0.1400 K/ft	Yes	0.0000	1.8333	No	N.A.
M24	E+Z	Force Z	0.1400 K/ft	Yes	0.0000	2.8958	No	N.A.

Support Extreme Reactions

Node	Extreme FX K	Extreme FY K	Extreme FZ K
N1	0.0858	-0.5502	-1.4626
N2	3.1488	-2.3210	-2.8401
N3	-3.1969	-1.5371	-1.7303
N4	0.0769	-0.1787	-2.6019
N5	0.0000	3.1085	0.0000
N6	0.0000	9.0615	0.0000
N7	0.0000	6.5253	0.0000
N8	0.0000	2.0856	0.0000
N9	0.0000	0.7568	0.0000
N24	0.0000	1.2581	0.0000
N25	0.0000	6.6300	0.0000
N26	0.0000	5.7505	0.0000

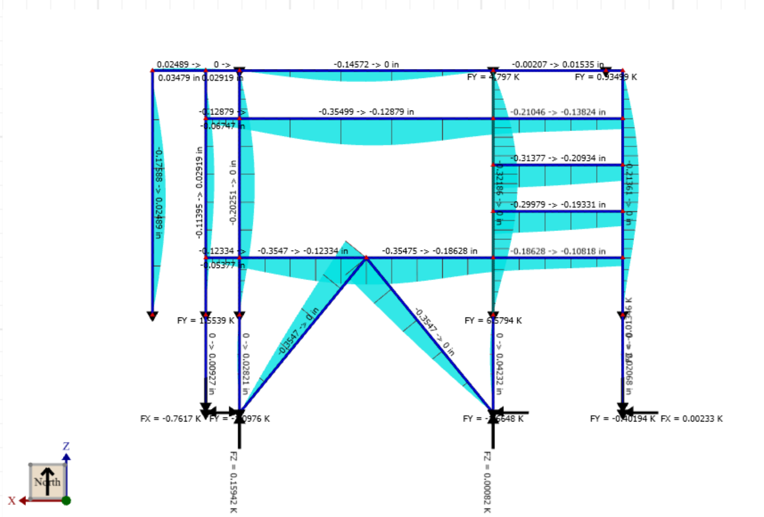
Visual Analysis Model Output:

UNITY



DEFLECTION
D+S (Dy)

Total L/240
Snow L/360



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Connections

Beam to Beam connection:

Worst case end reactions:

Member End Reactions (extreme rows: max and min)

Member	Result Case	Node	Fx K	Vy K	Mz K-ft
M6	6. 1.2D+E+L+0.2S »-X:Ω	N3	**-8.8957**	0.0000	0.0000
M6	6. 1.2D+E+L+0.2S »-X:Ω	N12	**-8.8957**	0.0000	0.0000
M7	6. 1.2D+E+L+0.2S »(-X+30%+Z):Ω	N11	-5.3247	-0.9363	**-3.7030**
M8	7. 0.9D+E »+X:Ω	N11	**7.8992**	0.6060	-2.5023
M8	7. 0.9D+E »+X:Ω	N12	**7.8992**	0.0553	-0.2221
M14	6. 1.2D+E+L+0.2S »(+Z+30%+X):Ω	N26	2.8871	**-2.7743**	4.7243
M19	3. 1.2D+1.6S+L	N21	-1.0115	**2.4512**	5.1768
M22	7. 0.9D+E »(+Z+30%+X):Ω	N26	1.7115	2.4379	**5.5620**

Check all around welds for rigid connections:

See screenshot below

Required Weld Size (strength) =
3/8" fillet or 1/4" pjp

WELD GROUP ANALYSIS
Using the Elastic Method for up to 24 Total Welds

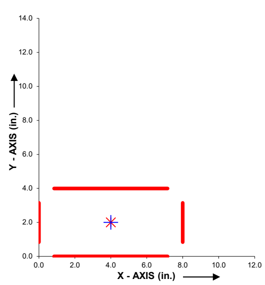
Job Name:	8480	Subject:	entry - rigid beam to beam connection
Job Number:		Originator:	haa Checker:

Input Data:

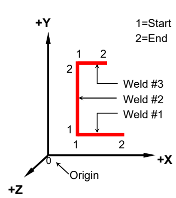
Number of Welds, Nw = 4

Weld Coordinates:

	Start X1 (in.)	Start Y1 (in.)	End X2 (in.)	End Y2 (in.)
Weld #1	0.844	0.000	7.157	0.000
Weld #2	0.844	4.000	7.157	4.000
Weld #3	0.000	0.844	0.000	3.150
Weld #4	8.000	0.844	8.000	3.150



WELD GROUP PLOT



NOMENCLATURE

No. of Load Points = 1

Load Point Data:

	Point #1
X-Coordinate (in.) =	4.000
Y-Coordinate (in.) =	2.000
Z-Coordinate (in.) =	0.000
Axial Load, Pz (k) =	-8.90
Shear Load, Px (k) =	6.50
Shear Load, Py (k) =	2.50
Moment, Mx (in-k) =	-34.80
Moment, My (in-k) =	156.00
Moment, Mz (in-k) =	67.20

(continued)

Results:

Weld Group Properties:

Lw =	17.239	in.
Xc =	4.000	in.
Yc =	1.999	in.
Ix =	52.55	in^3
Iy =	115.73	in^3
J =	168.28	in^3

Σ Loads @ C.G. of Weld Group:

Σ Pz =	-8.90	kips
Σ Px =	6.50	kips
Σ Py =	2.50	kips
Σ Mx =	-34.81	in-k
Σ My =	156.00	in-k
Σ Mz =	67.19	in-k

Weld Forces (k/in.)

	Fw(1)	Fw(2)
Weld #1	5.316	3.903
Weld #2	2.692	6.271
Weld #3	5.885	4.363
Weld #4	5.494	6.895

Required E70XX Weld Size:

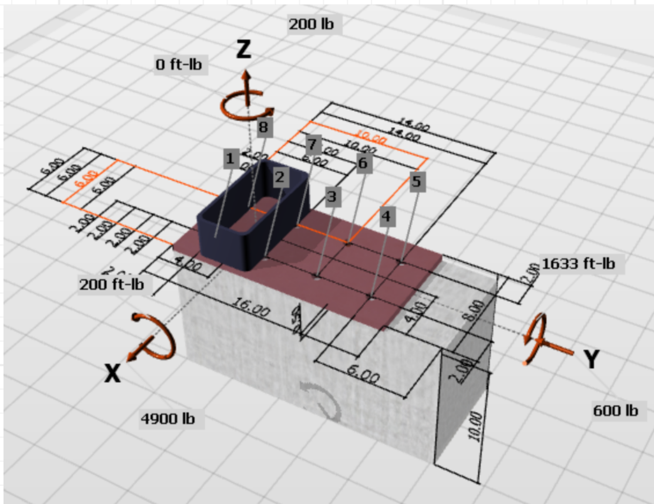
			Strength	
Fw(max) =	6.895	kips/in.		
Fillet (leg) =	0.464	in.	0.310	in.
Throat (eff) =	0.328	in.	0.219	in.

Entry Concrete end of wall conn.

VA Model Output:

Support Extreme Reactions

Node	Extreme FX K	Extreme FY K	Extreme FZ K
N1	0.1633	-0.5502	-2.8029
N2	5.5130	-2.3210	-5.4620
N3	-5.5979	-1.5371	-3.3737
N4	0.1420	-0.1787	-4.8976



Resulting Anchor Forces

#	Tension [lb]	Shear [lb]
1	0	1232
2	0	827
3	0	428
4	0	129
5	1006	279
6	986	495
7	966	863
8	945	1256

Shear Friction (aci 22.9):
Fy = 60ksi
Φ = 0.75 (shear)
μ = 0.7*1.0 = 0.7 (steel to concrete with stud/bars welded for shear transfer)
Max Vu = 3.5 k
Avf required = Vu / (Φ*fy*μ) = 3.5k / (0.75*60*0.7) = 0.12 in^2 --> No 4 bar = 0.2in^2 ok

Design bar for tension (aci 17.4)
Max Tu = 1k
ΦFy*As = 0.9*60ksi*0.2in^2 = 10.8k >1k ok

Check Weld
3/8" fillet: ΦRn = 8.352 k/in
8.352*3.14*0.5in = 13.1k ok

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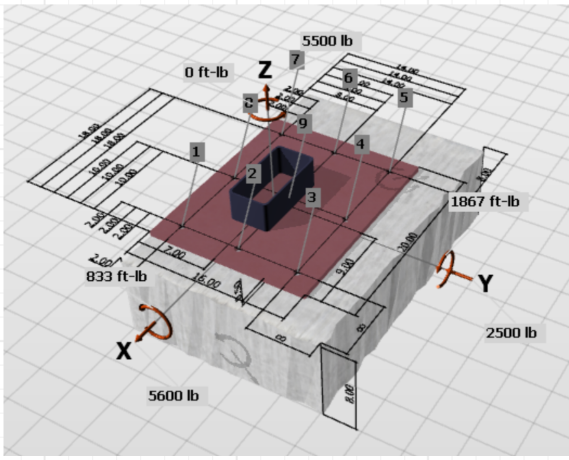
Entry Concrete end of wall conn.

VA Model Output:

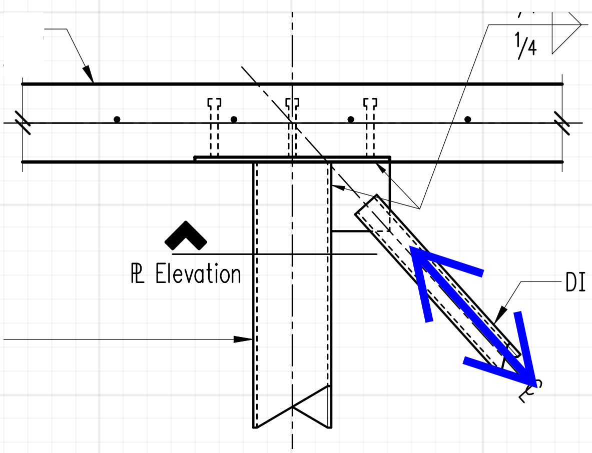
Support Extreme Reactions

Node	Extreme FX K	Extreme FY K	Extreme FZ K
N1	0.1633	-0.5502	-2.8029
N2	5.5130	-2.3210	-5.4620
N3	-5.5979	-1.5371	-3.3737
N4	0.1420	-0.1787	-4.8976



See Simpson Anchor output, next page

Brace Connection:



Brace forces from VA model
Fx = 9k (w/ Ω)

Check Weld
1/4" fillet: ΦRn = 5.568k/in
5.568*2*2in = 22k ok

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8480 Residence
PROJECT

DATE 10/12/2023
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DESIGN haa
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 318-19
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AWS Type A
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 6.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 7.50
C_{min} (inch): 1.38
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 8.00
State: Cracked
Compressive strength, f'_c (psi): 6000
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement not present
Supplemental edge 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): 20.00 x 16.00 x 0.75
Yield stress: 36000 psi

Profile type/size: HSS8X4X3/8

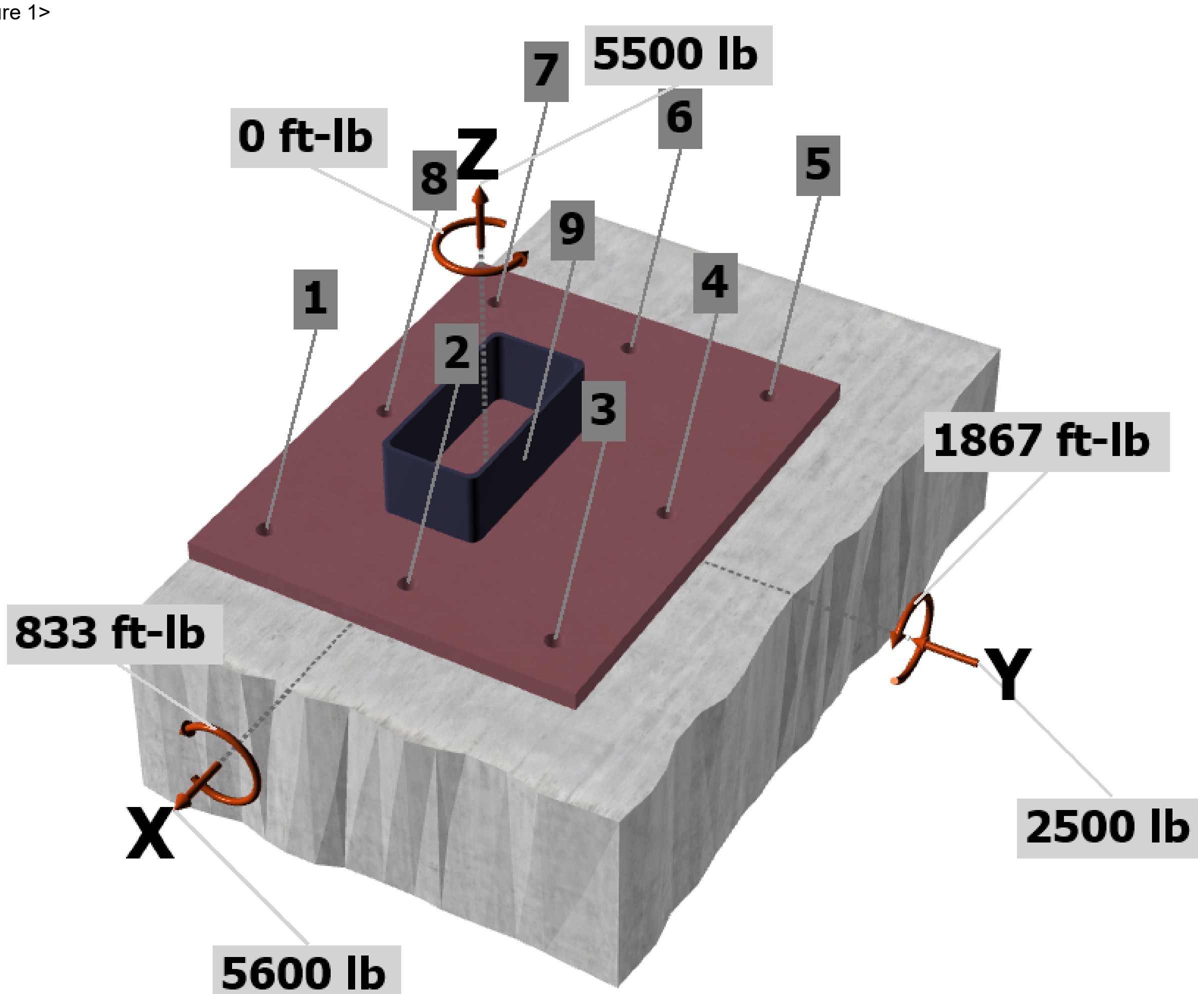
Recommended Anchor
Anchor Name: Headed Stud - 3/4"Ø AWS Type A Headed Stud

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: 17.10.5.2 not applicable
Ductility section for shear: 17.10.6.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

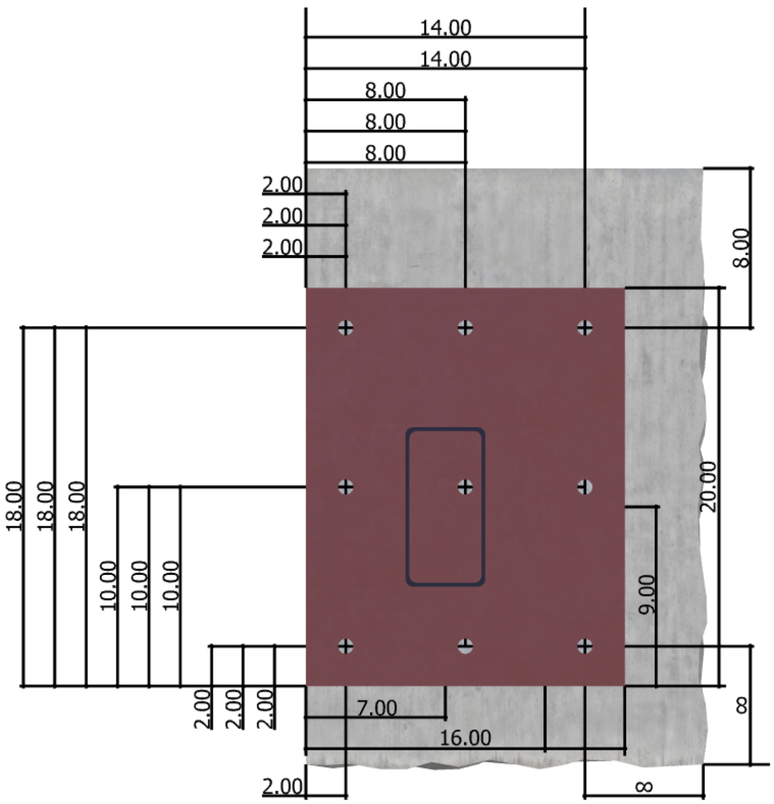
Strength level loads:

N_{ua} [lb]: 5500
V_{uax} [lb]: 5600
V_{uay} [lb]: -2500
M_{ux} [ft-lb]: 833
M_{uy} [ft-lb]: 1867
M_{uz} [ft-lb]: 0

<Figure 1>



<Figure 2>


14.00
14.00
8.00
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2.00
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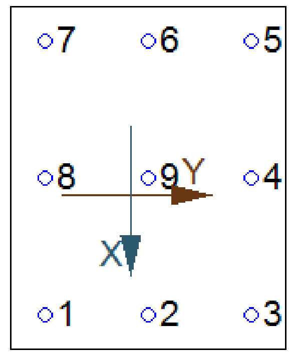
18.00
18.00
18.00
10.00
10.00
10.00
20.00
9.00
2.00
2.00
2.00
7.00
16.00
8
2.00
∞

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	134.3	653.2	-236.4	694.7
2	259.1	622.2	-236.4	665.6
3	383.9	591.2	-236.4	636.7
4	735.9	591.2	-277.8	653.2
5	1087.9	591.2	-319.1	671.8
6	963.1	622.2	-319.1	699.3
7	838.3	653.2	-319.1	727.0
8	486.3	653.2	-277.8	709.8
9	611.1	622.2	-277.8	681.4
Sum	5500.0	5600.0	-2500.0	6139.7

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 5500
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.82
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 3.07
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.46
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.21

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
26950	0.75	20213

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.6.2.2.1)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	6000	6.000	27322

$0.75\phi N_{cbg} = 0.75\phi (A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cbg}$ (lb)
759.00	324.00	2.00	0.683	0.767	1.00	1.000	27322	0.70	17608

6. Pullout Strength of Anchor in Tension (Sec. 17.6.3)

$0.75\phi N_{pn} = 0.75\phi \Psi_{c,P} N_p = 0.75\phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.5.1.2, Eq. 17.6.3.1 & 17.6.3.2.2a)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	0.79	6000	0.70	19782

7. Side-Face Blowout Strength of Anchor in Tension (Sec. 17.6.4)

$0.75\phi N_{sbg} = 0.75\phi\{(1+c_{a2}/c_{a1})/4\}(1+s/6c_{a1})N_{sb} = 0.75\phi\{(1+c_{a2}/c_{a1})/4\}(1+s/6c_{a1})(160c_{a1}\sqrt{A_{brg}})\lambda\sqrt{f'_c}$ (Sec. 17.5.1.2, Eq. 17.6.4.1 & 17.6.4.2)

s (in)	c_{a1} (in)	c_{a2} (in)	A_{brg} (in^2)	λ_a	f'_c (psi)	ϕ	$0.75\phi N_{sb}$ (lb)
16.00	2.00	8.00	0.79	1.00	6000	0.70	26903

8. Steel Strength of Anchor in Shear (Sec. 17.7.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
26950	1.0	0.65	17518

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.7.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.7.2.2.1a & Eq. 17.7.2.2.1b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
6.00	0.750	1.00	6000	14.00	36518

$\phi V_{cbgy} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.5.1.2 & Eq. 17.7.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgy} (lb)
360.00	882.00	0.964	0.814	1.000	1.620	36518	0.70	13264

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.7.2.2.1a & Eq. 17.7.2.2.1b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
6.00	0.750	1.00	6000	2.00	1972

$\phi V_{cbgy} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.5.1.2, 17.7.2.1(c) & Eq. 17.7.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
54.00	18.00	1.000	1.000	1.000	1.000	1972	0.70	8282

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.7.2.2.1a & Eq. 17.7.2.2.1b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
6.00	0.750	1.00	6000	8.00	15774

$\phi V_{cbgx} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.5.1.2, 17.7.2.1(c) & Eq. 17.7.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgx} (lb)
208.00	288.00	1.000	1.000	1.000	1.225	15774	0.70	19534

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.7.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.5.1.2 & Eq. 17.7.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	759.00	324.00	0.930	0.767	1.000	1.000	27322	0.70	63884

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	1088	20213	0.05	Pass
Concrete breakout	**5500**	**17608**	**0.31**	**Pass (Governs)**

Pullout	1088	19782	0.05	Pass
Side-face blowout	1459	26903	0.05	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	727	17518	0.04	Pass
T Concrete breakout y-	2500	13264	0.19	Pass
|| Concrete breakout y-	1960	8282	0.24	Pass
|| Concrete breakout x-	957	19534	0.05	Pass
Concrete breakout, combined	**-**	**-**	**0.24**	**Pass (Governs)**
Pryout	6133	63884	0.10	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.8.1	0.31	0.00	31.2%	1.0	Pass

3/4"Ø AWS Type A Headed Stud with hef = 6.000 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.535 inch

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 17.10.5.2 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 17.10.6.2 for shear need not be satisfied – designer to verify.

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

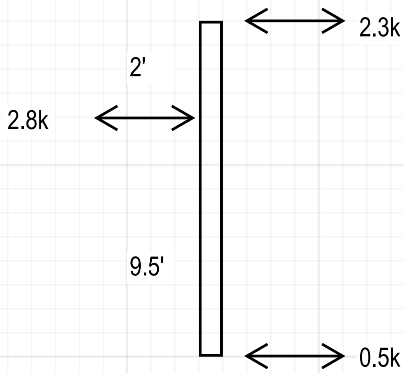
Entry concrete checks:

Support Reactions (w/out Ω)

Support Extreme Reactions

Node	Extreme FX K	Extreme FY K	Extreme FZ K
N1	0.0858	-0.5502	-1.4626
N2	3.1488	-2.3210	-2.8401
N3	-3.1969	-1.5371	-1.7303
N4	0.0769	-0.1787	-2.6019

Fx and Fy apply in plane, Check Fz out of plane:



Note: Top of wall braced per 19/S6.2 for
out of plane wall anchorage

Mmax = 4.6k-ft

Concrete 8" thick x 12" strip w/ #5 @ 12"oc ctrd.:
ΦMn = 5.4kft, ΦVn = 5.6k

Check Concrete column for out of plane load w/ 8" eccentricity:

Support Extreme Reactions

Node	Extreme FX K	Extreme FY K	Extreme FZ K
N1	0.1633	-0.5502	-2.8029
N2	5.5130	-2.3210	-5.4620
N3	-5.5979	-1.5371	-3.3737
N4	0.1420	-0.1787	-4.8976

ACI 22.7 Torsional Strength

Design Loads:

Tu	4.666666667	kft	Checking out of plane Fz w/ Ω=2.5 and eccentricity = (10"/2+6"/2) = e = 8" (between cl of col and cl of beam

Properties:

f'c	6000	psi	(nwc)
b	10	in	
w	12	in	
Acp	120	in^2	
pcp	44	in	
Nu	1250	lbs	
Ag	120	in^2	

Threshold Torsion (solid cross section)

non prestressed member:

Tth	25350.43645	lb-in
ΦTth	1.584402278	k-ft

non prestressed member subject to axial force

Tth	25773.05014	lb-in
ΦTth	1.610815634	k-ft

DCR (Tu/ΦTth) 2.897083048
IF DCR<1, neglect torsion
IF DCR>1, check torsion

Cracking Torsion

Non-prestressed Member

Tcr	101401.7458	lb-in
ΦTcr	6.337609112	k-ft

non prestressed member subject to axial force

Tcr	103092.2006	lb-in
ΦTcr	6.443262536	k-ft

DCR (Tu/ΦTcr) 0.724270762

Torsional Strength:

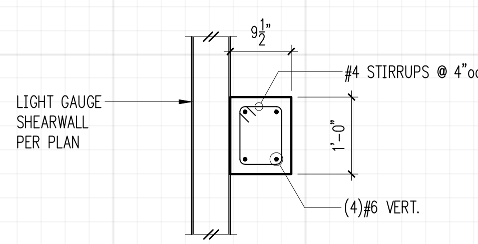
Assumptions:

Clear cover, cc	1.5	in
Vertical bar diameter,	0.75	in
Stirrup bar diameter	0.5	
Aoh	48	in^2
fyt	60	ksi
ph	30	in
theta	45	deg
s	9	in
Stirrups	0.20	no. 4
Vertical bars	1.77	(4) #6 bars
Tna	8.901179185	kft
Tnb	288.3982056	kft
ΦTn	6.675884389	kft
DCR (Tu/ΦTn)	0.699033476	

Cross Sectional Limits:

Assumptions:

Vu	5.6	k
Vc	14.9	k
d	9.625	in
first term:	0.432849669	
second term:	0.580947502	
Limit check	OK	
DCR	0.74507536	



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SSF STRUCTURAL ENGINEERING

PROJECT: 8480 Residence
DATE: 10/12/2023
PROJ. #: 01519-2021-09
DESIGN: haa
SHEET:

Check entry posts for out of plant wind

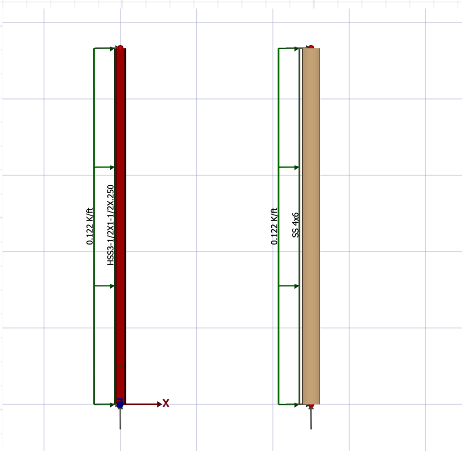
Note - gravity loads go into main house posts

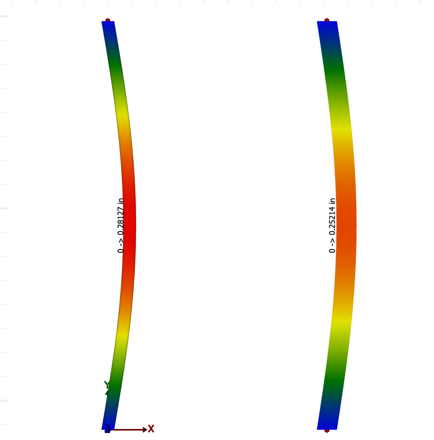
Determine Wind C&C Load:
Area = 110sf
For Wall in zone 5, design wind pressure = 18psf; -19.8psf

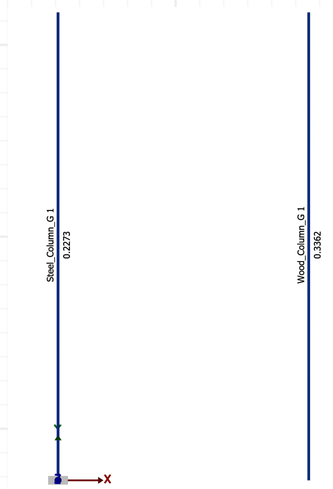
h = 9.33 ft

w = 19.8psf*12.25'/2 = 122plf

df 4x6 or hss 3.5x1.5x.25
(see va model)







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STRUCTURAL ENGINEERING

PROJECT	8480 Residence
DATE	08/01/2023
PROJ. #	01519-2021-09
DESIGN	haa
SHEET	

Entry Roof

Garage Lateral Updates

SWENSON SAY FAGÉT
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Lateral Design - N/S Direction (garage)

ROOF

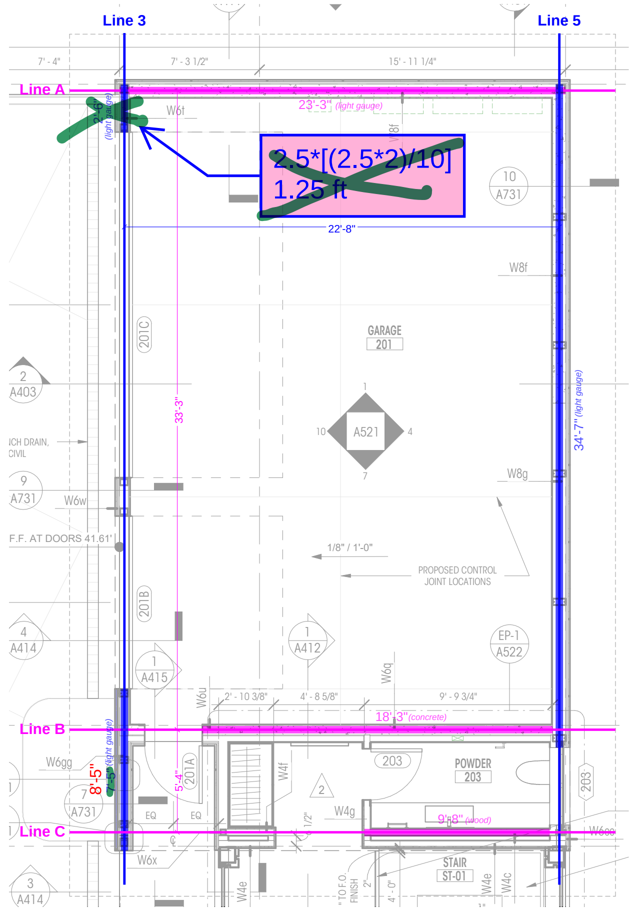
WIND --- Vx = 2.80 kips
Vx = 1.68 k, ASD
w = 1.68 k / 22.67 ft
w = 74 plf

EQ --- Vx = 33.50 kips, LRFD

Vx = 33.50 k * 0.7
Vx = 23.45 kips, ASD

Vx = 23.45 k * (5/6.5)
Vx = 18.04 k, ASD

w = 18.04 k / 22.67 ft
w = 796 plf



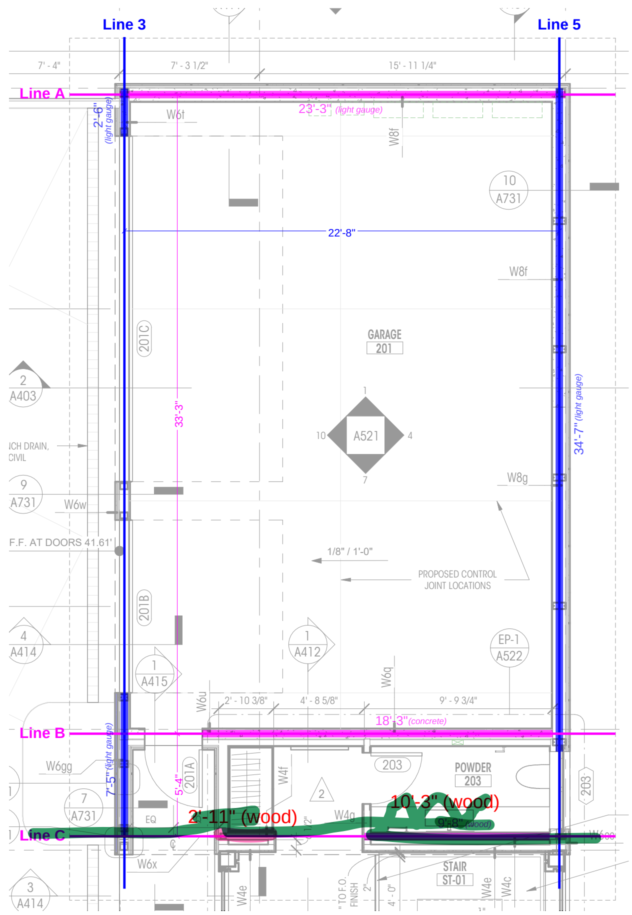
	Line 3	*Line 5*
V (k) W/EQ	0.84 / 11.62	0.84 / 11.62
V cum (k) W/EQ	0.84 / 11.62	0.84 / 11.62
L (ft)	8.42	34.58
V (plf) W/EQ	100 / 1381	25 / 336
SW type	(2)SW3 = SW5	SW1
OT (k)	34.52	8.40
OT cum. (k)	33.98	7.23
HD	(2) S/HDU9	S/HDU9

Lateral Design - E/W Direction (garage)

ROOF

WIND --- Vx = 4.82 kips
Vx = 2.89 k, ASD
w = 2.89 k / 38.67 ft
w = 75 plf

EQ --- Vx = 33.50 kips, LRFD
Vx = 33.50 k * 0.7
Vx = 23.45 kips, ASD
w = 23.45 k / 38.67 ft
w = 607 plf



	Line C	*Line B*	*Line A*
V (k) W/EQ	0.20 / 1.62	1.65 / 20.12	1.25 / 10.12
V cum (k) W/EQ	0.20 / 1.62	1.65 / 20.12	1.25 / 10.12
L (ft)	13.17 (11.95 red)	18.25	23.25
V (plf) W/EQ	16 / 136	-	54 / 436
SW type	W6	concrete	SW2
OT (k)	1.23	-	10.88
OT cum. (k)	-	-	9.61
HD	-	-	S/HDU9

See attached spreadsheet and spcol for concrete shearwall

8480 Wu
PROJECT

9/13/2022
DATE
01519-2021-09
PROJ. #
LAN
DESIGN
XX
SHEET

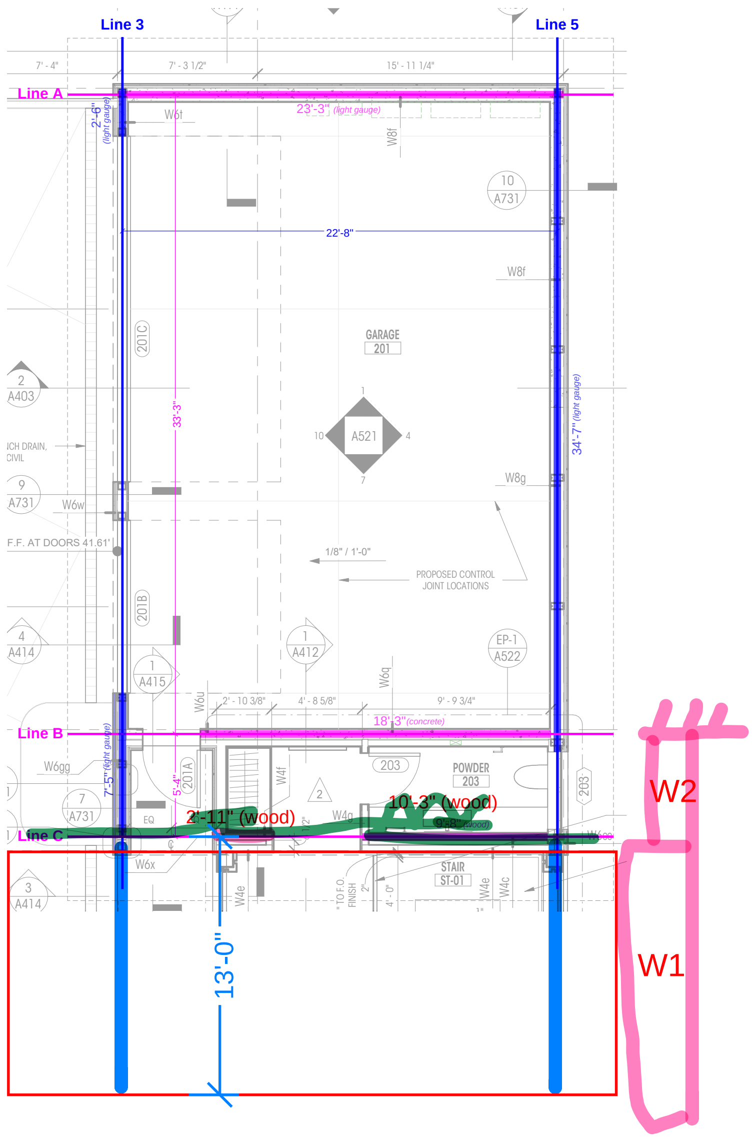
STRUCTURAL ENGINEERING

Lateral Design - E/W Direction (garage)

ROOF

WIND --- Vx = 4.82 kips
Vx = 2.89 k, ASD
w = 2.89 k / 38.67 ft
w = 75 plf

EQ --- Vx = 33.50 kips, LRFD
Vx = 33.50 k * 0.7
Vx = 23.45 kips, ASD
w = 23.45 k / 38.67 ft
w = 607 plf



CANTILEVER LENGTH = 13.00+5.33 = 18.33 FT
LOAD IN CANTILEVER = 10.00 KIPS
W1 = 520 PLF
W2 = 607 PLF
MOMENT FROM CANTILEVER = 89 KIP-FT
MOMENT ARM = 22.67 FT
T/C = 3.93 KIPS
^^ DOES NOT GOVERN FOR NS WALLS

FOR CHORD DESIGN: 3.93*2.5 = 9.82 KIPS
CHORD = use steel beam

Lateral Design - E/W Direction (garage)

CONCRETE WALL DESIGN

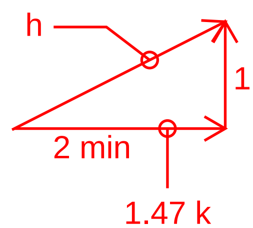
Line B out of plane shear
USE KICKER SPACED AT 4'-0"
wt = 11.75*0.5*4*0.67*150 = 2350 lbs
Fp eq. = 0.4*1.172*1*2350 = 1.11 kips, LRFD
Fp min. = 0.1*2350 = 0.24 kips, LRFD < 1.11
Fp = 1.11 kips at mid height of wall

moment = 1.11k*(11.75ft/2) = 6.52 k-ft per 4ft
moment = 0.28k*(11.75ft/2) = 1.65 k-ft per 1ft

capacity:
cover = 4 in, db min = min(3.25,2.75) = 2.75 in, bw = 12 in, As = 0.2 in^2
a = (0.2*60)/(0.85*4*12) = 0.294 in
ϕMn = 0.9*0.2*60*(2.75-(0.294/2)) = 28.11 k-ft > 1.65 k-ft

shear = 1.11 kips --> capacity = 3.14 k so OK

Line B out of plane shear anchorage
L_f = 33.33 ft
k_a = 1+(33.33/100) = 1.33 < 2.0
trib = 4.00 ft
wt = 2350 lbs
F_p eq. = 0.4*1.172*1*1.33*2350 = 1.47 kips, LRFD
F_p min. = 0.2*1*1.33*2350 = 0.63 kips, LRFD < 1.47
F_p = 1.47 kips



use 2H min to 1V --> theta max = 26.6 degrees
h = 1.47 / cos(26.6) = 1.65 k max, LRFD
kicker = 800S162-43

attachment to concrete wall
1.47*0.7 = 1.03 kips ASD --> 0.145"dia drive pin capacity (1.5" embed) = 209 lbs
so need (5) drive pins

attachment to angle/ rafter
1.65*0.7 = 1.16 kips ASD --> #8 capacity = 244 lbs
so need (5)#8

load getting into diaphragm
for parallel to rafter case- falls onto blkg btwn joists so that is 2'-0"
1030/2 = 515 plf < 886 plf OK

ACI 318-14 Special Concrete Shear Wall Design

Wall Pier PB-3

Input Information — Boundary Element Design Method: Section 18.10.6.3 (Stress Method)

Material Properties

f'_c =	6000	psi
f_y =	60000	psi

Typical Wall Reinforcement

Vertical Reinforcement:	1	curtains of	#5	@	12	in o.c.
Horizontal Reinforcement:	1	curtains of	#5	@	12	in o.c.

Section Properties

h_w =	22.125	ft
l_w =	18.916667	ft
t_w =	8	in
Top Flange?	No	
$b_{f,top}$ =	0	in
$t_{f,top}$ =	0	in
Bot Flange?	No	
$b_{f,bot}$ =	0	in
$t_{f,bot}$ =	0	in
I_{gross} =	7798055	in^4
A_{gross} =	1816	in^2
S_{Bot} =	68705	in^3
S_{Top} =	68705	in^3

Section Properties (Cont'd)

Top BE Width =		in	or	Top BE Area =		in^2
Bottom BE Width =		in	or	Bot BE Area =		in^2
hu=	11.58333	ft				

Loads and Displacements

Wall Ultimate Shear Load, V_u=	27	k	m = 3
Wall Ultimate Axial Load, P_u+=	28	k	
Wall Ultimate Moment, M_u+=	226	k-ft	m = 6
Top of Wall Inelastic Displ, du=	0.25	in	

SP Column Note - Use negative moment for "c" calculation of "Top" B.E., use positive moment for calculation of "Bottom" B.E. - - The x-axis is the strong axis of the wall segment.

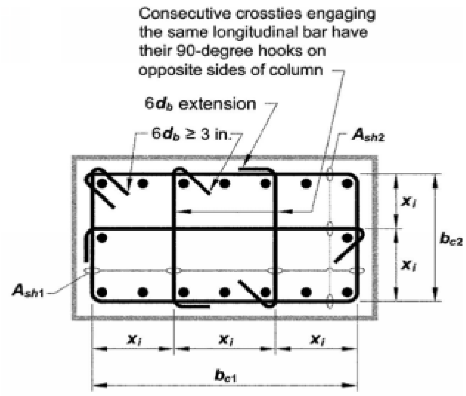
---> Max Ext Fiber Comp Stress, Left = 55 psi
↘ 0.009 f'c

---> Max Ext Fiber Comp Stress, Right = 55 psi
↘ 0.009 f'c

ACI 18.10.2 - Reinforcement

ρl=	0.0032	ρl, min=	0.0012
ρt=	0.0032	ρt,min=	0.0020

Check ACI 18.10.2.1 Min Longitudinal Reinforcement Ratio	**OK**
Check ACI 18.10.2.1 Min Transverse Reinforcement Ratio	**OK**
Check ACI 18.10.2.1 Maximum Bar Spacing	**OK**
Check ACI 18.10.2.2 Two Curtains of Reinforcement	**OK**
Check ACI 18.10.4.3 Horizontal To Vertical Reinforcement Ratio	**OK**



The dimension x_i from centerline to centerline of tie legs is not to exceed 14 inches. The term h_x used in equation 21-2 is taken as the largest value of x_i.

ACI 18.10.4 - Shear Strength

Acv=	1816	in2	ΦVn max=	464.3	kips (ACI 18.10.4.1)
α=	3.0		ΦVn max=	844.0	kips (ACI 18.10.4.4)

Check ACI 18.10.4.1 and 18.10.4.4 Shearwall Shear Strength **OK**

ACI 18.10.5 - Design for Flexure and Axial Loads

ΦMn (See SP Column Output)
ΦPn (See SP Column Output)
Check ACI Chapter 11 Shearwall Moment Strength
Check ACI Chapter 11 Shearwall Axial Strength

ACI 18.10.6 - Boundary Elements of Structural Walls

"Left" End of Wall BE

c= in (Use SP Column or other analysis)
No Special Boundary Element Required
Use B.E. Vert Reinforcement: 2
Check ACI 18.10.6.5 Tie Requirement **#N/A**

Use B.E. Length= in

"Right" End of Wall BE

c= in (Use SP Column or other analysis)
No Special Boundary Element Required
Use B.E. Vert Reinforcement: 2
Check ACI 18.10.6.5 Tie Requirement **#N/A**

Use B.E. Length= in

	Left End	Right End
Check ACI 18.10.6.4a Boundary Element Length	**N/A**	**N/A**
Check ACI 18.10.6.4b Slenderness of Boundary Element	**N/A**	**N/A**
Check ACI 18.10.6.4c Width of Boundary Element	**N/A**	**N/A**
Check ACI 18.10.6.4e Boundary Element Transverse Reinforcement Spacing	**N/A**	**N/A**
Check ACI 18.10.6.4f Boundary Element Ash2 Reinforcement Area	**N/A**	**N/A**
Check ACI 18.10.6.4f Boundary Element Ash1 Reinforcement Area	**N/A**	**N/A**

ACI 318-14 Special Concrete Shear Wall Design

Wall Pier PB-2

Input Information — Boundary Element Design Method: Section 18.10.6.2 (Displacement Method)

Material Properties

f'_c =	6000	psi
f_y =	60000	psi

Typical Wall Reinforcement

Vertical Reinforcement:	1	curtains of	#5	@	12	in o.c.
Horizontal Reinforcement:	1	curtains of	#5	@	12	in o.c.

Section Properties

h_w =	22.125	ft
l_w =	10.916667	ft
t_w =	8	in
Top Flange?	No	
$b_{f,top}$ =	0	in
$t_{f,top}$ =	0	in
Bot Flange?	No	
$b_{f,bot}$ =	0	in
$t_{f,bot}$ =	0	in
I_{gross} =	1498727	in^4
A_{gross} =	1048	in^2
S_{Bot} =	22881	in^3
S_{Top} =	22881	in^3

Section Properties (Cont'd)

Top BE Width =		in	or	Top BE Area =	in^2
Bottom BE Width =		in	or	Bot BE Area =	in^2
hu=	11.58333	ft			

Loads and Displacements

Wall Ultimate Shear Load, V_u=	43	k	m = 3
Wall Ultimate Axial Load, P_u+=	83	k	
Wall Ultimate Moment, M_u+=	191	k-ft	m = 6
Top of Wall Inelastic Displ, du=	0.25	in	

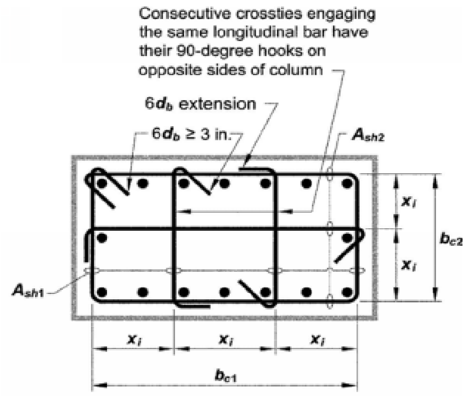
SP Column Note - Use negative moment for "c" calculation of "Top" B.E., use positive moment for calculation of "Bottom" B.E. - - The x-axis is the strong axis of the wall segment.

---> Max Ext Fiber Comp Stress, Left = 180 psi
Not Considered - Using Displacement Method ↘ 0.030 f'c
---> Max Ext Fiber Comp Stress, Right = 180 psi
Not Considered - Using Displacement Method ↘ 0.030 f'c

ACI 18.10.2 - Reinforcement

ρl=	0.0032	ρl, min=	0.0012
ρt=	0.0032	ρt,min=	0.0020

Check ACI 18.10.2.1 Min Longitudinal Reinforcement Ratio **OK**
Check ACI 18.10.2.1 Min Transverse Reinforcement Ratio **OK**
Check ACI 18.10.2.1 Maximum Bar Spacing **OK**
Check ACI 18.10.2.2 Two Curtains of Reinforcement **OK**
Check ACI 18.10.4.3 Horizontal To Vertical Reinforcement Ratio **OK**



The dimension x_i from centerline to centerline of tie legs is not to exceed 14 inches. The term h_x used in equation 21-2 is taken as the largest value of x_i.

ACI 18.10.4 - Shear Strength

Acv=	1048	in2	ΦVn max=	219.2	kips	(ACI 18.10.4.1)
α=	2.0		ΦVn max=	487.1	kips	(ACI 18.10.4.4)

Check ACI 18.10.4.1 and 18.10.4.4 Shearwall Shear Strength **OK**

ACI 18.10.5 - Design for Flexure and Axial Loads

ΦMn (See SP Column Output)
ΦPn (See SP Column Output)
Check ACI Chapter 11 Shearwall Moment Strength
Check ACI Chapter 11 Shearwall Axial Strength

ACI 18.10.6 - Boundary Elements of Structural Walls

"Left" End of Wall BE

c= 4.94 in (Use SP Column or other analysis)
No Special Boundary Element Required
Use B.E. Vert Reinforcement: 2
Check ACI 18.10.6.5 Tie Requirement **#N/A**

Use B.E. Length= in

"Right" End of Wall BE

c= 4.94 in (Use SP Column or other analysis)
No Special Boundary Element Required
Use B.E. Vert Reinforcement: 2
Check ACI 18.10.6.5 Tie Requirement **#N/A**

Use B.E. Length= in

	Left End	**Right End**
Check ACI 18.10.6.4a Boundary Element Length	**N/A**	**N/A**
Check ACI 18.10.6.4b Slenderness of Boundary Element	**N/A**	**N/A**
Check ACI 18.10.6.4c Width of Boundary Element	**N/A**	**N/A**
Check ACI 18.10.6.4e Boundary Element Transverse Reinforcement Spacing	**N/A**	**N/A**
Check ACI 18.10.6.4f Boundary Element Ash2 Reinforcement Area	**N/A**	**N/A**
Check ACI 18.10.6.4f Boundary Element Ash1 Reinforcement Area	**N/A**	**N/A**

ACI 318-14 Special Concrete Shear Wall Design

Wall Pier PB-1

Input Information Boundary Element Design Method: Section 18.10.6.2 (Displacement Method)

Material Properties			Typical Wall Reinforcement						
f'_c =	6000	psi	Vertical Reinforcement:	1	curtains of	#5	@	12	in o.c.
f_y =	60000	psi	Horizontal Reinforcement:	1	curtains of	#5	@	12	in o.c.

Section Properties			Section Properties (Cont'd)						
h_w =	22.125	ft	Top BE Width =		in	or	Top BE Area =		in^2
l_w =	8.625	ft	Bottom BE Width =		in	or	Bot BE Area =		in^2
t_w =	8	in	hu=	11.58333	ft				
Top Flange?	No								
$b_{f,top}$ =	0	in	Loads and Displacements						
$t_{f,top}$ =	0	in	Wall Ultimate Shear Load, V_u=	38	k	m = 3			
Bot Flange?	No		Wall Ultimate Axial Load, P_u+=	61	k				
$b_{f,bot}$ =	0	in	Wall Ultimate Moment, M_u+=	130	k-ft	m = 6			
$t_{f,bot}$ =	0	in	Top of Wall Inelastic Displ, du=	0.25	in				
I_{gross} =	739145	in^4							
A_{gross} =	828	in^2							
S_{Bot} =	14283	in^3							
S_{Top} =	14283	in^3							

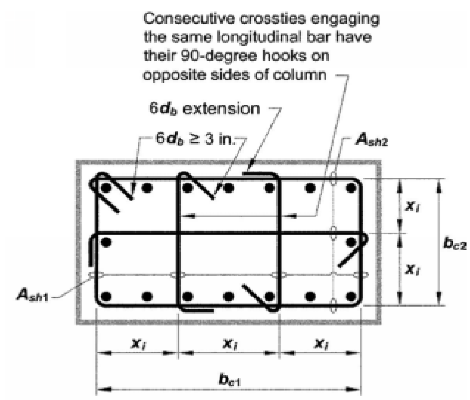
SP Column Note - Use negative moment for "c" calculation of "Top" B.E., use positive moment for calculation of "Bottom" B.E. - - The x-axis is the strong axis of the wall segment.

---> Max Ext Fiber Comp Stress, Left = 183 psi
Not Considered - Using Displacement Method ↘ 0.030 f'c
---> Max Ext Fiber Comp Stress, Right = 183 psi
Not Considered - Using Displacement Method ↘ 0.030 f'c

ACI 18.10.2 - Reinforcement

ρl=	0.0032	ρl, min=	0.0012
ρt=	0.0032	ρt,min=	0.0020

Check ACI 18.10.2.1 Min Longitudinal Reinforcement Ratio **OK**
Check ACI 18.10.2.1 Min Transverse Reinforcement Ratio **OK**
Check ACI 18.10.2.1 Maximum Bar Spacing **OK**
Check ACI 18.10.2.2 Two Curtains of Reinforcement **OK**
Check ACI 18.10.4.3 Horizontal To Vertical Reinforcement Ratio **OK**



The dimension x_i from centerline to centerline of tie legs is not to exceed 14 inches. The term h_x used in equation 21-2 is taken as the largest value of x_i.

ACI 18.10.4 - Shear Strength

Acv=	828	in2	ΦVn max=	173.2	kips (ACI 18.10.4.1)
α=	2.0		ΦVn max=	384.8	kips (ACI 18.10.4.4)

Check ACI 18.10.4.1 and 18.10.4.4 Shearwall Shear Strength **OK**

ACI 18.10.5 - Design for Flexure and Axial Loads

ΦMn (See SP Column Output)
ΦPn (See SP Column Output)
Check ACI Chapter 11 Shearwall Moment Strength
Check ACI Chapter 11 Shearwall Axial Strength

ACI 18.10.6 - Boundary Elements of Structural Walls

"Left" End of Wall BE

c= 4.94 in (Use SP Column or other analysis)
No Special Boundary Element Required
Use B.E. Vert Reinforcement: 2
Check ACI 18.10.6.5 Tie Requirement **#N/A**

Use B.E. Length= in

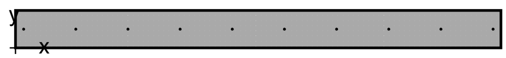
"Right" End of Wall BE

c= 4.94 in (Use SP Column or other analysis)
No Special Boundary Element Required
Use B.E. Vert Reinforcement: 2
Check ACI 18.10.6.5 Tie Requirement **#N/A**

Use B.E. Length= in

	Left End	Right End
Check ACI 18.10.6.4a Boundary Element Length	**N/A**	**N/A**
Check ACI 18.10.6.4b Slenderness of Boundary Element	**N/A**	**N/A**
Check ACI 18.10.6.4c Width of Boundary Element	**N/A**	**N/A**
Check ACI 18.10.6.4e Boundary Element Transverse Reinforcement Spacing	**N/A**	**N/A**
Check ACI 18.10.6.4f Boundary Element Ash2 Reinforcement Area	**N/A**	**N/A**
Check ACI 18.10.6.4f Boundary Element Ash1 Reinforcement Area	**N/A**	**N/A**

spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections



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List of Figures

Figure 1: Column section 4

1. General Information

File Name	K:\2021\01519-2...\Concrete shearwall Grid B.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	6	ksi
E_c	4415.21	ksi
f_c	5.1	ksi
ε_u	0.003	in/in
β_1	0.75	

2.2. Steel

Type	Standard	
f_y	60	ksi
E_s	29000	ksi
ε_{ty}	0.00206897	in/in

3. Section

3.1. Shape and Properties

Type	Irregular	
A_g	828	in^2
I_x	4416	in^4
I_y	739145	in^4
r_x	2.3094	in
r_y	29.8779	in
X_o	51.75	in
Y_o	4	in

3.2. Section Figure

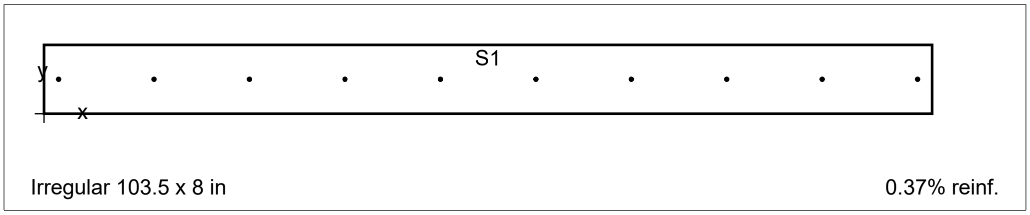


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X	Y	Points	X	Y	Points	X	Y
	in	in		in	in		in	in
1	0.0	8.0	2	0.0	0.0	3	103.5	0.0
4	103.5	8.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	in	in^2		in	in^2		in	in^2
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	3.10 in^2
Rho	0.37 %
Minimum clear spacing	10.50 in

(Note: Rho < 0.50%)

4.4. Bars Provided

Area in^2	X in	Y in	Area in^2	X in	Y in	Area in^2	X in	Y in
0.31	1.7	4.0	0.31	12.8	4.0	0.31	23.9	4.0
0.31	35.1	4.0	0.31	46.2	4.0	0.31	57.3	4.0
0.31	68.4	4.0	0.31	79.6	4.0	0.31	90.7	4.0
0.31	101.8	4.0						

5. Control Points

About	Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d_t Depth in	ε_t	ϕ
X	@ Max compression	2855.4	0.00	0.00	12.89	4.00	-0.00207	0.65000
X	@ Allowable comp.	2284.4	145.30	0.00	8.55	4.00	-0.00160	0.65000
X	@ f_s = 0.0	1029.3	214.44	0.00	4.00	4.00	0.00000	0.65000
X	@ f_s = 0.5 f_y	704.9	183.99	0.00	2.97	4.00	0.00103	0.65000
X	@ Balanced point	488.3	157.99	0.00	2.37	4.00	0.00207	0.65000
X	@ Tension control	362.5	152.00	0.00	1.49	4.00	0.00507	0.90000
X	@ Pure bending	0.0	53.34	0.00	0.47	4.00	0.02254	0.90000
X	@ Max tension	-167.4	0.00	0.00	0.00	4.00	9.99999	0.90000
Y	@ Max compression	2855.4	0.00	0.00	328.06	101.81	-0.00207	0.65000
Y	@ Allowable comp.	2284.4	0.00	1949.16	111.07	101.81	-0.00025	0.65000
Y	@ f_s = 0.0	2094.4	0.00	2391.50	101.81	101.81	0.00000	0.65000
Y	@ f_s = 0.5 f_y	1548.6	0.00	3087.33	75.71	101.81	0.00103	0.65000
Y	@ Balanced point	1212.5	0.00	3121.66	60.26	101.81	0.00207	0.65000
Y	@ Tension control	996.9	0.00	3583.34	37.85	101.81	0.00507	0.90000
Y	@ Pure bending	0.0	0.00	696.51	4.94	101.81	0.05881	0.90000
Y	@ Max tension	-167.4	0.00	0.00	0.00	101.81	9.99999	0.90000
-X	@ Max compression	2855.4	0.00	0.00	12.89	4.00	-0.00207	0.65000
-X	@ Allowable comp.	2284.4	-145.30	-0.01	8.55	4.00	-0.00160	0.65000
-X	@ f_s = 0.0	1029.3	-214.44	-0.01	4.00	4.00	0.00000	0.65000
-X	@ f_s = 0.5 f_y	704.9	-183.99	-0.01	2.97	4.00	0.00103	0.65000
-X	@ Balanced point	488.3	-157.99	-0.01	2.37	4.00	0.00207	0.65000
-X	@ Tension control	362.5	-152.00	-0.01	1.49	4.00	0.00507	0.90000
-X	@ Pure bending	0.0	-53.34	-0.01	0.47	4.00	0.02254	0.90000
-X	@ Max tension	-167.4	0.00	0.00	0.00	4.00	9.99999	0.90000
-Y	@ Max compression	2855.4	0.00	0.00	328.06	101.81	-0.00207	0.65000
-Y	@ Allowable comp.	2284.4	0.00	-1949.16	111.07	101.81	-0.00025	0.65000
-Y	@ f_s = 0.0	2094.4	0.00	-2391.50	101.81	101.81	0.00000	0.65000
-Y	@ f_s = 0.5 f_y	1548.6	0.00	-3087.33	75.71	101.81	0.00103	0.65000
-Y	@ Balanced point	1212.5	0.00	-3121.66	60.26	101.81	0.00207	0.65000
-Y	@ Tension control	996.9	0.00	-3583.34	37.85	101.81	0.00507	0.90000
-Y	@ Pure bending	0.0	0.00	-696.51	4.94	101.81	0.05881	0.90000
-Y	@ Max tension	-167.4	0.00	0.00	0.00	101.81	9.99999	0.90000

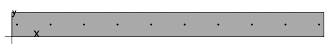
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Critical Capacity" Method.

No.	Demand			Capacity			Parameters at Capacity			Capacity Ratio
	P_u kip	M_{ux} k-ft	M_{uy} k-ft	ϕP_n kip	ϕM_{nx} k-ft	ϕM_{ny} k-ft	NA Depth in	ε_t	ϕ	
1	500.00	0.00	191.00	0.00	0.00	696.51	4.94	0.05881	0.900	0.74
2	0.00	0.00	191.00	0.00	0.00	696.51	4.94	0.05881	0.900	0.86

7. Diagrams

7.1. PM at θ=0 [deg]



103.5 x 8 in

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials

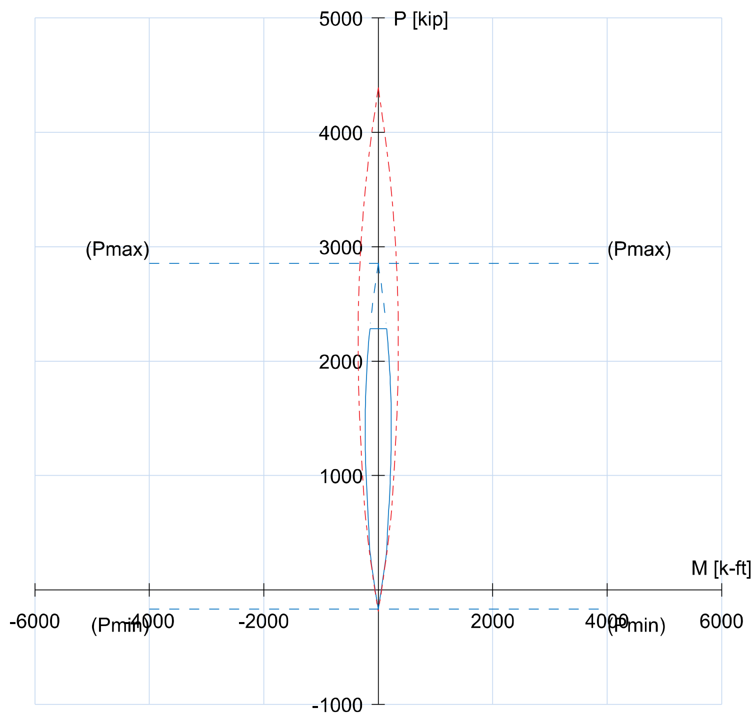
f'_c	6 ksi
E_c	4415.21 ksi
f_y	60 ksi
E_s	29000 ksi

Section

Type	Irregular
A_g	828 in²
I_x	4416 in⁴
I_y	739145 in⁴

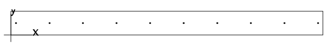
Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	3.10 in²
Rho	0.37 %
Min. clear spacing	10.50 in

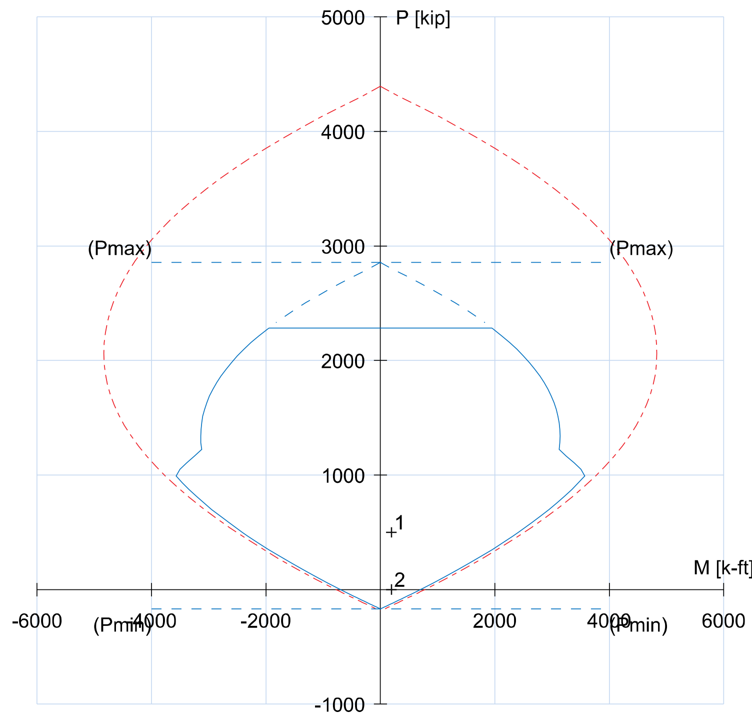


PM at 0.0 [deg]

7.2. PM at θ=90 [deg]



103.5 x 8 in



PM at 90.0 [deg]

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials

f'_c	6 ksi
E_c	4415.21 ksi
f_y	60 ksi
E_s	29000 ksi

Section

Type	Irregular
A_g	828 in²
I_x	4416 in⁴
I_y	739145 in⁴

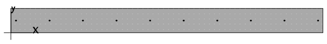
Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	3.10 in²
Rho	0.37 %
Min. clear spacing	10.50 in

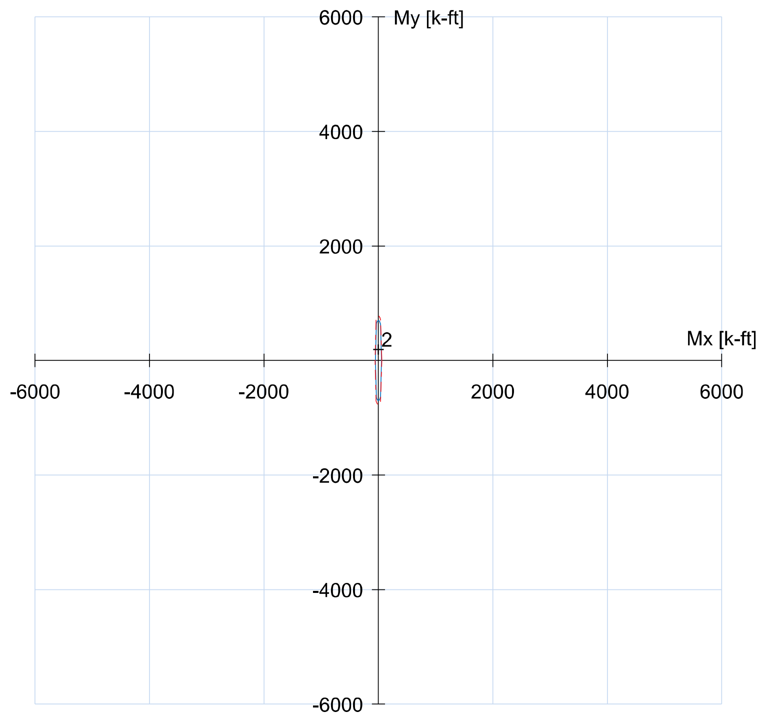
No.	P_u	M_{ux}	M_{uy}	ϕP_n	ϕM_{nx}	ϕM_{ny}	Capacity
	kip	k-ft	k-ft	kip	k-ft	k-ft	Ratio
2	0.0	0.0	191.0	0.00	0.00	696.51	0.86
1	500.0	0.0	191.0	0.00	0.00	696.51	0.74

Max. Capacity Ratio: 0.86

7.3. MM at P=0 [kip]



103.5 x 8 in



MM at P=0.0 [kip]

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials

f'_c	6 ksi
E_c	4415.21 ksi
f_y	60 ksi
E_s	29000 ksi

Section

Type	Irregular
A_g	828 in²
I_x	4416 in⁴
I_y	739145 in⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	3.10 in²
Rho	0.37 %
Min. clear spacing	10.50 in

No.	P_u	M_{ux}	M_{uy}	ϕP_n	ϕM_{nx}	ϕM_{ny}	Capacity
	kip	k-ft	k-ft	kip	k-ft	k-ft	Ratio
2	0.0	0.0	191.0	0.00	0.00	696.51	0.86

Max. Capacity Ratio: 0.86