PMA | Engineering

STRUCTURAL DESIGN CALCULATIONS
For

M. Aden Monheiser and Jennifer A. Monheiser

Lee's Summit, Missouri
Jackson County

Permit
December 2, 2022

December 2,2022

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Professional Engineer
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Page McNaghten Associates, Inc.
Professional Engineering Corporation
Missouri State Certificate of Authority #001400

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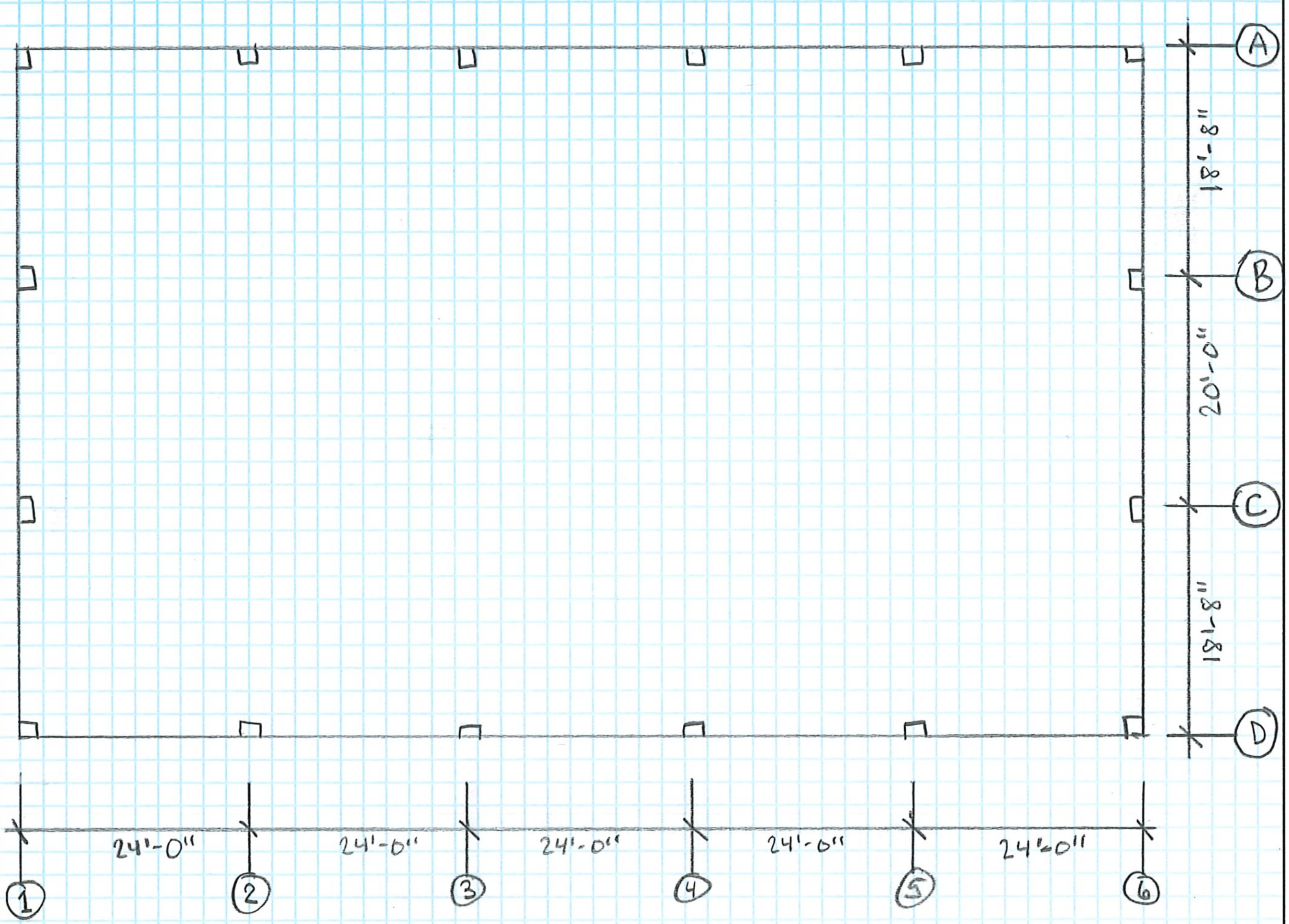
Project: P22221 Aden Monheiser PEMB | Sheet ___ of ___



Frame (Grids 2-5)

Vert Loads → DL = 3.2k LL = 15.9k SL = 15.1k WL = -17.3k ↑ EL = 0.2k

Horiz Loads → DL = 1.5k LL = 8.2k SL = 7.8k WL = -11.8k EL = 0.4k

Vert	(ASD)	Horiz	
4.5k	(3.2k)	2.1k	1.4D
36.83k	(19.1k)	18.8k	1.2D + 1.6L + 0.5S
43.9k	(18.3k)	22.5k	1.2D + 1.6S + L
27.3k	(26.45k)	13.9k	1.2D + L + 0.5S
-14.4k ↑	(-8.46k) ↑	-13.2k	0.9D + W

LRFD Design

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Assume Bearing Pressure → 2,000 PSF

Vert Load = 26.5^k → 26,500 LB

26.5 / 2 = 13.25 SF → $\sqrt{13.25}$ = 3.64'

Need min 4'-0" x 4'-0" spread footing = M4.0

Endwall Col's

			DL	LL	SL	WL	After LC
Grid 1 –	Corners →	Vertical =	0.5^k	2.0^k	1.9^k	4.1^k	→ 5.64^k
		Horiz =	—	—	—	4.7^k	→ 4.25^k
	Middle →	Vert =	1.4^k	6.0^k	5.7^k	7.8^k	→ 16.8^k
		Horiz =	—	—	—	11.3^k	→ -10^k (-6.78^k)
Grid 6 –	Corners →	Vert =	0.6^k	2.6^k	2.5^k	3.2^k	→ 7.3^k
		Horiz =	—	—	—	5.7^k	→ 5.2^k
	Middle →	Vert =	1.2^k	6.0^k	5.1^k	8.1^k	→ 15.6^k
		Horiz =	—	—	—	13.8^k	→ 12.7^k (-8.28^k)

Corners → 7.3^k / 2 KSF = 3.65SF $\sqrt{3.65}$ = 1.91'

Need 2' x 2' MIN = M3.0

Middle → 16.8^k / 2KSF = 8.4 SF $\sqrt{8.4}$ = 2.9'

Need 3' x 3' MIN = M3.0

Check Uplift Loads

Uplift on grids 2-5 → 14.4^k = 14,400 LB

M5.0 = 5' x 5' x 2.5' = 62.5 ft^3 Concrete = 150 pcf

150 x 62.5 = 9375 lb

Grade Beam = 2.5' x 1.5' = 3.75 ft^2 150 x 3.75 = 562.5 plf

Need 5025 lb more → 5025 / 562.5 = 8.9'

24' between columns → Uplift is OK

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Uplift on corners → $4.1^k - 0.9(0.5) = 3.65^k$

3650 LB

$M3.0 = 3' \times 3' \times 2.5' = 22.5\ ft^3 \times 150\ pcf = 3375\ lb$

∴ Uplift is OK (From grade beam)

Uplift on ends in middle → $8.1^k - 0.9(1.2) = 7.0^k$

7000 LB

$M3.0 = 22.5\ ft^3 \times 150\ pcf = 3375\ lb$

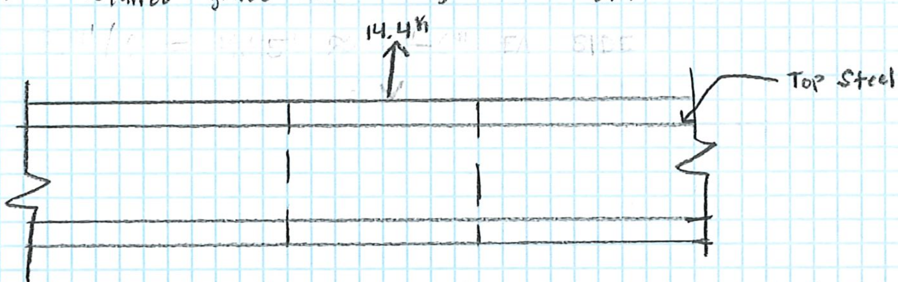
Grade Beam = 562.5 plf

$7000 - 3375 = 3625\ lb / 562.5 = 6.4'$ needed < 18'-8" Avail.

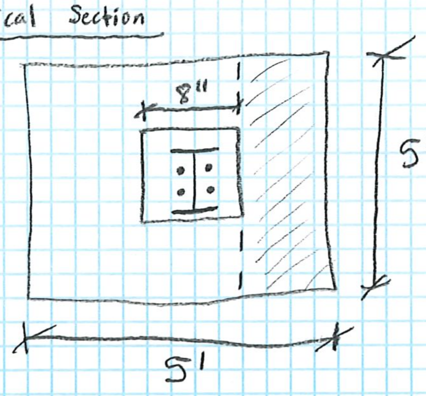
∴ Uplift is OK

Check FTG Top Steel for the Uplift Moment

Max required grade beam length → 8.9'



Critical Section



$P_u = 14.4^k$

$q_u = \frac{14.4^k}{25\ ft^2} = 576\ psf$

$M_u = q_u\left(\frac{L-c}{2}\right)^2 (b)/2$

$= 576\left(\frac{5' - 2'\text{-}2"}{2}\right)^2 (5')/2$

$= 2.89\ k\text{-}ft$

$M_u = 2.89$ k-ft

$a = \frac{A_s (60\,ksi)}{0.85(4.5)(5')} = 0.26 A_s$

$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right)$

$2.89\ k\text{-}ft \leq \phi A_s f_y \left(d - \frac{0.26 A_s}{2}\right)$

Try $A_{s\,min} = .0018(5')(\frac{12\,in}{ft})(30\,in)$

$= \underline{3.24\,in^2}$

$2.89 \leq 0.9(3.24)(60)\left(27'' - \frac{0.26(3.24)}{2}\right)$

$2.89 \leq 387.5$ k-ft ✓

Use (11) #5 bars Eq. Spaced

$\#5 = 0.31\,in^2 \times 11 = 3.41\,in^2 > 3.24\,in^2$ ✓

Only using the (2) #5's from grade beam → $A_s = 0.62\,in^2$

$\underline{\phi M_n = 75.1\ k\text{-}ft}$ ✓

Use (5) #8's for constructability ✓

Check Sliding on Foundations

From IBC 2018 Table 1806.2 → 2,000 PSF Bearing, 150 psf Lateral, 0.25 coefficient of friction

Will hair pin ties work? 4" slab = 50 psf × 0.25 = 12.5 psf

Available slab = 24'-0" × 28'-8" = 688 ft^2 × 12.5 = 8600 lb or 8.6 k

8.6k < 22.5 k → N.G.

Will need thickened slab with lateral ties.

For end columns on grids 1 & 6 → Horiz Force = 8.3k

Calculate As required to resist sliding → 1.2(1.5) + 1.6(7.8) + 8.2 = 22.5k

$F = A_s f_y$ $\quad 22.5^k = \phi A_s (60 \text{ ksi})$ → $A_s = 0.417 \text{ in}^2$ / 2 bars

= 0.2083 in^2 req'd per bar

$\boxed{\text{Use (2) \#5's as ties}}$

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Design:	Concrete - Nov 18, 2022	Date:	11/18/2022
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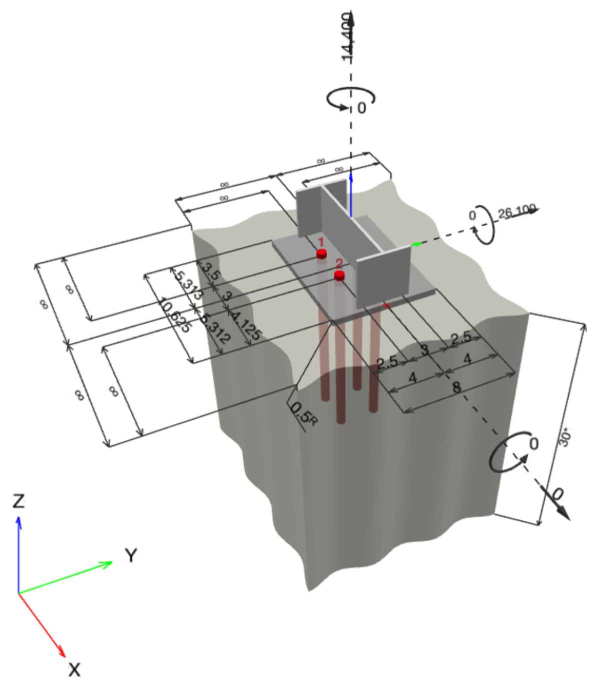
Specifier's comments:

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 55 3/4**
Item number:	not available
Effective embedment depth:	h_{ef} = 14.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 10.625 in. x 8.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	W shape (AISC), W10X15; (L x W x T x FT) = 9.990 in. x 4.000 in. x 0.230 in. x 0.270 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 30.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



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1.1 Design results

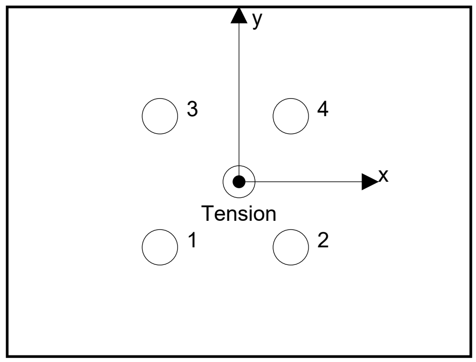
Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 14,400; V_x = 0; V_y = 26,100; M_x = 0; M_y = 0; M_z = 0;	no	75

2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	2,849	5,884	681	5,844
2	4,351	7,238	681	7,206
3	2,849	5,884	-681	5,844
4	4,351	7,238	-681	7,206



max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 14,400 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	4,351	18,787	24	OK
Pullout Strength*	4,351	14,650	30	OK
Concrete Breakout Failure**	14,400	65,157	23	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

$N_{sa} = A_{se,N}\, f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

N_{sa} [lb]
25,050

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
25,050	0.750	18,787	4,351

3.2 Pullout Strength

$N_{pN} = \psi_{c,p}\, N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8\, A_{brg}\, f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi\ N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.65	1.000	4,000

Calculations

N_p [lb]
20,928

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
20,928	0.700	14,650	4,351

3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16\ \lambda_a \sqrt{f'_c}\ h_{ef}^{5/3}$ — ACI 318-19 Eq. (17.6.2.2.3)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
14.000	0.313	0.000	∞	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
2,025.00	1,764.00	0.985	1.000	1.000	1.000	82,293

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
93,082	0.700	65,157	14,400

4 Shear load

	Load V_{ua} [lb]	Capacity $\phi\ V_n$ [lb]	Utilization $\beta_V = V_{ua}/\phi\ V_n$	Status
Steel Strength*	7,238	9,769	75	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	26,100	130,314	21	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

V_{sa} [lb]
15,030

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
15,030	0.650	9,769	7,238

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4.2 Pryout Strength

$$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b \right]$$ ACI 318-19 Eq. (17.7.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}} \right) \leq 1.0$$ ACI 318-19 Eq. (17.6.2.3.1)

$$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$$ ACI 318-19 Eq. (17.6.2.4.1b)

$$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{5/3}$ ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	14.000	0.313	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	∞	16	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
2,025.00	1,764.00	0.985	1.000	1.000	1.000	82,293

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
186,163	0.700	130,314	26,100

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.297	0.741	5/3	74	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta}$ <= 1

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/

Fastening meets the design criteria!

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7 Installation data

Profile: W shape (AISC), W10X15; (L x W x T x FT) = 9.990 in. x 4.000 in. x 0.230 in. x 0.270 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 0.500 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Hex Head ASTM F 1554 GR. 55 3/4

Item number: not available

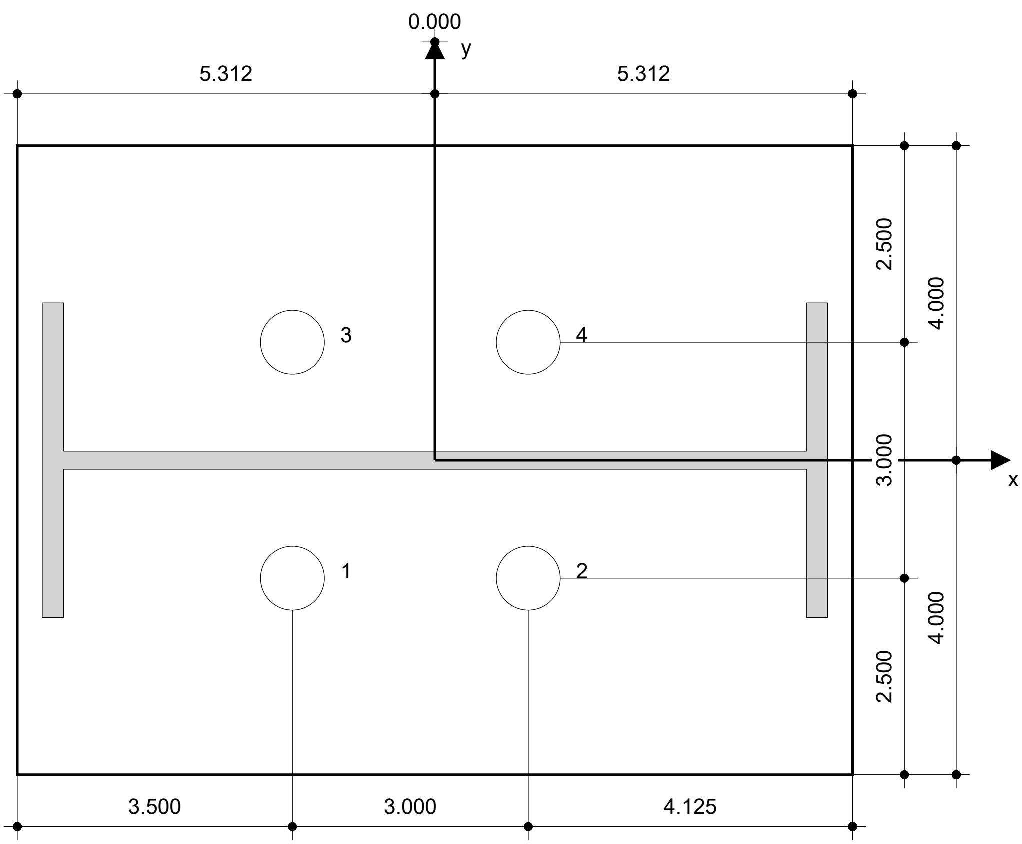
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 14.000 in.

Minimum thickness of the base material: 15.000 in.

Hilti Hex Head headed stud anchor with 14 in embedment, 3/4, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-1.813	-1.500	-	-	-	-
2	1.187	-1.500	-	-	-	-
3	-1.813	1.500	-	-	-	-
4	1.187	1.500	-	-	-	-

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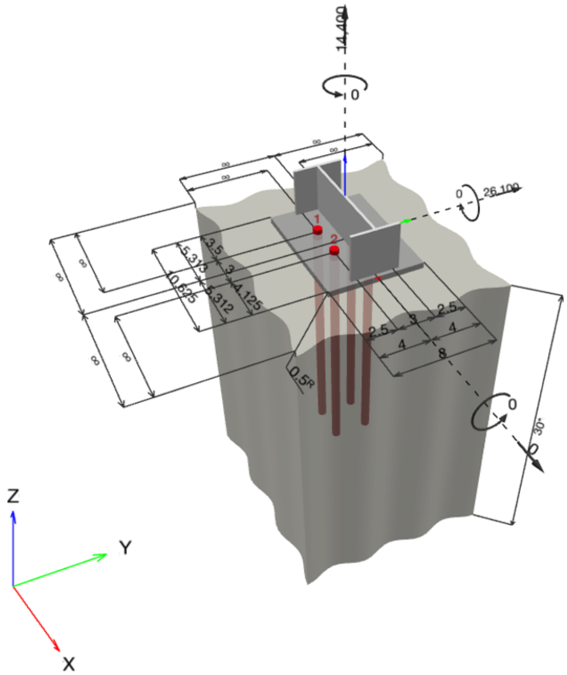
Specifier's comments:

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 55 3/4**
Item number:	not available
Effective embedment depth:	h_{ef} = 20.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 10.625 in. x 8.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	W shape (AISC), W10X15; (L x W x T x FT) = 9.990 in. x 4.000 in. x 0.230 in. x 0.270 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 30.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



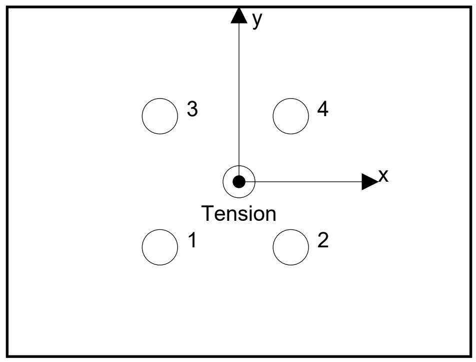
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 14{,}400; V_x = 0; V_y = 26{,}100;$ $M_x = 0; M_y = 0; M_z = 0;$	no	75

2 Load case/Resulting anchor forces

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	2,849	5,884	681	5,844
2	4,351	7,238	681	7,206
3	2,849	5,884	-681	5,844
4	4,351	7,238	-681	7,206



max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 14,400 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	4,351	18,787	24	OK
Pullout Strength*	4,351	14,650	30	OK
Concrete Breakout Failure**	14,400	113,894	13	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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3.1 Steel Strength

$N_{sa} = A_{se,N} \, f_{uta}$ ACI 318-19 Eq. (17.6.1.2)

$\phi \, N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

N_{sa} [lb]
25,050

Results

N_{sa} [lb]	ϕ_{steel}	$\phi \, N_{sa}$ [lb]	N_{ua} [lb]
25,050	0.750	18,787	4,351

3.2 Pullout Strength

$N_{pN} = \psi_{c,p} \, N_p$ ACI 318-19 Eq. (17.6.3.1)

$N_p = 8 \, A_{brg} \, f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)

$\phi \, N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.65	1.000	4,000

Calculations

N_p [lb]
20,928

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi \, N_{pn}$ [lb]	N_{ua} [lb]
20,928	0.700	14,650	4,351

3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$	ACI 318-19 Eq. (17.6.2.1b)
$\phi N_{cbg} \geq N_{ua}$	ACI 318-19 Table 17.5.2
A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)	
$A_{Nc0} = 9 h_{ef}^2$	ACI 318-19 Eq. (17.6.2.1.4)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.3.1)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.4.1b)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.6.1b)
$N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3}$	ACI 318-19 Eq. (17.6.2.2.3)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
20.000	0.313	0.000	∞	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
3,969.00	3,600.00	0.990	1.000	1.000	1.000	149,119

Results

N_{cbg} [lb]	$\phi_{concrete}$	ϕN_{cbg} [lb]	N_{ua} [lb]
162,706	0.700	113,894	14,400

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	7,238	9,769	75	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	26,100	227,789	12	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.33	75,000

Calculations

V_{sa} [lb]
15,030

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
15,030	0.650	9,769	7,238

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{5/3}$ ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	20.000	0.313	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	∞	16	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
3,969.00	3,600.00	0.990	1.000	1.000	1.000	149,119

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
325,412	0.700	227,789	26,100

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.297	0.741	5/3	74	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Nov 18, 2022	Date:	11/18/2022
Fastening point:			

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/

Fastening meets the design criteria!

7 Installation data

Profile: W shape (AISC), W10X15; (L x W x T x FT) = 9.990 in. x 4.000 in. x 0.230 in. x 0.270 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 0.500 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Hex Head ASTM F 1554 GR. 55 3/4

Item number: not available

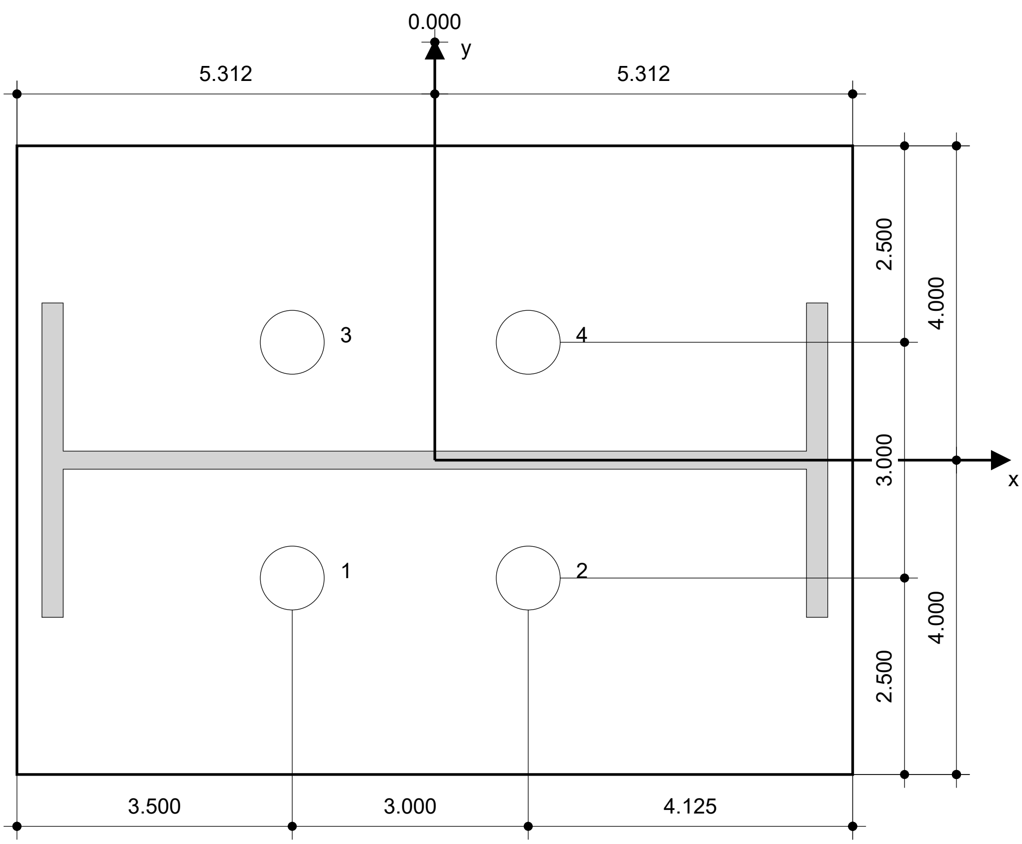
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 20.000 in.

Minimum thickness of the base material: 21.000 in.

Hilti Hex Head headed stud anchor with 20 in embedment, 3/4, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-1.813	-1.500	-	-	-	-
2	1.187	-1.500	-	-	-	-
3	-1.813	1.500	-	-	-	-
4	1.187	1.500	-	-	-	-

Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Nov 18, 2022	Date:	11/18/2022
Fastening point:			

8 Remarks; Your Cooperation Duties

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Appendix to

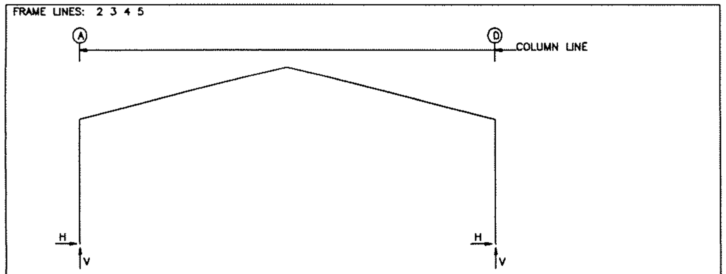
STRUCTURAL DESIGN CALCULATIONS
For

M. Aden Monheiser and Jennifer A. Monheiser

Lee's Summit, Missouri
Jackson County

6717 Shawnee Mission Parkway | Suite 100 | Overland Park, KS 66202 | 913-831-1262 | Fax 913-831-0148 pmaengineering.com

FRAME LINES: 2 3 4 5



RIGID FRAME: ANCHOR BOLTS & BASE PLATES

Frm Line	Col Line	Anc._Bolt Qty	Anc._Bolt Dia	Base_Plate (in) Width	Base_Plate (in) Length	Base_Plate (in) Thick	Elev. (in)
2*	A	4	0.750	8.000	10.63	0.375	0.0
2*	D	4	0.750	8.000	10.63	0.375	0.0

2* Frame lines: 2 3 4 5

ENDWALL COLUMN: ANCHOR BOLTS & BASE PLATES

Frm Line	Col Line	Anc._Bolt Qty	Anc._Bolt Dia	Base_Plate (in) Width	Base_Plate (in) Length	Base_Plate (in) Thick	Elev. (in)
1	A	4	0.750	8.000	8.250	0.375	0.0
1	B	4	0.750	8.000	8.250	0.375	0.0
1	C	4	0.750	8.000	8.250	0.375	0.0
1	D	4	0.750	8.000	8.250	0.375	0.0
6	D	4	0.750	8.000	8.250	0.375	0.0
6	C	4	0.750	8.000	8.250	0.375	0.0
6	B	4	0.750	8.000	8.250	0.375	0.0
6	A	4	0.750	8.000	8.250	0.375	0.0

GENERAL NOTES

1. ALL LOADING CONDITIONS ARE EXAMINED. THE MAXIMUM AND MINIMUM HORIZONTAL (H) AND VERTICAL (V) REACTIONS AND THE CORRESPONDING VERTICAL (V) OR HORIZONTAL (H) REACTIONS ARE REPORTED.
2. REACTIONS ARE PROVIDED BY LOAD CASE IN ORDER TO AID THE FOUNDATION ENGINEER IN DETERMINING THE APPROPRIATE LOAD FACTORS AND COMBINATIONS TO BE USED WITH EITHER WORKING STRESS OR ULTIMATE STRENGTH DESIGN METHODS. WIND LOAD CASES ARE GIVEN FOR EACH PRIMARY WIND DIRECTION.
3. FOR ASCE7-10 AND LATER BASED BUILDING CODES, THE UNFACTORED LOAD CASE REACTIONS DUE TO WIND ARE GENERATED USING THE ULTIMATE DESIGN WIND SPEED (Vult).
4. POSITIVE (+) REACTIONS ARE AS SHOWN ABOVE. FOUNDATION LOADS ARE IN OPPOSITE DIRECTIONS.
5. BRACING REACTIONS ARE IN THE PLANE OF THE BRACE WITH THE HORIZONTAL REACTION (H) ACTING AWAY FROM THE BRACED BAY AND THE VERTICAL REACTION (V) ACTING DOWNWARD.

****** RIGID FRAME LOAD CASE ABBREVIATIONS: ******
Wind_L1/Wind_R1: LATERAL WIND FROM THE LEFT/RIGHT, CASE 1
Wind_L2/Wind_R2: LATERAL WIND FROM THE LEFT/RIGHT, CASE 2
Wind_Ln1/Wind_Ln2: LONGITUDINAL WIND, CASE 1/2
Seismic_L/Seismic_R: LATERAL SEISMIC LOAD FROM LEFT/RIGHT
LWIND#_L#E/LWIND#_R#E: LONGITUDINAL WIND EDGE ZONES
F#UNB_SL_L/F#UNB_SL_R: UNBALANCED ROOF SNOW WITH WIND FROM LEFT/RIGHT
F#PAT_LL #/F#PAT_SL #: PARTIAL LIVE/SNOW LOADING FOR CONTINUOUS BEAM SYSTEMS

****** ENDWALL COLUMN LOAD CASE ABBREVIATIONS: ******
Collat: COLLATERAL LOAD
Rafter Wind_L/Rafter Wind_R: LATERAL WIND FROM THE LEFT/RIGHT
Brace Wind_L/Brace Wind_R: LATERAL WIND FROM THE LEFT/RIGHT
Wind_P/Wind_S: LONGITUDINAL WIND PRESSURE/SUCTION ON COLUMNS
Wind_Ln: LONGITUDINAL WIND SUCTION ON ROOF
Seis_L/Seis_R: LATERAL SEISMIC LOAD FROM LEFT/RIGHT
E#UNB_SL_L/E#UNB_SL_R: UNBALANCED ROOF SNOW WITH WIND FROM LEFT/RIGHT
E#PAT_LL #/E#PAT_SL #: PARTIAL LIVE/SNOW LOADING FOR CONTINUOUS BEAM SYSTEMS

RIGID FRAME: BASIC COLUMN REACTIONS (k)

Frame Line	Column Line	Dead Horiz	Dead Vert	Collateral Horiz	Collateral Vert	Live Horiz	Live Vert	Snow Horiz	Snow Vert	Wind_Left1 Horiz	Wind_Left1 Vert	Wind_Right1 Horiz	Wind_Right1 Vert
2*	A	1.5	3.2	0.4	0.8	7.8	15.1	5.4	10.6	-11.8	-17.3	-0.4	-11.1
2*	D	-1.5	3.2	-0.4	0.8	-7.8	15.1	-5.4	10.6	0.4	-11.1	11.8	-17.3

Frame Line	Column Line	Wind_Left2 Horiz	Wind_Left2 Vert	Wind_Right2 Horiz	Wind_Right2 Vert	Wind_Long1 Horiz	Wind_Long1 Vert	Wind_Long2 Horiz	Wind_Long2 Vert	Seismic_Left Horiz	Seismic_Left Vert	Seismic_Right Horiz	Seismic_Right Vert
2*	A	-10.9	-10.4	0.6	-4.2	-1.8	-14.6	-3.3	-12.6	-0.4	-0.2	0.4	0.2
2*	D	-0.6	-4.2	10.9	-10.4	3.3	-12.6	1.8	-14.6	-0.4	0.2	0.4	-0.2

Frame Line	Column Line	MIN_SNOW Horiz	MIN_SNOW Vert	F1UNB_SL_L Horiz	F1UNB_SL_L Vert	F1UNB_SL_R Horiz	F1UNB_SL_R Vert
2*	A	7.8	15.1	4.7	10.4	4.7	6.2
2*	D	-7.8	15.1	-4.7	6.2	-4.7	10.4

2* Frame lines: 2 3 4 5

ENDWALL COLUMN: BASIC COLUMN REACTIONS (k)

Frm Line	Col Line	Dead Vert	Collat Vert	Live Vert	Snow Vert	Wind Left1 Vert	Wind Right1 Vert	Wind Left2 Vert	Wind Right2 Vert	Wind Press Horz	Wind Suct Horz	Wind Long1 Vert	Wind Long2 Vert
1	A	0.5	0.1	1.9	1.3	-3.2	-2.0	-1.8	-0.5	-2.2	2.5	-3.7	-2.7
1	B	1.4	0.3	5.7	4.0	-7.8	-5.5	-5.8	-3.4	-5.4	5.9	-7.4	-3.9
1	C	1.4	0.3	5.7	4.0	-4.5	-8.1	-2.4	-6.2	-5.4	5.9	-4.1	-6.9
1	D	0.5	0.1	1.9	1.3	-3.0	-2.9	-1.5	-1.4	-2.2	2.5	-2.6	-4.1

Frm Line	Col Line	Seis Left Vert	Seis Right Vert	MIN_SNOW Horz	MIN_SNOW Vert	E1UNB_SL_L Horz	E1UNB_SL_L Vert	E1UNB_SL_R Horz	E1UNB_SL_R Vert
1	A	0.0	0.1	0.0	1.9	0.0	1.2	0.0	0.1
1	B	0.0	-0.1	0.0	5.7	0.0	5.1	0.0	2.0
1	C	0.0	0.0	0.0	5.7	0.0	2.0	0.0	5.1
1	D	0.0	0.0	0.0	1.9	0.0	0.1	0.0	1.2

Frm Line	Col Line	Dead Vert	Collat Vert	Live Vert	Snow Vert	Wind Left1 Vert	Wind Right1 Vert	Wind Left2 Vert	Wind Right2 Vert	Wind Press Horz	Wind Suct Horz	Wind Long1 Vert	Wind Long2 Vert
6	D	0.6	0.1	2.5	1.8	-2.9	-2.2	-1.6	-1.1	-2.2	2.5	-3.2	-2.4
6	C	1.2	0.9	5.1	3.6	-8.1	-5.3	-6.0	-2.8	-5.4	5.9	-7.9	-4.2
6	B	1.2	0.9	5.1	3.6	-5.3	-8.1	-2.8	-6.0	-5.4	5.9	-4.2	-7.9
6	A	0.6	0.1	2.5	1.8	-2.2	-2.9	-1.1	-1.6	-2.2	2.5	-2.4	-3.2

Frm Line	Col Line	Seis Left Vert	Seis Right Vert	MIN_SNOW Horz	MIN_SNOW Vert	E2UNB_SL_L Horz	E2UNB_SL_L Vert	E2UNB_SL_R Horz	E2UNB_SL_R Vert
6	D	0.0	0.0	0.0	2.5	0.0	2.0	0.0	1.0
6	C	0.0	0.0	0.0	5.1	0.0	4.3	0.0	1.0
6	B	0.0	0.0	0.0	5.1	0.0	1.1	0.0	4.3
6	A	0.0	0.0	0.0	2.5	0.0	1.0	0.0	2.0

BUILDING BRACING REACTIONS

Wall Loc	Wall Line	Col Line	± Reactions(k) Wind Horz	Wind Vert	Seismic Horz	Seismic Vert	Panel_Shear (lb/ft) Wind	Panel_Shear Seis	Note
L_EW	1	A,B	2.6	3.0	0.3	0.3			
F_SW	D	3,4	6.5	4.3	1.1	0.8			
R_EW	6								(i)
B_SW	A	4,3	6.5	4.3	1.1	0.8			

(i)Bracing in roof to rigid frame

ISSUE	DWN	CHK	ENG	PE	DATE
ANCHOR BOLTS	MBS	ARK	ARHR	TY	10/12/2022
PERMITS	MBS	ARK	ARHR	TY	10/12/2022
FINALS	CNB	WLR	ARHR	TY	10/20/2022

TOPLINE STEEL BUILDINGS
13323 SW BUTLER ROAD
ROSE HILL, KS 67133
PHONE
(800) 359-3882

PROJECT NAME
METRO DUMPSTER
2620 SE RANSON RD, LEE'S SUMMIT, MO 64082
CUSTOMER NAME
METRO DUMPSTER
LEE'S SUMMIT, MO 64082
JOB NUMBER
T22O8017A
SHEET TITLE

SHEET
R1