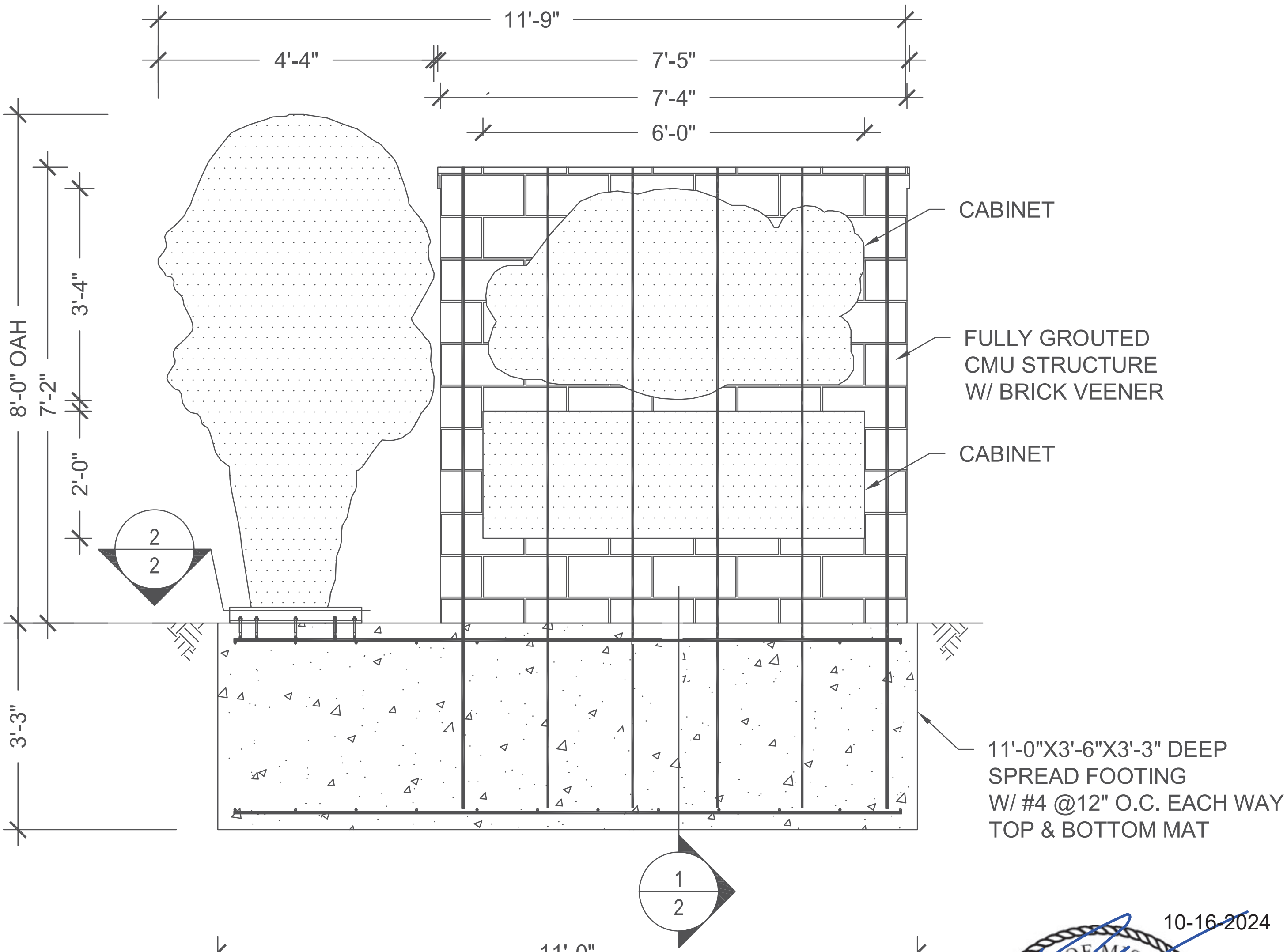
SULLAWAY ENGINEERING

12396 WORLD TRADE DRIVE, SUITE 312
SAN DIEGO, CA 92128
PROJECTMANAGER@SULLAWAYENG.COM
PHONE: 1-858-312-5150 FAX: 1-858-777-3534

PROJECT: ANDY'S FROZEN CUSTARD, 630 NW. CHIPMAN RD., LEE'S SUMMIT, MO
PROJECT #: 48162
CLIENT: PINNACLE SIGN GROUP

DATE: 10/14/2024
ENGINEER: RM
LAST REVISED:



1 - ELEVATION

10-16-2024

STATE OF MISSOURI
MICHAEL F. SULLAWAY
NUMBER PE-2010022411
PROFESSIONAL ENGINEER

GENERAL NOTES

1. DESIGN CODE: IBC 2018
2. DESIGN LOADS: ASCE 7-16
3. WIND VELOCITY 110 MPH EXPOSURE C
4. CONCRETE 2500 PSI MINIMUM
5. PLATE STEEL ASTM A36
6. HILTI KB-TZ2 SS-304 ANCHOR BOLTS PER ICC ESR-4266
7. THE CMU BLOCK SHALL CONFIRM TO ASTM C90, WITH STRENGTH f'm=1500 PSI MIN.
8. MORTAR PER ASTM C270, TYPE S WITH A MINIMUM STRENGTH OF 1800 PSI MIN.
9. GROUT SHALL BE IN CONFORMANCE WITH ASTM C476. COMPRESSIVE STRENGTH AS PER ASTM C1019
10. CELLS SHALL BE FILLED SOLID WITH GROUT
11. VERTICAL BARS ARE TO BE TIED OR FIXED IN POSITION BY OTHER MEANS AND IN INTERVALS NOT LESS THAN 200 BAR DIAMETERS.
12. STEEL REINFORCEMENT IN CONCRETE ASTM A615 GR 60
13. PROVIDE MIN. 3" CLEAR COVER ON ALL STEEL EMBEDDED IN CONCRETE WHEN CAST AGAINST SOIL
14. VERTICAL SOIL BEARING PER IBC CLASS 4 (2000 PSF)
15. PROVIDE PROTECTION AGAINST DISSIMILAR METALS
16. ALL DIMENSIONS TO BE VERIFIED PRIOR TO FABRICATION

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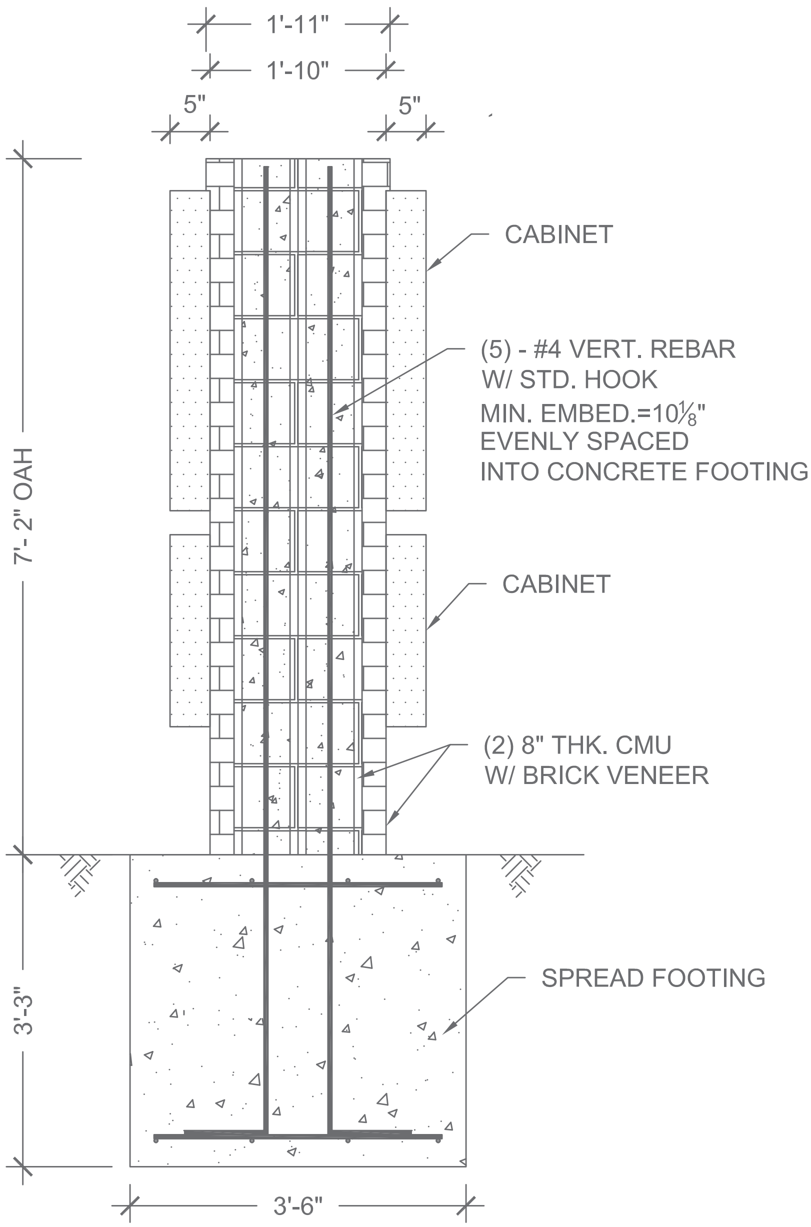

1 - SIDE VIEW

STATE OF MISSOURI
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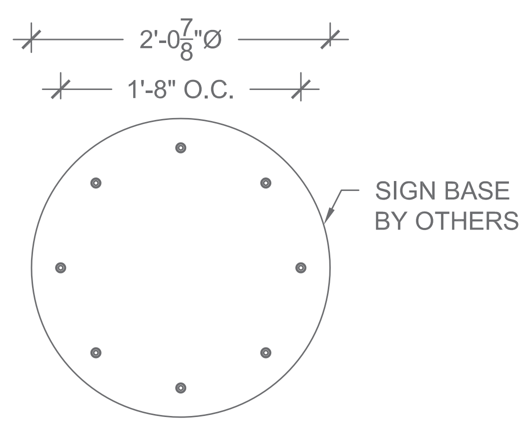

(8) ⅜"Ø HILTI KB-TZ2 - SS 304
MIN. EMBED.=2½" INTO CONCRETE

2 - ANCHOR DETAIL

SULLAWAY ENGINEERING

PROJECT: ANDY'S FROZEN CUSTARD — DATE: 10/14/2024
PROJ. NO.: 48162 — ENGINEER: RM
CLIENT: PINNACLE SIGN GROUP

V5.5 — units; pounds, feet unless noted otherwise

Applied Wind Loads; from ASCE 7-16 LRFD: (For Spread footing)

$F=q_z*G*C_f*A_f$ with $q_z = 0.00256K_zK_{zt}K_dV^2$ (29.3.2 & 29.4)

C_f= 1.467 (Fig. 29.3-1) 1.00 0 max. height= 8.00

K_{zt}= 1.0 (26.8.2) (=1.0 unless unusual landscape)

K_z= from table 28.3-1 Exposure= c

K_d= 0.85 for signs (table 26.6-1)

V= 110 mph

G= 0.85 (26.9)

s/h= 1.000 M_{DL}= 0.00 k-ft

B/s= 0.92

Pole Loads	structure component	height at section c.g.	K_z	q_z	pressure q_z*G*C_f	A_f	shear	Wind Moment M_W				
	1	0.13	0.850	22.4	27.90	2.10	59	7				
	2	3.71	0.850	22.4	27.90	71.51	1995	7399				
	3	7.58	0.850	22.4	27.90	1.81	50	382				
					sums:	75.42	2104	7.79	(M_w)	k-ft	arm=	3.7
	for s/h=1, add 10% (asce fig. 29.4-1):				x 1.10			8.57				
		P_u=	0.91	kip			M=	8.57	k-ft	$M=\sqrt{M_{DL}^2+M_w^2}$		
	$M_u=\sqrt{1.2M_{DL}^2+1.0M_W^2}$ =		8.57	k-ft								

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V5.5

Longitude Direction

units; pounds, feet unless noted otherwise

applied shear at grade	v=	**1.315**	kip	unfactored load	2.104	k (factored)
applied moment at grade	m=	**5.355**	kip-ft	unfactored load	8.57	k-ft
depth of soil above footing	h_s=	**0.0**	ft			
allowable soil bearing	p=	**2.000**	ksf			
Signage Weight=	w1	**0.553**	**k**	(55.33 ft^2)*10 psf	(Area From AutoCAD)	
CMU W/ Brick veneer=	w2	**12.08**	**k**	((7'-4")*(6'-10")*(1'-10")+(7'-5")*(4")*(1'-11"))*0.125k/ft^3		

Spread Footing Design

moment m= 9.6 k-ft

Footing size (ft) b= **11.00** L= **3.50** h= **3.25** S= 22.5

Footing Weight= 18.8 k See Above ,w1+w2= **12.63** k soil 0.00 total= 31.40

Overturning; M_c= 55 M_c>1.5M 1.750 ok

soil pressure; max= 1.244 ksf ok

forces on concrete pad; V= 20 k V_r= 32 k (=1.6V)

M= 17 k-ft M_r= 27.8 k-ft

Check Slab;

ϕ= 0.9 f_y= 60 ksi f_c= 2.5 ksi 150 lbs/ft3

Flexure A_s= **0.25** d= 35.0 in

$\phi M_n=\phi A_s f_y(d-a/2)$= 472 k-in = 39 k-ft $M_r<\phi M_n$ ok

$a=A_s f_y/0.85 f_c b$= 0.053 in

Check minimum A_{smin}=3sqrt(f_c)bd/f_y = 11.55 200bd/fy= 15.40 or 1.333A_s= 0.33 in^2

ACI 10.3.1

Use A_s= 0.33 in^2

short direction $\gamma_s=2/(\beta+1)$ = 0.8 with β= 1.5 short direction; γA_s= 0.27 in^2

Use #4@12" each direction

Shear; $\phi V_n=\phi 2$sqrt(f_c)bd ϕV_c= 346.5 ϕ= 0.75 $V_r<\phi V_n$ ok

Top Slab

Assume half of footing is in uplift

weight= 9 kip arm= 0.9 ft

M= 8 k-ft M_r= 9.9 k-ft

Flexure A_s= **0.150**

$\phi M_n=\phi A_s f_y(d-a/2)$= 283 k-in = 23.6 k-ft $M_r<\phi M_n$ ok

$a=A_s f_y/0.85 f_c b$= 0.101 in

Check minimum A_{smin}=2sqrt(f_c)bd/f_y = 3.675 200bd/fy= 4.90 or 1.333A_s= 0.2 in^2

Use A_s= 0.20 in^2

short direction $\gamma_s=2/(\beta+1)$ = 0.8 with β= 1.5 short direction; γA_s= 0.16 in^2

Use #4@12" each direction

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Applied Wind Loads; from ASCE 7-16 LRFD: (For Cone)

$F=q_z*G*C_f*A_f$ with $q_z = 0.00256K_zK_{zt}K_dV^2$ (29.3.2 & 29.4)

C_f= 1.542 (Fig. 29.3-1) 1.00 0 max. height= 8.00

K_{zt}= 1.0 (26.8.2) (=1.0 unless unusual landscape)

K_z= from table 28.3-1 Exposure= c

K_d= 0.85 for signs (table 26.6-1)

V= 110 mph

G= 0.85 (26.9)

s/h= 1.000 M_{DL}= 0.00 k-ft

B/s= 0.54

Pole Loads	structure component	height at section c.g.	K_z	q_z	pressure q_z*G*C_f	A_f	shear	Wind Moment M_W			
	1	0.13	0.850	22.4	29.33	0.52	15	2			
	2	4.13	0.850	22.4	29.33	22.57	662	2730			
					sums:	23.09	677	2.73	(M_w)	k-ft	arm= 4.0
	for s/h=1, add 10% (asce fig. 29.4-1):				x 1.10			3.01			
		P_u=	0.28	kip			M=	3.01	k-ft	$M=\sqrt{M_{DL}^2+M_w^2}$	
	$M_u=\sqrt{1.2M_{DL}^2+1.0M_W^2}$ =		3.01	k-ft							

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PROJ. NO.: 48162 | ENGINEER: RM
CLIENT: PINNACLE SIGN GROUP

V5.5 | units; pounds, feet unless noted otherwise

Applied Wind Loads; from ASCE 7-16 (LRFD): (CMU Wall)

$F=q_z*G*C_f*A_f$ with $q_z = 0.00256K_zK_{zt}K_dV^2$ (29.3.2 & 29.4)

C_f= 1.449 (Fig. 29.3-1) 1.00 0 max. height= 7.17

K_{zt}= 1.0 (26.8.2) (=1.0 unless unusual landscape)

K_z= from table 28.3-1 Exposure= c

K_d= 0.85 for signs (table 26.6-1)

V= 110 mph

G= 0.85 (26.9)

s/h= 1.000 M_{DL}= 0.00 k-ft

B/s= 1.02

Pole Loads	structure component	height at section c.g.	K_z	q_z	pressure q_z*G*C_f	A_f	shear	Wind Moment M_W			
	1	3.42	0.850	22.4	27.56	50.11	1381	4719			
	2	7.00	0.850	22.4	27.56	2.47	68	477			
					sums:	52.58	1449	5.20	(M_w)	k-ft	arm= 3.6
	for s/h=1, add 10% (asce fig. 29.4-1):				x 1.10			5.72			
		P_u=	0.63	kip			M=	5.72	k-ft	$M=sqrt(M_{DL}^2+M_w^2)$	
	$M_u=sqrt(1.2M_{DL}^2+1.0M_W^2)$ =		5.72	k-ft							

H	M_u(k-ft)
at grade	5.72

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PROJECT: ANDY'S FROZEN CUSTARD DATE: 10/14/24
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V5.5 units; pounds, feet unless noted otherwise

Check CMU wall - cantilever flexure (fc = 1500psi) (6) #4 bar Evenly Spaced ϕ= 0.90

Moment at grade=		Mu= 5.72 k-ft	(See page#6)			
fy =	60 ksi	d = 12 in	(16"-4")		b =	88 in
bar #=	4	A = 0.20 in^2			n*As=6*As =	1.178 in^2
			No of rod=6 nos			
a=		$a=A_sf_y/0.85f_cb=$	0.630 in			
ϕMn=		$\phi A_sf_y(d-a/2)=$	61.95 k-ft per 88 in			
Ratio Check=		Mu/ϕMn=	0.09 <1	**ok**		

Std. Hook Development ACI Ch. 25.4

bar #=	4
bar dia.=	0.5
fy=	60000 psi
f'c=	2500 psi
λ=	1
ψe=	1.2
ψc=	0.7
ψr=	1

Greater of:

a) (fyψeψcψr/50λ√f'c)db	=	10.08 in
b) 8db	=	4.00 in
c)	=	6.00 in
	use	**10.08** in

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:		Date:	
Fastening point:			

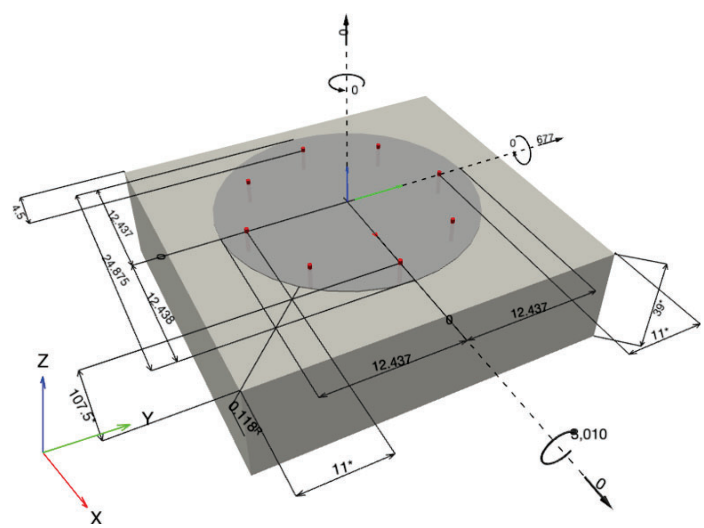
Specifier's comments:

1 Input data

Anchor type and diameter:	**Kwik Bolt TZ2 - SS 304 3/8 (2) hnom2**
Item number:	2210245 KB-TZ2 3/8x5 SS304
Specification text:	Hilti KB-TZ2 stud anchor with 2.5 in embedment, 3/8 (2) hnom2, Stainless steel, installation per ESR-4266
Effective embedment depth:	$h_{ef,act}$ = 2.000 in., h_{nom} = 2.500 in.
Material:	AISI 304
Evaluation Service Report:	ESR-4266
Issued I Valid:	12/1/2023 | 12/1/2025
Proof:	Design Method ACI 318-14 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.118 in.
Anchor plate^R :	l_x x l_y x t = 24.875 in. x 24.875 in. x 0.118 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 39.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

^{R} - The anchor calculation is based on a rigid anchor plate assumption.

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



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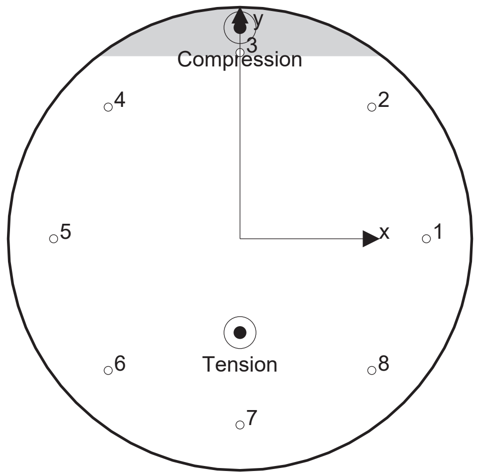
Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	48162-hilti	Date:	
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [ft.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 0; V_y = 677; M_x = -3,010.000; M_y = 0.000; M_z = 0.000;	no	36

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	275	85	0	85
2	78	85	0	85
3	0	85	0	85
4	78	85	0	85
5	275	85	0	85
6	472	85	0	85
7	554	85	0	85
8	472	85	0	85

Max. concrete compressive strain:	0.05 [‰]
Max. concrete compressive stress:	209 [psi]
Resulting tension force in (x/y)=(-0.000/-5.038):	2,205 [lb]
Resulting compression force in (x/y)=(-0.000/11.343):	2,205 [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*	554	4,637	12	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	2,205	6,134	36	OK

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

Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	48162-hilti	Date:	
Fastening point:			

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-4266

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.05	120,104

Calculations

N_{sa} [lb]
6,182

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
6,182	0.750	4,637	554

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	48162-hilti	Date:	
Fastening point:			

3.2 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 318-14 Eq. (17.4.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1+\frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-14 Eq. (17.4.2.2a)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
2.000	0.000	3.609	4.500	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
5.500	21	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
252.00	36.00	1.000	0.454	1.000	1.000	2,970

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
9,436	0.650	6,134	2,205

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	48162-hilti	Date:	
Fastening point:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi\ V_n$	Status
Steel Strength*	85	3,177	3	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	677	16,631	5	OK
Concrete edge failure in direction x-**	677	4,003	17	OK

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

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-4266
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.05	120,104

Calculations

V_{sa} [lb]
4,887

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
4,887	0.650	3,177	85

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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	48162-hilti	Date:	
Fastening point:			

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 318-14 Eq. (17.5.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
1	2.000	0.000	0.000	4.500

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	5.500	21	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
288.00	36.00	1.000	1.000	1.000	1.000	2,970

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
23,759	0.700	16,631	677

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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	48162-hilti	Date:	
Fastening point:			

4.3 Concrete edge failure in direction x-

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ — ACI 318-14 Eq. (17.5.2.1a)

$\phi\ V_{cb} \geq V_{ua}$ — ACI 318-14 Table 17.3.1.1

A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ — ACI 318-14 Eq. (17.5.2.1c)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.5.2.6b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-14 Eq. (17.5.2.8)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c}\ c_{a1}^{1.5}$ — ACI 318-14 Eq. (17.5.2.2a)

Variables

c_{a1} [in.]	c_{a2} [in.]	$\psi_{c,V}$	h_a [in.]	l_e [in.]
4.500	21.000	1.000	39.000	2.000

λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
1.000	0.375	2,500	2.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
91.13	91.13	1.000	1.000	2,860

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi\ V_{cb}$ [lb]	V_{ua} [lb]
5,719	0.700	4,003	677

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.359	0.169	5/3	24	OK

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

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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	48162-hilti	Date:	
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.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-14, Section 17.8.1.

Fastening meets the design criteria!

Hilti PROFIS Engineering 3.1.4

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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	48162-hilti	Date:	
Fastening point:			

7 Installation data

Anchor type and diameter: Kwik Bolt TZ2 - SS 304 3/8 (2) hnom2

Item number: 2210245 KB-TZ2 3/8x5 SS304

Maximum installation torque: 30.093 ft.lb

Hole diameter in the base material: 0.375 in.

Hole depth in the base material: 2.750 in.

Minimum thickness of the base material: 4.000 in.

Profile: no profile

Hole diameter in the fixture: d_f = 0.438 in.

Plate thickness (input): 0.118 in.

Recommended plate thickness: not calculated

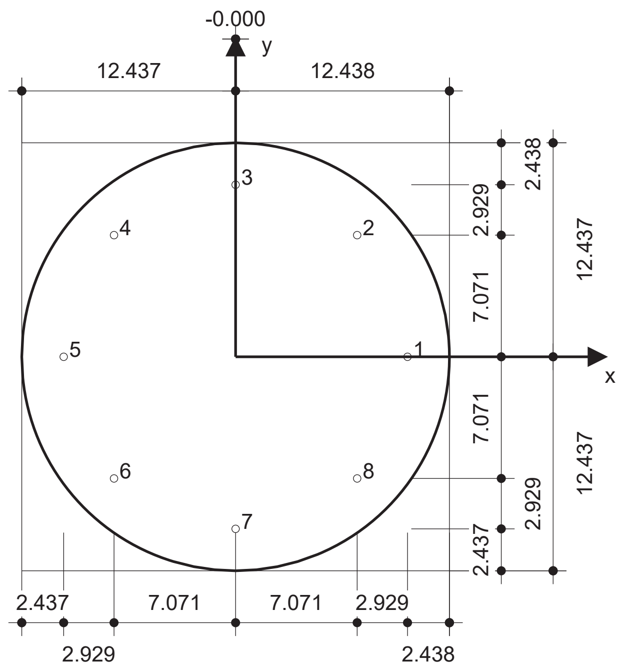
Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Hilti KB-TZ2 stud anchor with 2.5 in embedment, 3/8 (2) hnom2, Stainless steel, installation per ESR-4266

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque controlled cordless impact tool • Torque wrench • Hammer



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	10.000	0.000	24.500	107.500	21.000	21.000
2	7.071	7.071	21.571	110.429	28.071	13.929
3	0.000	10.000	14.500	117.500	31.000	11.000
4	-7.071	7.071	7.429	124.571	28.071	13.929

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
5	-10.000	0.000	4.500	127.500	21.000	21.000
6	-7.071	-7.071	7.429	124.571	13.929	28.071
7	0.000	-10.000	14.500	117.500	11.000	31.000
8	7.071	-7.071	21.571	110.429	13.929	28.071

Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	48162-hilti	Date:	
Fastening point:			

8 Remarks; Your Cooperation Duties

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