MOUNTAIN VIEW ENGINEERING, INC.
Structural Engineering • Consulting • Design
345 North Main Street, Suite A • Brigham City, Utah 84302 • Phone (435) 734-9700 • Fax (435) 734-9519 • mvengr.net

REVIEWED FOR CODE COMPLIANCE
09/18/2023

September 15, 2023

RE: Clay Hockel – C&B Holdings
Hayden, Colorado

To Whom It May Concern:

Our firm prepared the foundation design and details for the referenced project. The original foundation calculations dated July 31, 2023 included calculations for the anchor rods and with its anchor rod reinforcement. Please see pages 9 & 10 of the original foundation calculations, which we have attached, for anchor rod calculations for the worst case 1" diameter bolts (Sidewall) and 5/8" diameter bolts (Endwall and Corners). The anchor rod calculations are per ACI 318-14.

Please contact me if you have any questions.

Respectfully,

COLORADO LICENSED
JEDEDIAH VERNE LARSEN
57894
PROFESSIONAL ENGINEER

SEP 15 2023

Jed Larsen, P.E.
Mountain View Engineering, Inc.

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FOUNDATION CALCULATIONS

65'x70' Metal Building

MVE #23-0706

CLAY HOCKEL C&B HOLDINGS
Hayden, Colorado

Metal Building Supplied By:

METAL BUILDING OUTLET CORP.
7651 Shaffer Pkwy, Ste. A
Littleton, CO 80127

COLORADO LICENSED JEDEDIAH VERNE LARSEN 57894 PROFESSIONAL ENGINEER

JUL 31 2023

Foundation Design by:

MOUNTAIN VIEW ENGINEERING, INC.
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12 pages of Calculations

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DESIGN CRITERIA :

Ground Snow Load	80	psf
Roof Snow Load	60	psf
Roof Live Load	20	psf
Roof Collateral Load	1	psf

Code: 2018 IBC

S_{DS}	0.409
Seismic Design Category	C
Site Class	D

Wind Speed	115	mph
Exposure	C	
Importance Factor	1.0	

Other Loads:

Soil Bearing	3500	psf (Per Geotech)
Minimum Bearing	1500	psf (Per Geotech)
Frost Depth	48	inches (Per Geotech)
Passive Pressure	250	psf/ft (Assumed)
Coefficient of Friction	0.25	(Assumed)

Notes:

Reactions per METAL BUILDING OUTLET CORP. drawings.

The foundation has been designed, and the site soils should be prepared for the foundation, in accordance with the recommendations in the geotechnical investigation report by:

Erika K. Hill, P.E., P.G. of NWCC, Inc.
2580 Copper Ridge Drive, Steamboat Springs, CO 80487
Phone: (970) 879-7888   Job Number: 22-12817

Concrete and Reinforcement:

Concrete Strength
- 3000 P.S.I. for Foundations
- 3500 P.S.I. for Slabs
- 2500 P.S.I. Used for design, no special inspection required.

Rebar - ASTM A615 grade 60

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Sidewall Footings
(Lines 2 & 3 / Grids A & E)

P_{D+L} = 53.7 kips
F_H = 30.3 kips
Uplift = 9.3 kips

Use 5.0 ft. x 5.0 ft. x 30 inch deep footing

Horizontal Force *Use rebar tension ties across the building to resist horizontal force at the column base.*

Top of Pier to Center of Ties =	18	in	Number of Ties =	4	ties
Tensile Strength of Rebar =	24	ksi	Tie Size =	#6	rebar
Area Required =	1.263	in^2			

Use (4) #6 tension ties.

Weights

Weight of Pier =	1.36	kips
Weight of Soil Above Footing =	3.58	kips
Weight of Spot Footing =	9.06	kips
Weight of Continuous Wall =	0.00	kips
Weight of Continuous Ftg. =	0.00	kips
Use Passive Res. to Resist Moment?	YES	

Passive Soil Resistance

Wall Length for Passive Res. =	1.5	ft
Ftg. Width for Passive Res. =	5	ft
Passive Earth Pressure =	200	psf/ft
Passive Res. (Spot Footing) =	8.13	kips
Passive Res. (Wall & Pier) =	0.30	kips
Passive Res. (Cont. Ftg.) =	0.00	kips
Total Passive Resistance =	**8.43**	**kips**

Check Soil Bearing

Moment Arm =	1.5	ft
P (total) =	53.70	kips
Overturning Moment =	45.45	kip*ft
OTM Eccentricity =	10.2	inches
Footing Offset =	3.25	inches
Offset Resisting Moment =	- 14.54	kip*ft
Passive Resisting Moment =	- 7.72	kip*ft
Net Eccentricity =	5.2	inches
B/6 = 10 inches OK		

Allowable Bearing Pressure =	3500	psf
Top of Wall to Grade =	6	in
OS Conc. to CL A.R. =	16.25	in
Pier Width =	18	in
Pier Depth (wall included) =	30	in
Pier Height =	30	in
Wall Thickness =	8	in
Wall Height =	48	in
Footing Width =	16	in
Footing Depth =	8	in

Bearing Pressure, q (max.) = 3261 psf OK

Minimum Bearing Pressure = 1763 psf OK

Offset footing 3.25 inches.

Uplift

Weight of Footing and Pier =	10.42	kips
Weight of Soil Above Footing =	3.58	kips
Weight of Cont. Wall & Footing =	10.63	kips
Total =	24.64	kips

Wall Length used for Uplift =	23	ft
Cont. Ftg. Length for Uplift =	23.0	ft

Factor of Safety = 2.65 > 1.0 OK

Check Footing Flexure (Reinforcing in Direction of Horizontal Force)

q (min.) =	1035	psf	Rebar d' =	3.5	in
OS Footing Edge from Wall =	1.417	ft	Rebar d =	26.5	in
q (at face of wall) =	2630	psf	Rebar fy =	60000	psi
Moment in Footing (Mu, ULT) =	20.74	k*ft	Concrete f'c =	2500	psi
As (req'd by calc.) =	0.174	in^2	ACI 7.12 As (min) =	3.240	in^2

Opposite Direction Reinforcing

Min. Steel Ratio = 0.0018
As per ACI 7.12

Use (6) #5 bars each way at top of footing and use (6) #5 bars each way at bottom of footing.

Check Footing Shear

Shear in Footing (Vu, ULT) =	29.27	kips	
Required Thickness =	10.01	in	OK

For Pier Design
****See pier calculation on page 3.**

Nu =	**86**	**kips**
Mu =	**73**	**kip*ft**
Vu =	**48**	**kips**

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Concrete Column Analysis (ACI 318)

For X-Axis Flexure with Axial Compression or Tension Load
Assuming "Short", Non-Slender Member with Symmetric Reinforcing

Input

f_c' =	2500	psi
f_y =	60	ksi
d' =	2.375	in
b =	18	in
h =	30	in
ϕ =	0.65	

Loading

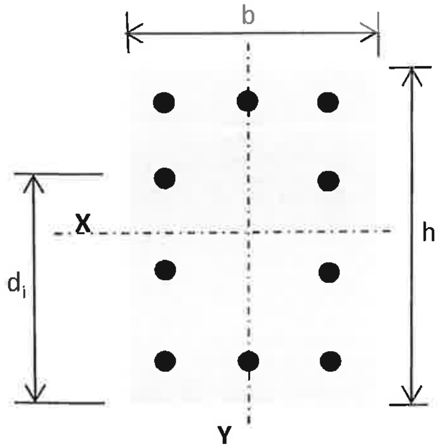
P_{ux} =	51.6	kips
M_{ux} =	13.2	kip-ft
V_{ux} =	3.3	kips

Column Geometry

Bar Size = 5   Total # of Bars 14
# of Bars b Face 4   Tie Size = 4
# of Bars h Face 5

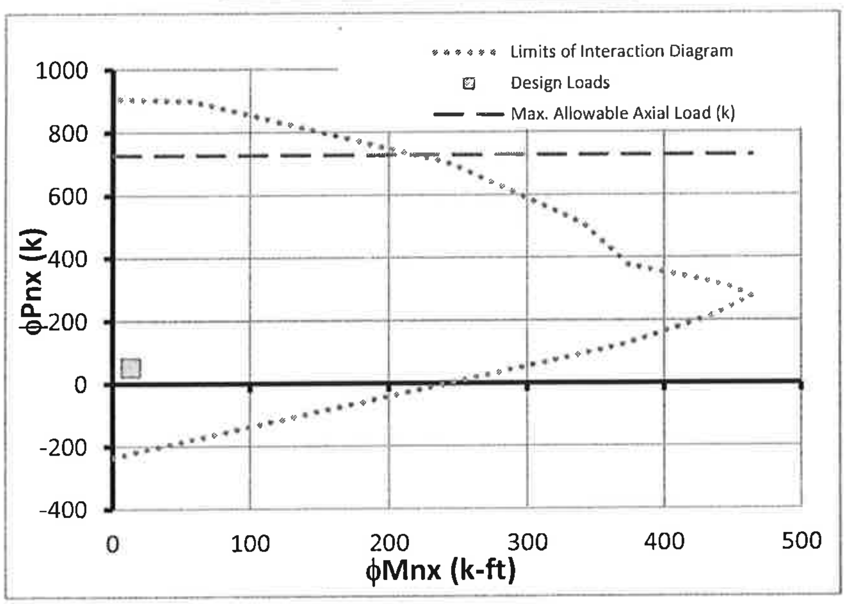
Placement of Reinforcement Steel

	d_i	A_{si}
Edge Layer (d_1)	27.63	1.24
Interior Layer (d_2)	21.31	0.62
Interior Layer (d_3)	15.00	0.62
Interior Layer (d_4)	8.69	0.62
Interior Layer (d_5)	0.00	0.00
Edge Layer (d_6)	2.38	1.24



Typical Member Section

X-AXIS INTERACTION DIAGRAM



DESIGN LOADS FALL WITHIN THE LIMITS OF THE INTERACTION DIAGRAM, THEREFORE, USE (14) # 5 VERTICAL BARS IN COLUMN.

Shear Design ϕV_c = 39.076 $\phi V_c/2$ = 19.538 $V_u < \phi V_c/2$

If $V_u < \phi V_c/2$ then Vertical Spacing of ties shall not exceed the least of:

16 x (longitudal bar diameters)	=	10	in
48 x (tie bar diameter)	=	24	in
Least dimension of column	=	18	in

If $V_u > \phi V_c/2$ then vertical spacing of ties shall not exceed the least of:

s max =	$A_v f_y/(0.75\sqrt{f_c'}\,b)$	=	35.556	in
s max =	$A_v f_y/(50b)$	=	26.667	in
s max =	d/2 ≤ 24 in	=	13.813	in

USE # 4 TIES AT 8.00 INCHES ON CENTER WITH (3) IN THE TOP SIX INCHES OF PIER.

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Endwall Footings

(Line 1 / Grids B, D, & E and Line 4 / Grids B, C, & E)

P_{D+L} = 25.3 kips
F_H = 2.9 kips
Uplift = 5.9 kips

Use 3.5 ft. x 3.5 ft. x 12 inch deep footing

Check Soil Bearing

Moment Arm =	5	ft	Allowable Bearing Pressure =	3500	psf
P (total) =	25.30	kips	Top of Wall to Grade =	6	in
Overturning Moment =	14.5	kip*ft	OS Conc. to CL A.R. =	4	in
OTM Eccentricity =	6.9	inches	Pier Width =	12	in
Footing Offset =	0	inches	Pier Depth (wall included) =	8	in
Offset Resisting Moment =	- 0.00	kip*ft	Pier Height =	48	in
Passive Resisting Moment =	- 4.34	kip*ft	Wall Thickness =	8	in
Net Eccentricity =	4.8	inches	Wall Height =	48	in
B/6 = 7 inches OK			Footing Width =	16	in
			Footing Depth =	8	in

Bearing Pressure, q (max.) = 3487 psf OK

Minimum Bearing Pressure = 1783 psf OK

Offset footing 0 inches.

Sliding Resistance

Coefficient of Friction =	0.25		Wall Length for Sliding =	4.0	ft
Weight of Pier =	0.39	kips	Wall Length for Passive Res. =	4.0	ft
Weight of Soil Above Footing =	3.24	kips	Ftg. Width for Sliding/Passive =	4.0	ft
Weight of Spot Footing =	1.78	kips	Passive Earth Pressure =	200	psf/ft
Weight of Continuous Wall =	1.16	kips	Passive Res. (Spot Footing) =	2.80	kips
Weight of Continuous Ftg. =	0.06	kips	Passive Res. (Wall & Pier) =	1.40	kips
Sliding Resistance from Footing & Pier =	0.54	kips	Passive Res. (Cont. Ftg.) =	0.26	kips
Sliding Resistance from Soil above Ftg. =	0.81	kips	**Total Passive Resistance =**	**4.46**	**kips**
Sliding Resistance from Vertical Load =	6.33	kips	**Total Sliding Resistance =**	**7.98**	**kips**
Sliding Resistance from Wall & Ftg. =	0.31	kips	**Factor of Safety =**	**4.29**	**> 1.0 OK**

Uplift

Weight of Footing and Pier =	2.16	kips	Wall Length used for Uplift =	14.5	ft
Weight of Soil Above Footing =	3.24	kips	Cont. Ftg. Length for Uplift =	14.5	ft
Weight of Cont. Wall & Footing =	6.64	kips			
Total =	12.04	kips	**Factor of Safety =**	**2.04**	**> 1.0 OK**

Check Footing Flexure (Reinforcing in Direction of Horizontal Force)

						Options	
q (min.) =	644	psf	Rebar d' =	3.5	in		
OS Footing Edge from Wall =	1.417	ft	Rebar d =	8.5	in	5	#4 bars
q (at face of wall) =	2336	psf	Rebar fy =	60000	psi	3	#5 bars
Moment in Footing (Mu, ULT) =	15.55	k*ft	Concrete f'c =	2500	psi	3	#6 bars
As (req'd by calc.) =	0.411	in^2	ACI 7.12 As (min) =	0.907	in^2		

Opposite Direction Reinforcing

Min. Steel Ratio = 0.0018
As per ACI 7.12

Options

5	#4 bars
3	#5 bars
3	#6 bars

Use (5) #4 bars in direction of horizontal force and use (5) #4 bars in the opposite direction.

Check Footing Shear

Shear in Footing (Vu, ULT) = 21.95 kips
Required Thickness = 10.47 in OK

For Pier Design

Nu =	**40**	**kips**
Mu =	**19**	**kip*ft**
Vu =	**5**	**kips**

****See pier calculation on page 5.**

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Concrete Column Analysis (ACI 318)

For X-Axis Flexure with Axial Compression or Tension Load
Assuming "Short", Non-Slender Member with Symmetric Reinforcing

Input

f_c' =	2500	psi
f_y =	60	ksi
d' =	2	in
b =	12	in
h =	8	in
ϕ =	0.65	

Loading

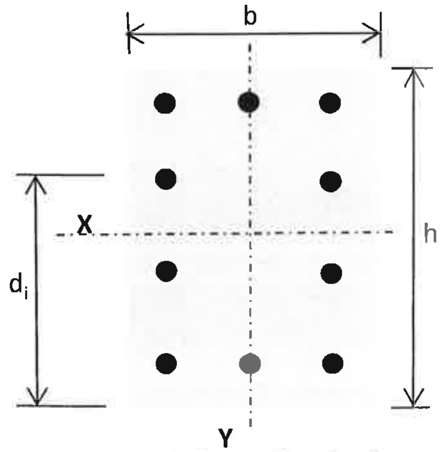
P_{ux} =	51.6	kips
M_{ux} =	13.2	kip-ft
V_{ux} =	3.3	kips

Column Geometry

Bar Size =	4	Total # of Bars	6
# of Bars b Face	3	Tie Size =	3
# of Bars h Face	2		

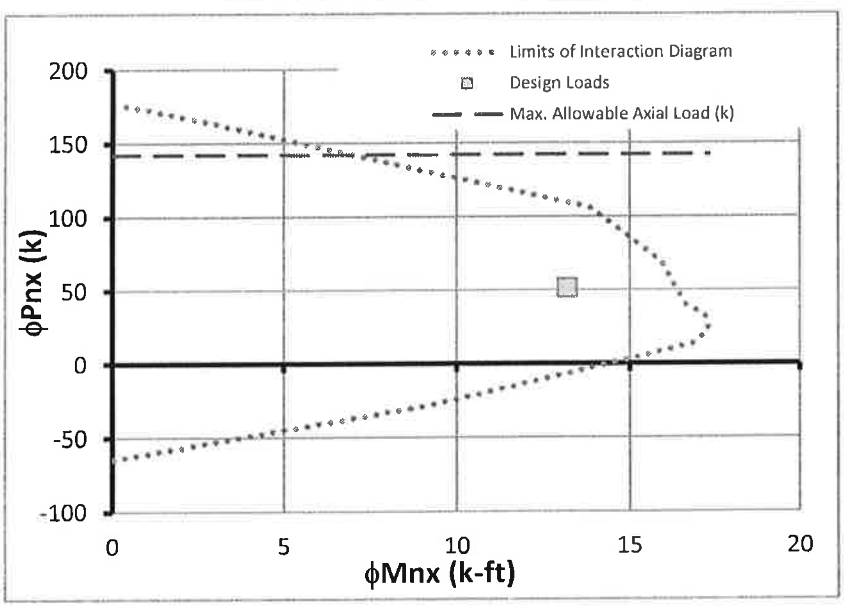
Placement of Reinforcement Steel

	d_i	A_{st}
Edge Layer (d_1)	6.00	0.60
Interior Layer (d_2)	0.00	0.00
Interior Layer (d_3)	0.00	0.00
Interior Layer (d_4)	0.00	0.00
Interior Layer (d_5)	0.00	0.00
Edge Layer (d_6)	2.00	0.60



Typical Member Section

X-AXIS INTERACTION DIAGRAM



DESIGN LOADS FALL WITHIN THE LIMITS OF THE INTERACTION DIAGRAM, THEREFORE, USE (6) # 4 VERTICAL BARS IN COLUMN.

Shear Design ϕV_c = 6.8513 $\phi V_c/2$ = 3.4256 $V_u < \phi V_c/2$

If $V_u < \phi V_c/2$ then Vertical Spacing of ties shall not exceed the least of:

16 x (longitudal bar diameters)	=	8	in
48 x (tie bar diameter)	=	18	in
Least dimension of column	=	8	in

If $V_u > \phi V_c/2$ then vertical spacing of ties shall not exceed the least of:

s max =	$A_v f_y/(0.75\sqrt{f_c'}\,b)$	=	29.333	in
s max =	$A_v f_y/(50b)$	=	22	in
s max =	d/2 ≤ 24 in	=	3	in

USE # 3 TIES AT 8.00 INCHES ON CENTER WITH (3) IN THE TOP FIVE INCHES OF PIER.

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Corner Footings

(Lines 1 & 4 / Grids A & E)

P_{D+L} = 9.3 kips
Uplift = 4.3 kips

Check Soil Bearing

Allowable Pressure = 3500 psf
Minimum Pressure = 1500 psf
B req'd = **1.63** ft

Use 2.5 ft^2 x 12 inch deep footing reinforced with (4) #4 bars each way.

q = **3355** psf **OK**
q_{min} = **1867** psf **OK**

Uplift

Design uplift =	**4.3**	kips
Slab Thickness =	0	inches
Depth to top of Ftg. =	42	inches
(EW) OS Conc. to CL Footing =	4	inches
(SW) OS Conc. to CL Footing =	9.8	inches
(EW) Length of Wall for Uplift =	7.5	feet
(SW) Length of Wall for Uplift =	11.5	feet
Wall Thickness =	8	inches

Weight of Footing and Soil =	3.13	kips
Weight of Concrete Slab =	0.00	kips
Weight of Foundation Wall & Ftg. =	8.54	kips
Total =	11.67	kips

Factor of Safety = **2.71 > 1.0 OK**

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Endwall Footings

(Lines 2 & 3 / Grid D.1)

P_{D+L} = 0.6 kips
F_H = 2.1 kips
Uplift = 0.0 kips

Use 2.5 ft. x 2.5 ft. x 12 inch deep footing

Check Soil Bearing

Moment Arm =	1	ft
P (total) =	14.8	kips
Overturning Moment =	2.058	kip*ft
OTM Eccentricity =	1.7	inches
Footing Offset =	0	inches
Offset Resisting Moment =	- 0.00	kip*ft
Passive Resisting Moment =	- 0.00	kip*ft
Net Eccentricity =	1.7	inches
B/6 = 5 inches OK		

Allowable Bearing Pressure =	3500	psf
Top of Wall to Grade =	6	in
OS Conc. to CL A.R. =	4	in
Pier Width =	12	in
Pier Depth (wall included) =	8	in
Pier Height =	48	in
Wall Thickness =	8	in
Wall Height =	48	in
Footing Width =	16	in
Footing Depth =	8	in

Bearing Pressure, q (max.) = 3159 psf OK

Minimum Bearing Pressure = 3159 psf OK

Offset footing 0 inches.

Sliding Resistance

Coefficient of Friction =	0.25	
Weight of Pier =	0.40	kips
Weight of Soil Above Footing =	1.51	kips
Weight of Spot Footing =	0.94	kips
Weight of Continuous Wall =	0.39	kips
Weight of Continuous Ftg. =	-0.07	kips
Sliding Resistance from Footing & Pier =	0.33	kips
Sliding Resistance from Soil above Ftg. =	0.38	kips
Sliding Resistance from Vertical Load =	0.16	kips
Sliding Resistance from Wall & Ftg. =	0.08	kips

Wall Length for Sliding =	2.0	ft
Wall Length for Passive Res. =	2.0	ft
Ftg. Width for Sliding/Passive =	2.0	ft
Passive Earth Pressure =	200	psf/ft
Passive Res. (Spot Footing) =	2.00	kips
Passive Res. (Wall & Pier) =	0.70	kips
Passive Res. (Cont. Ftg.) =	-0.26	kips
Total Passive Resistance =	**2.44**	**kips**
Total Sliding Resistance =	**0.95**	**kips**
Factor of Safety =	**1.65**	**> 1.0 OK**

Uplift

Weight of Footing and Pier =	1.34	kips
Weight of Soil Above Footing =	1.51	kips
Weight of Cont. Wall & Footing =	11.32	kips
Total =	14.16	kips

Wall Length used for Uplift =	23.33	ft
Cont. Ftg. Length for Uplift =	23.33	ft
Factor of Safety =	**na**	

Check Footing Flexure (Reinforcing in Direction of Horizontal Force)

q (min.) =	1578	psf	Rebar d' =	3.5	in
OS Footing Edge from Wall =	0.917	ft	Rebar d =	8.5	in
q (at face of wall) =	2579	psf	Rebar fy =	60000	psi
Moment in Footing (Mu, ULT) =	4.58	k*ft	Concrete f'c =	2500	psi
As (req'd by calc.) =	0.120	in^2	ACI 7.12 As (min) =	0.648	in^2

Opposite Direction Reinforcing

Min. Steel Ratio = 0.0018
As per ACI 7.12

Use (4) #4 bars in direction of horizontal force and use (4) #4 bars in the opposite direction.

Check Footing Shear

Shear in Footing (Vu, ULT) =	9.99	kips	
Required Thickness =	7.94	in	OK

For Pier Design

Nu =	**1**	**kips**
Mu =	**13**	**kip*ft**
Vu =	**3**	**kips**

****See pier calculation on page 8.**

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Job: **MVE #23-0706 METAL BUILDING OUTLET CORP.**
Subject: **CLAY HOCKEL C&B HOLDINGS**

Date: 07/31/23
By: BLC

Concrete Column Analysis (ACI 318)

For X-Axis Flexure with Axial Compression or Tension Load
Assuming "Short", Non-Slender Member with Symmetric Reinforcing

Input

f_c' =	2500	psi
f_y =	60	ksi
d' =	2	in
b =	12	in
h =	8	in
ϕ =	0.65	

Loading

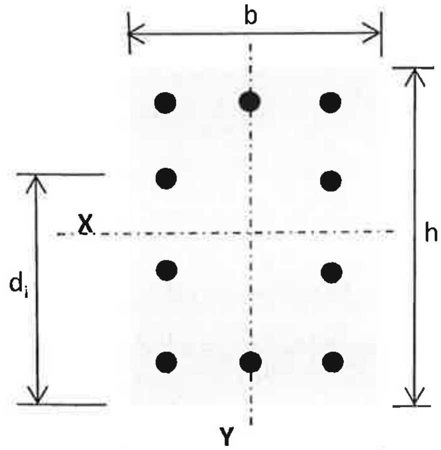
P_{ux} =	51.6	kips
M_{ux} =	13.2	kip-ft
V_{ux} =	3.3	kips

Column Geometry

Bar Size =	4	Total # of Bars	4
# of Bars b Face	2	Tie Size =	3
# of Bars h Face	2		

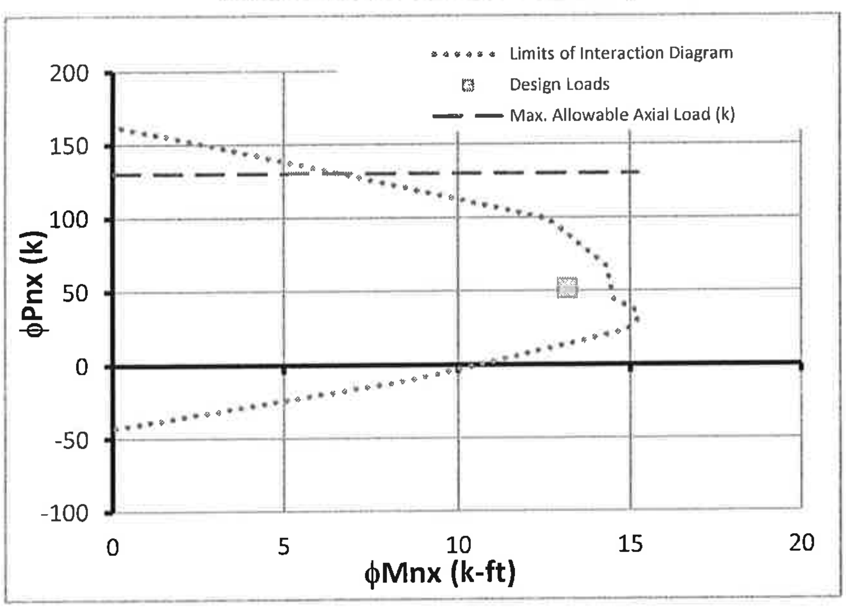
Placement of Reinforcement Steel

	d_i	A_{st}
Edge Layer (d_1)	6.00	0.40
Interior Layer (d_2)	0.00	0.00
Interior Layer (d_3)	0.00	0.00
Interior Layer (d_4)	0.00	0.00
Interior Layer (d_5)	0.00	0.00
Edge Layer (d_6)	2.00	0.40



Typical Member Section

X-AXIS INTERACTION DIAGRAM



DESIGN LOADS FALL WITHIN THE LIMITS OF THE INTERACTION DIAGRAM, THEREFORE, USE (4) # 4 VERTICAL BARS IN COLUMN.

Shear Design ϕV_c = 6.8513 $\phi V_c/2$ = 3.4256 **V_u < $\phi V_c/2$**

If V_u < $\phi V_c/2$ then Vertical Spacing of ties shall not exceed the least of:

16 x (longitudal bar diameters)	=	8	in
48 x (tie bar diameter)	=	18	in
Least dimension of column	=	8	in

If V_u > $\phi V_c/2$ then vertical spacing of ties shall not exceed the least of:

s max =	$A_v f_y/(0.75\sqrt{(f_c')}b)$	=	29.333	in
s max =	$A_v f_y/(50b)$	=	22	in
s max =	d/2 ≤ 24 in	=	3	in

USE # 3 TIES AT 8.00 INCHES ON CENTER WITH (3) IN THE TOP FIVE INCHES OF PIER.

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Worst Case: Sidewall Anchors (Tension and Shear)

ANCHOR ROD GROUP CHECK (per ACI 318-14 Chapter 17, headed anchors)

Tensile Force on Anchors (N_U) =	14.88	kips
Shear Force on Anchors (V_U) =	48.48	kips
Seismic Design Category =	C	
Number of Anchors (n) =	6	
Number of Anchors in Tension (n) =	6	
Anchor Diameter (d_a) = 1 in, A_{SE} =	0.606	in^2
Anchor Spacing Perpendicular to Load (s_1) =	4	in
Anchor Spacing Parallel to Load (s_2) =	4	in
Spacing of outer Anchors (s_0) =	8	in
Embedment Depth (h_{EF}) =	22	in
Yield Strength of Anchors (Fy) =	36	ksi
Tensile Strength of Anchors (F_{uta}) =	58	ksi
Edge Distance in Load Direction (c_{a1}) =	12.25	in
Edge Distance Perpendicular to Load (c_{a2}) =	7	in
Concrete Strength (f'_C) =	2500	psi
Axial Eccentricity (e'_N) =	0	in
Shear Eccentricity (e'_V) =	0	in

Steel Strength of Anchors in Tension (17.4.1.2)

$N_{sa} = nA_{SE}F_{uta}$ = 210.89 kips
ϕ = 0.75 ϕN_{sa} = 158.17 kips

Concrete Breakout Strength of Anchors in Tension (17.4.2)

A_{Nc} = 540 in^2 $A_{Nco} = 9h_{ef}^2$ = 4356 in^2
$\Psi_{ec,N} = 1/(1+2e'_N/3h_{EF}) \le 1.0$ $\Psi_{ec,N}$ = 1.000
$c_{a\,(min.)} < 1.5h_{EF}$? YES $\Psi_{ed,N}$ = 0.764
Cracking at Service Loads ? YES $\Psi_{c,N}$ = 1.000
Headed anchors used, therefore $\Psi_{cp,N}$ = 1.000
$h_{EF} \ge 11$ in. ? YES k_C = 24

$N_b = k_C\lambda_a f'_C{}^{0.5}h_{EF}{}^{5/3}$ = 138.19 kips

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b = 13.08 \text{ kips}$$

Anchor bolts are tied into the footings with rebar cages, so concrete breakout will be limited by the tensile strength of the rebar verticals in the cage.
Number of Verticals = 8 Size of rebar = # 5
Tensile strength of the reinforcement = 148.8 kips (ULT)

ϕ = 0.75 ϕN_{CBG} = 111.60 kips

Pullout Strength of Anchors in Tension (17.4.3)

A_{BRG} = 1.163 in^2
$N_P = 8A_{BRG}f'_C$ = 23.26 kips
Cracking at Service Loads? YES $\Psi_{c,P}$ = 1
N_{PN} (group) = $n\Psi_{c,P}N_P$ = 139.56 kips
ϕ = 0.7 ϕN_{PN} (group) = 97.69 kips

Side-face Blowout of Anchors in Tension (17.4.4)

$h_{EF} > 2.5\,c_{a1}$ NO **SIDE-FACE BLOWOUT WILL**
$s_1 < 6c_{a1}$? YES **NOT CONTROL**
$c_{a2} < 3c_{a1}$? YES R = 0.39
$N_{sb} = 160c_{a1}A_{BRG}{}^{0.5}\lambda_a f'_C{}^{0.5}R$ = 41.52 kips
Therefore, $N_{sbg} = (1+s_0/6c_{a1})N_{sb}$ = 46.04 kips
ϕ = 0.75 ϕN_{sbg} = 34.53 kips

Steel Strength of Anchors in Shear (17.5.1)

Built-up grout pads used? NO
$V_S = 0.6nA_{SE}F_{uta}$ = 126.53 kips
ϕ = 0.65 ϕV_S = 82.25 kips

Concrete Breakout Strength of Anchors in Shear (17.5.2)

A_{Vc} = 616 in^2 $A_{Vco} = 4.5(c_{a1})^2$ = 675.281 in^2
$\Psi_{ec,V} = 1/(1+2e'_V/3c_{a1}) \le 1.0$ $\Psi_{ec,V}$ = 1.000
$c_{a2} \ge 1.5c_{a1}$? NO $\Psi_{ed,V}$ = 0.814
Cracking at Service Loads ? YES $\Psi_{c,V}$ = 1.200
Thickness: h_a = 48 in $\Psi_{h,V}$ = 1.000
$V_b = 7\,(L_e/d_a)^{0.2}d_a{}^{0.5}f'c^{0.5}c_{a1}{}^{1.5}$ = 22.75 kips
$V_b = 9\lambda_a f'c^{0.5}c_{a1}{}^{1.5}$ = 19.29 kips CONTROLS

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_b = 17.20 \text{ kips}$$

Concrete breakout in shear will be resisted by rebar ties at the top of the footing, tensile capacity of the bars will control.
Tie Size = # 4 bar Bar Area = 0.2 in^2
Number of Ties at top = 3 Fy = 60 ksi
Tensile Capacity of Ties = 108 kips
Therefore, V_{cbg} = 108.00 kips
ϕ = 0.75 ϕV_{cbg} = 81.00 kips

Concrete Pryout Strength of Anchors in Shear (17.5.3)

Concrete pryout of anchors in shear will only occur where anchors are relatively stiff and short.

$$\frac{h_{EF}}{d_0} = 22 > 6$$

THEREFORE, CONCRETE PRYOUT WILL NOT CONTROL SHEAR DESIGN OF ANCHORS.

ANCHOR ROD GROUP CAPACITY

SDC = C, D, E, & F REDUCE CONCRETE ALLOW. TENSION BY 0.75

ALLOW. TENSION (ΦN_N) = 73.27 kips > 14.9 kips **OK**
ALLOW. SHEAR (ΦV_N) = 81.00 kips > 48.5 kips **OK**

CHECK SHEAR/TENSION INTERACTION

$N_{UA} > 0.2\Phi N_N$? YES
$V_{UA} > 0.2\Phi V_N$? YES
UNITY CHECK REQUIRED

$$\frac{N_{UA}}{\Phi N_N} + \frac{V_{UA}}{\Phi V_N} = \mathbf{0.80} < \mathbf{1.20}$$ **OK**

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Worst Case: Endwall Anchors (Tension and Shear)(Wind Controls)

ANCHOR ROD GROUP CHECK (per ACI 318-14 Chapter 17, headed anchors)

Tensile Force on Anchors (N_U) =	9.44	kips
Shear Force on Anchors (V_U) =	4.64	kips
Seismic Design Category =	C	
Number of Anchors (n) =	4	
Number of Anchors in Tension (n) =	4	
Anchor Diameter (d_a) = 0.625 in A_{SE} =	0.226	in^2
Anchor Spacing Perpendicular to Load (s_1) =	4	in
Anchor Spacing Parallel to Load (s_2) =	3	in
Spacing of outer Anchors (s_0) =	3	in
Embedment Depth (h_{EF}) =	12	in
Yield Strength of Anchors (Fy) =	36	ksi
Tensile Strength of Anchors (F_{uta}) =	58	ksi
Edge Distance in Load Direction (c_{a1}) =	2.5	in
Edge Distance Perpendicular to Load (c_{a2}) =	12	in
Concrete Strength (f'_C) =	2500	psi
Axial Eccentricity (e'_N) =	0	in
Shear Eccentricity (e'_V) =	0	in

Steel Strength of Anchors in Tension (17.4.1.2)

$N_{sa} = nA_{SE}F_{uta}$ = 52.43 kips
ϕ = 0.75 ϕN_{sa} = 39.32 kips

Concrete Breakout Strength of Anchors in Tension (17.4.2)

A_{Nc} = 96 in^2 $A_{Nco} = 9h_{ef}^2$ = 1296 in^2
$\Psi_{ec,N} = 1/(1+2e'_N/3h_{EF}) <= 1.0$ $\Psi_{ec,N}$ = 1.000
$c_{a\,(min.)} < 1.5h_{EF}$? YES $\Psi_{ed,N}$ = 0.742
Cracking at Service Loads ? YES $\Psi_{c,N}$ = 1.000
Headed anchors used, therefore $\Psi_{cp,N}$ = 1.000
$h_{EF} >= 11$ in. ? YES k_C = 24

$N_b = k_C\lambda_a f'_C{}^{0.5}h_{EF}{}^{5/3}$ = 50.32 kips

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b = 2.76 \text{ kips}$$

Anchor bolts are tied into the footings with rebar cages, so concrete breakout will be limited by the tensile strength of the rebar verticals in the cage.
Number of Verticals = 6 Size of rebar = # 4
Tensile strength of the reinforcement = 72 kips (ULT)

ϕ = 0.75 ϕN_{CBG} = 54.00 kips

Pullout Strength of Anchors in Tension (17.4.3)

A_{BRG} = 0.454 in^2
$N_P = 8A_{BRG}f'_C$ = 9.08 kips
Cracking at Service Loads? YES $\Psi_{c,P}$ = 1
N_{PN} (group) = $n\Psi_{c,P}N_P$ = 36.32 kips
ϕ = 0.7 ϕN_{PN} (group) = 25.42 kips

Side-face Blowout of Anchors in Tension (17.4.4)

$h_{EF} > 2.5\,c_{a1}$ YES
$s_1 < 6c_{a1}$? YES
$c_{a2} < 3c_{a1}$? NO R = 1
$N_{sb} = 160c_{a1}A_{BRG}{}^{0.5}\lambda_a f'_C{}^{0.5}R$ = 13.48 kips
Therefore, $N_{sbg} = (1+s_0/6c_{a1})N_{sb}$ = 16.17 kips
ϕ = 0.75 ϕN_{sbg} = 12.13 kips

Steel Strength of Anchors in Shear (17.5.1)

Built-up grout pads used? NO
$V_S = 0.6nA_{SE}F_{uta}$ = 31.46 kips
ϕ = 0.65 ϕV_S = 20.45 kips

Concrete Breakout Strength of Anchors in Shear (17.5.2)

A_{Vc} = 28.125 in^2 $A_{Vco} = 4.5(c_{a1})^2$ = 28.125 in^2
$\Psi_{ec,V} = 1/(1+2e'_V/3c_{a1}) <= 1.0$ $\Psi_{ec,V}$ = 1.000
$c_{a2} >= 1.5c_{a1}$? YES $\Psi_{ed,V}$ = 1.000
Cracking at Service Loads ? YES $\Psi_{c,V}$ = 1.200
Thickness: h_a = 48 in $\Psi_{h,V}$ = 1.000
$V_b = 7\,(L_e/d_a)^{0.2}d_a{}^{0.5}f'c^{0.5}c_{a1}{}^{1.5}$ = 1.66 kips CONTROLS
$V_b = 9\lambda_a f'c^{0.5}c_{a1}{}^{1.5}$ = 1.78 kips

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_b = 1.99 \text{ kips}$$

Concrete breakout in shear will be resisted by rebar ties at the top of the footing, tensile capacity of the bars will control.
Tie Size = # 3 bar Bar Area = 0.11 in^2
Number of Ties at top = 3 Fy = 60 ksi
Tensile Capacity of Ties = 39.6 kips
Therefore, V_{cbg} = 39.60 kips
ϕ = 0.75 ϕV_{cbg} = 29.70 kips

Concrete Pryout Strength of Anchors in Shear (17.5.3)

Concrete pryout of anchors in shear will only occur where anchors are relatively stiff and short.

$$\frac{h_{EF}}{d_0} = 19.2 > 6$$

THEREFORE, CONCRETE PRYOUT WILL NOT CONTROL SHEAR DESIGN OF ANCHORS.

ANCHOR ROD GROUP CAPACITY

ALLOW. TENSION (ΦN_N) = 12.13 kips > 9.44 kips **OK**
ALLOW. SHEAR (ΦV_N) = 20.45 kips > 4.64 kips **OK**

CHECK SHEAR/TENSION INTERACTION

$N_{UA} > 0.2\Phi N_N$? YES
$V_{UA} > 0.2\Phi V_N$? YES
UNITY CHECK REQUIRED

$$\frac{N_{UA}}{\Phi N_N} + \frac{V_{UA}}{\Phi V_N} = 1.01 < 1.20 \quad \text{OK}$$

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Project Title: Clay Hockel C&B Holdings
Engineer: BLC
Project ID: 23-0706
Project Descr: 65'x70' Metal Building

REVIEWED FOR CODE COMPLIANCE 09/18/2023

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Concrete Beam

Project File: 23-0706.ec6

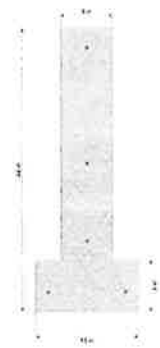
DESCRIPTION: Grade Beam

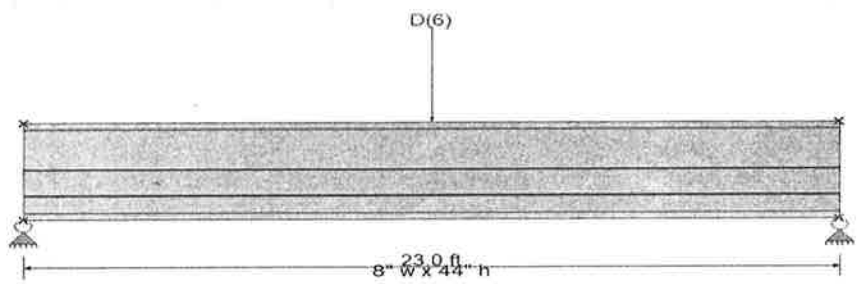
CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

General Information

f'c	=	2.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c$^{1/2}$ * 7.50	=	375.0 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	2,850.0 ksi	Fy - Stirrups		60.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		4
			Number of Resisting Legs Per Stirrup	=	1.0





Cross Section & Reinforcing Details

Inverted Tee Section, Stem Width = 8.0 in, Total Height = 44.0 in, Top Flange Width = 16.0 in, Flange Thickness = 8.0 in
Span #1 Reinforcing....
2-#4 at 3.0 in from Bottom, from 0.0 to 23.0 ft in this span
1-#4 at 11.0 in from Bottom, from 0.0 to 23.0 ft in this span
1-#4 at 3.0 in from Top, from 0.0 to 23.0 ft in this span
1-#4 at 21.0 in from Top, from 0.0 to 23.0 ft in this span

Beam self weight calculated and added to loads
Point Load : D = 6.0 k @ 11.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.742** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	86.991	k-ft
Mn * Phi : Allowable	117.297	k-ft
Location of maximum on span	11.521	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0	<360.0	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360.0	
Max Downward Total Deflection	0.025 in	Ratio =	11159	>=240.0	Span: 1 : D Only
Max Upward Total Deflection	0.000 in	Ratio =	0	<240.0	Span: 1 : D Only

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	7.817	7.817
Max Upward from Load Combinations	4.690	4.690
Max Upward from Load Cases	7.817	7.817
D Only	7.817	7.817
+0.60D	4.690	4.690

Shear Stirrup Requirements

Entire Beam Span Length : Vu < Phi*Vc / 2, Req'd Vs = Not Reqd per 9.6.3.1, Stirrups are not required.

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REVIEWED FOR CODE COMPLIANCE 09/18/2023

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Concrete Beam

Project File: 23-0706.ec6

LIC# : KW-06014791, Build:20.23.07.20 MOUNTAIN VIEW ENGINEERING, INC.

DESCRIPTION: Grade Beam

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	23.000	86.99	117.30	0.74
+1.40D					
Span # 1	1	23.000	86.99	117.30	0.74
+1.20D					
Span # 1	1	23.000	74.56	117.30	0.64
+0.90D					
Span # 1	1	23.000	55.92	117.30	0.48

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
D Only	1	0.0247	11.500		0.0000	0.000