REVERENCE ENGINEERING

STRUCTURAL CALCULATIONS

for

Church of Christ LED Sign

at

201 NW Chipman Rd

Lee's Summit, MO 64063

Prepared for:

Mega LED Techonology

Package Type:

Initial Submittal

Project #:

2503169

DESIGN SPECIFICATIONS

1. International Building Code (IBC) 2018
2. ASCE 7-16: Minimum Design Loads for Buildings and Other Structures
3. ACI 318-14: Building Code Requirements for Structural Concrete
4. ANSI/AISC 360-16: Specification for Structural Steel Buildings
5. Aluminum Design Manual (ADM-1) 2015

DESIGN CRITERA

::Wind::	V_{ult} =	115	mph
Exposure: C			
::Ground Snow Load::	p_g =	20	psf
::Soils::			
Per Building Code Presumptive Class 4			
Allowable Lateral Bearing:		150	psf/ft
Allowable Vertical Bearing:		2000	psf

STATE OF MISSOURI
BRETT J. SAUERS
NUMBER
PE-2019010905
PROFESSIONAL ENGINEER

Date Signed: 01/04/2025

Reverence Engineering
501 W Broadway, STE 425
San Diego, CA 92101
(o) 619-354-1152
projects@reverenceengineering.com

Wind Pressure Analysis

Design Wind Loads on Solid Signs

Notes & Assumptions:

1. Wind Loads on other structures and building appurtenances per Chapter 29, Section 29.5 (Solid Signs).

V_{ult} =	115	mph	Basic Wind Speed	z_g =	900	ft	Terrain Exposure Constant [Table 26.9-1]
Exp:	C	-	Exposure Category	α =	9.5	-	Terrain Exposure Constant [Table 26.9-1]
ε =	100%	-	Solid to Gross Area Ratio (porosity)	K_d =	0.85		Directionality Factor [Table 26.6-1]
Add'l =	0%	-	Add'l. capacity applied to p_z	K_{zt} =	1		Topographic Factor [26.8.2]
b =	6	ft	2-pole spacing ('0' for one pole or other)	G =	0.85	-	Gust Effect Factor [26.9.1]

Cab. #	h (ft)	s (ft)	B (ft)	Encl.[1]	t^2 (ft)	s/h	B/s	R_{min}	R_{max}	~~~~~~ Cases A & B C_f Interpolation ~~~~~~						C_f	0.2B	Case B Distrib.
1	10	5	8	Yes	1.0156	0.50	1.60	0.20	0.13	1.75	1.70	1.80	1.80	1.72	1.80	1.67	1.35	**0.72**
2																		
3																		
4																		
5																		
6																		
7																		
8																		

[1]For double-faced signs with all sides enclosed select "Yes" for C_f reduction per code. ("No" or blank is conservative if unknown)

[2]Cabinet Thickness (Use 0 or leave blank for unknown)

Figure 29.4-1 C_f Cases A & B

s/h	Aspect Ratio B/s												
	0	0.05	0.1	0.2	0.5	1	2	4	5	10	20	30	45	100
1.00	1.80	1.80	1.70	1.65	1.55	1.45	1.40	1.35	1.35	1.30	1.30	1.30	1.30	1.30
0.90	1.85	1.85	1.75	1.70	1.60	1.55	1.50	1.45	1.45	1.40	1.40	1.40	1.40	1.40
0.70	1.90	1.90	1.85	1.75	1.70	1.65	1.60	1.60	1.55	1.55	1.55	1.55	1.55	1.55
0.50	1.95	1.95	1.85	1.80	1.75	1.75	1.70	1.70	1.70	1.70	1.70	1.70	1.75	1.75
0.30	1.95	1.95	1.90	1.85	1.80	1.80	1.80	1.80	1.80	1.80	1.85	1.85	1.85	1.85
0.20	1.95	1.95	1.90	1.85	1.80	1.80	1.80	1.80	1.80	1.85	1.90	1.90	1.95	1.95
0.16	1.95	1.95	1.90	1.85	1.85	1.80	1.80	1.85	1.85	1.85	1.90	1.90	1.95	1.95
0.00	1.95	1.95	1.90	1.85	1.85	1.80	1.80	1.85	1.85	1.85	1.90	1.90	1.95	1.95

h = Average Height (ft) at top of section/element
s = Vertical sign dimension (ft) `
B = Horizontal sign dimension (ft)
z = Centroid of Section Height
K_z = Velocity Pressure Exposure Coefficient
q_h = Velocity Pressure (psf) $q_z = 0.00256 * K_d * K_z * K_{zt} * V^2$
p = Design Wind Pressure (psf) $p = q_h * G * C_f$

s/h = Cabinet to Average Height Ratio
B/s = Cabinet Width to Cabinet Height Ratio

Wind Load Analysis

SIGN BUILD

FORCE is PER POLE and reflects distribution per Case B to applicable components

LOW - > HIGH

#	Δh (ft)	Width (ft)	Type	Cant.	Cab. #	h (ft)	z (ft)	C_f	K_z	q_z (psf)	p_z (psf)	A_F (sqft)	F_u (kip)	M_u (k-ft)	T_u (k-ft)	M_{u1}
1	5	6.25	Sq. Normal			**5.0**	2.5	1.3	0.8489	24.429	26.994	31.3	0.42	1.05	0.0	
2	5	8	Cabinet		1	**10.0**	7.5	1.6735	0.8489	24.429	34.75	40.0	1.01	7.55	0.0	
3																
4																
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25																
H = 10 ft	← Overall height check[1]								"Grade Reactions" (Intermediate Calculation) →				1.4 kip	8.6 kip-ft	0.0 kip-ft	0.0 kip-ft

[1]For evaluation, may not be equivalent to the actual structure height above grade

Splice or Section Loads / Reactions

Notes & Assumptions:

1. Moments this section include an additional 10% per 29.4-1 if s/h=1.
2. Moments and Shears include either 50% split or Distribution Factor in the case of 2 poles (torsion is zero).
3. Splice Reactions may be a physical splice in the system or any other point of interest

Splice	h (ft)	Stage	w (ft)	h+ (ft)	p (psf)	A_F (sqft)	F_u (kip)	z (ft)
1	5	2	8	10	34.75	40.00	1.01	2.50
2								
3								
4								
5								
6								

V (kip)	M (k-ft)	T (k-ft)
1.007	2.517	0.000

	V (kip)	M (k-ft)	T (k-ft)
GRADE REACTIONS* →	1.43	8.61	0.00

*Grade Reactions are not necessarily at physical grade, but represent the reactions at the lowest elevation (Elev. = 0'-0") calculated from the above "Sign Build" table.

Foundation Design

SPREAD FOOTING CALCULATION W/ DIRECT BURIAL POST

FOOTING PARAMETERS:				INITIAL REINFORCEMENT:				MATERIAL PROPERTIES:			
L =	3.25	ft	Width (perp to sign face)	#	5	bars		f'_c =	2500	psi	Concrete Comp. Strength
B =	7.5	ft	Length (parallel to sign face)	d_b =	0.625	in	Bar diameter	f_y =	60	ksi	Rebar Yield Strength
H =	42	in	Footing Depth	A_b =	0.3068	in²	Bar area	E =	29000	ksi	
H_s =	0	in	Depth of soil above	CC =	3	in	Clear Cover	γ_c =	145	pcf	Concrete Specific Weight
d =	4	in	Member depth					γ_s =	25	pcf	Soil Specific Weight
								λ =	1	-	Lt Wt Agg Factor

LOAD PARAMETERS:

M_u =	11.9	k-ft	Moment at grade	M_s =	7.12845	k-ft	Service M (0.6^*M_u)
V_u =	2.0	k	Shear at grade	V_s =	1.18338	k	Service V (0.6^*M_u)
P_u =	720.0	lb	Axial at grade	P_s =	600	lb	Service P (P_u/1.2)

CONCRETE VOLUME (GROSS PAD ONLY)
= 85.313 ft³
= 3.1597 yd³

SOIL PROPERTIES

q_a =	2000	psf	A_f =	24.375	ft²	Footing Area
x	1.33	-	I_f =	21.455	ft⁴	Footing Inertia
q_a' =	2660	psf				

ASD ALT LC #7 - D + ω0.6W				ASD LC #7 - .6D + 0.6W			LRFD LC #4 - 1.2D + W		
ω =	1.3	-		M_{OT} =	11.27	k-ft	$q_{u,+}$ =	1538.4	psf
q_a' =	2660	psf		M_R =	12.646	k-ft	$q_{u,-}$ =	-261.31	psf
$q_{s,+}$ =	1233.99	psf	BEARING: **OK**	OVERTURN: **OK**			*slope* =	*-553.8*	*lb/ft*
$q_{s,-}$ =	-677.26	psf	UPLIFT: **Yes**						

ESTIMATE FLEXURAL CRITICAL SECTION (BOTTOM FLEXURAL REBAR)

L' =	1.4583	ft	t =	0.015	in	t =	0.1379	in	Req'd Steel per ft.	0.0112	in²	
q_u =	731	psf	λ =	0.0075	in	λ =	0.069	in	Bar Spacing:	18	in	
M_u =	10.122	kip-ft	M_n =	14.665	k-in	M_n =	134.75	k-in	OVERRIDE	12	in	
=	121.5	k-in	φ =	0.9	-	φ =	0.9	-	Available width:	83.375	in	
d =	38.375	in	φM_n =	13.198	k-in	φM_n =	121.27	k-in	# Bars	7	-	
λ =	3.8375	in	D/C:	9.2032	-	D/C:	1.00	-	Balance Ea. End:	9	in	
φ =	0.9	-	A'_s =	0.0586	in²	A'_s =	0.0587	in²	A_s =	2.1476	in²	**OK**
A_s =	0.0064	in²	A'_c =	1.6554	in²	x	1.33					
A_c =	0.1799	in²				$A_{s,req}$ =	0.0781	in²	**Use (7) - #5 Bars @ 12 in O.C.**			

FLEXURAL CRITICAL SECTION (TOP FLEXURAL REBAR)

L' =	0.4719	ft	A_s =	0.0064	in²	φM_n =	13.198	k-in
M_u =	1.7961	kip-ft	A_c =	0.1799	in²	D/C:	1.633	-
=	21.553	k-in	t =	0.015	in	A'_s =	0.0104	in²
d =	38.375	in	λ =	0.0075	in	x	1.33	in²
λ =	3.8375	in	M_n =	14.665	k-in	$A_{s,req}$ =	0.0138	in²
φ =	0.9	-	φ =	0.9	-			

SHEAR CRITICAL SECTION

Assume concrete outside top and bottom layers carries no shear

V_c =	324	kip
φ =	0.75	-
φV_c =	243	kip
V_u =	12.41	kip
D/C:	0.05	**OK**

Pole Design

MEMBER SELECTION **STAGE: 1** **LRFD DESIGN**

Section Type: SquareHSS

M_u =	8.61	k-ft	T_u =	0.00	k-ft	φ =	0.9	F_y = 50 ksi
V_u =	1.43	kip	L =	5	ft	=	60 in	Z_{req} = 2.2949 in^3

		Section Name	Z (in^3)	φM_n (k-ft)	D/C	Check	φV_n (kip)	D/C	Check	φT_n (k-ft)	D/C	Check	Combined D/C	Combined Check
( )	LEAST Z (Flexure Only)	HSS3X3X1/4	2.48	9.3	0.9254	OK!	28.939	0.0494	OK!	7.9177	0	OK!	0.9254	OK!
(X)	MANUAL SELECTION	HSS4X4X3/16	3.67	13.763	0.6253	OK!	32.698	0.0437	OK!	11.416	0	OK!	0.6253	OK!

MEMBER SELECTION **STAGE: 2** **LRFD DESIGN**

Section Type: SquareHSS

M_u =	2.52	k-ft	T_u =	0.00	k-ft	φ =	0.9	F_y = 50 ksi
V_u =	1.01	kip	L =	5	ft	=	60 in	Z_{req} = 0.6712 in^3

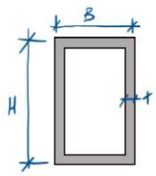
		Section Name	Z (in^3)	φM_n (k-ft)	D/C	Check	φV_n (kip)	D/C	Check	φT_n (k-ft)	D/C	Check	Combined D/C	Combined Check
( )	LEAST Z (Flexure Only)	HSS2-1/4X2-1/4X1/8	0.755	2.8313	0.889	OK!	11.902	0.0846	OK!	2.3636	0	OK!	0.889	OK!
(X)	MANUAL SELECTION	HSS3X3X1/8	1.4	5.25	0.4795	OK!	16.6	0.0607	OK!	4.3281	0	OK!	0.4795	OK!

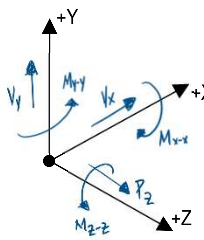
Match Plate Design

ECCENTRIC WELD GROUP (RECTANGULAR SECTION, MULTI-AXIAL)
Top Plate

Mild Steel		
F_{EXX} =	70000	psi

B =	3	in	Weld Pattern Width
H =	3	in	Weld Pattern Height (Strong Axis)
S =	0.1875	in	Fillet weld size
V_y =	1.007	kip	In Plane Force
V_x =	0	kip	In Plane Force
P_z =	0	kip	Out-of-Plane Force
$M_{x\text{-}x}$ =	30.205	kip-in	Weld Group Moment
$M_{y\text{-}y}$ =	0	kip-in	Weld Group Moment
$M_{z\text{-}z}$ =	0	kip-in	Weld Group Torsion
F_{EXX} =	70	ksi	Weld tensile strength
$F_{n,w}$ =	42	ksi	Nominal weld strength
φ =	0.75	-	Strength Reduction Factor
$\varphi F_{n,w}$ =	31.5	ksi	Weld design capacity





a =	0.1326	in	Weld Throat
A_w =	1.5908	in^2	Fillet weld area
$I_{w,x\text{-}x}$ =	2.3861	in	Weld Moment of Inertia
$c_{x\text{-}x}$ =	1.5	in	Center of Gravity
$I_{w,y\text{-}y}$ =	2.3861	in	Weld Moment of Inertia
$c_{y\text{-}y}$ =	1.5	in	Center of Gravity
J =	4.7723	in	Polar Moment of Inertia
c_J =	4.2426	in	Radial Distance
$\tau_{v,y}$ =	0.6329	ksi	Weld stress due to V_y
$\tau_{v,x}$ =	0	ksi	Weld stress due to V_x
$\sigma_{v,z}$ =	0	ksi	Weld stress due to P_z
$\sigma_{b,x\text{-}x}$ =	18.988	ksi	Weld force due to $M_{x\text{-}x}$
$\sigma_{b,y\text{-}y}$ =	0	ksi	Weld force due to $M_{y\text{-}y}$
$\tau_{z\text{-}z}$ =	0	ksi	Torsion shear stress due to $M_{z\text{-}z}$
$\tau_{z\text{-}z,y}$ =	0	ksi	Torsion shear stress Y-component
$\tau_{z\text{-}z,x}$ =	0	ksi	Torsion shear stress X-component

COMPARATIVE STRESSES

τ_x =	0	ksi	Total shear stress in X			
τ_y =	0.6329	ksi	Total shear stress in Y			
σ_z =	18.988	ksi	Total normal stress in Z			
f_{RES} =	18.999	ksi	Resultant Stress	D/C:	0.6031	OK

STEEL PLATE — SQUARE POST, SQUARE PLATE — UNIAXIAL BENDING
Top Plate

M_{u1} =	2.517	k-ft	Design Moment
b =	8	in	Plate width
edge =	1	in	Bolt Edge Distance
d =	3	in	Tube Depth
s_b =	6	in	Fastener spacing
arm =	1.5	in	Plate Bending Arm
b_{eff} =	6	in	Bending Width Override
b_{eff} =	6	in	Effective Bending Width

T_{grp1} =	5.0342	kip	Group Tension
$M_{u,pl}$ =	7.5514	k-in	Plate Moment
Z =	0.375	in^3	Plastic Section Modulus
M_n =	13.5	k-in	Nominal Yield Moment
φ_b =	0.9	-	Strength Reduction Factor
φM_n =	12.15	k-in	Yield Moment Capacity
D/C:	0.6215	OK	

F_y =	36	ksi	Plate Yield
t =	0.50	in	Plate thickness
n =	2	-	# T fasteners
Fastener T_u =	2.5	$kip_{(LRFD)}$	
x 0.6 =	1.5	$kip_{(ASD)}$	

A307 BOLT CHECK

T_u =	2.5	kip	Fastener Tension
V_u =	0.503	kip	Fastener Shear
d_{bolt} =	0.5	in	Bolt diameter
A_{bolt}	0.1963	in^2	Area of Bolt
f_v	2.5639	ksi	Required Shear Stress
F'_{nt}	43.5	ksi	Modified Nominal Tensile Stress

F_u =	58	ksi
F_{nt} =	43.5	ksi
F_{nv} =	26.1	ksi

φ	0.75	-	Strength Reduction Factor
φT_n =	6.4059	kip	Tensile Rupture Capacity
D/C:	0.3929	OK	
φV_n =	3.8435	kip	Shear Rupture Capacity
D/C:	0.131	OK	
φR_n =	6.4059	kip	Combined Rupture Capacity
D/C:	0.3929	OK	

Match Plate Design

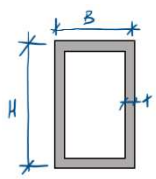
STEEL PLATE | **SQUARE POST, SQUARE PLATE** | **UNIAXIAL BENDING** | *Bottom Match Plate*

M_{u1} =	2.517	k-ft	Design Moment	T_{grp1} =	5.0342	kip	Group Tension	F_y =	36	ksi	Plate Yield
b =	8	in	Plate width	$M_{u,pl}$ =	5.0342	k-in	Plate Moment	t =	0.5	in	Plate thickness
edge =	1	in	Bolt Edge Distance	Z =	0.25	in^3	Plastic Section Modulus	n =	2	-	# T fasteners
d =	4	in	Tube Depth	M_n =	9	k-in	Nominal Yield Moment				
s_b =	6	in	Fastener spacing	φ_b =	0.9	-	Strength Reduction Factor				
arm =	1	in	Plate Bending Arm	φM_n =	8.1	k-in	Yield Moment Capacity				
b_{eff} =	4	in	Bending Width Override					Fastener T_u =	2.5	$kip_{(LRFD)}$	
b_{eff} =	4	in	Effective Bending Width	D/C:	0.6215	OK		x 0.6 =	1.5	$kip_{(ASD)}$	

ECCENTRIC WELD GROUP (RECTANGULAR SECTION, MULTI-AXIAL)

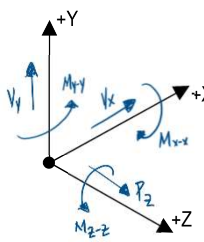
B =	4	in	Weld Pattern Width	a =	0.1326	in	Weld Throat
H =	4	in	Weld Pattern Height (Strong Axis)	A_w =	2.121	in^2	Fillet weld area
S =	0.1875	in	Fillet weld size				

Mild Steel		
F_{EXX} =	70000	psi



V_y =	1.007	kip	In Plane Force	$I_{w,x\text{-}x}$ =	5.656	in^4	Weld Moment of Inertia
V_x =	0	kip	In Plane Force	$c_{x\text{-}x}$ =	2	in	Center of Gravity
P_z =	0	kip	Out-of-Plane Force	$I_{w,y\text{-}y}$ =	5.656	in^4	Weld Moment of Inertia
$M_{x\text{-}x}$ =	30.21	kip-in	Weld Group Moment	$c_{y\text{-}y}$ =	2	in	Center of Gravity
$M_{y\text{-}y}$ =	0	kip-in	Weld Group Moment	J =	11.312	in^4	Polar Moment of Inertia
$M_{z\text{-}z}$ =	0	kip-in	Weld Group Torsion	c_J =	5.6569	in	Radial Distance

F_{EXX} =	70	ksi	Weld tensile strength	$\tau_{v,y}$ =	0.4747	ksi	Weld stress due to V_y
$F_{n,w}$ =	42	ksi	Nominal weld strength	$\tau_{v,x}$ =	0	ksi	Weld stress due to V_x
φ =	0.75	-	Strength Reduction Factor	$\sigma_{v,z}$ =	0	ksi	Weld stress due to P_z
$\varphi F_{n,w}$ =	31.5	ksi	Weld design capacity	$\sigma_{b,x\text{-}x}$ =	10.681	ksi	Weld force due to $M_{x\text{-}x}$
				$\sigma_{b,y\text{-}y}$ =	0	ksi	Weld force due to $M_{y\text{-}y}$
				$\tau_{z\text{-}z}$ =	0	ksi	Torsion shear stress due to $M_{z\text{-}z}$
				$\tau_{z\text{-}z,y}$ =	0	ksi	Torsion shear stress Y-component
				$\tau_{z\text{-}z,x}$ =	0	ksi	Torsion shear stress X-component



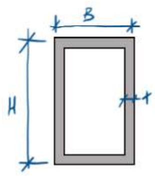
COMPARATIVE STRESSES

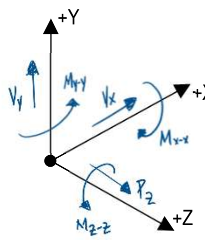
τ_x =	0	ksi	Total shear stress in X		
τ_y =	0.4747	ksi	Total shear stress in Y		
σ_z =	10.681	ksi	Total normal stress in Z		
f_{RES} =	10.691	ksi	Resultant Stress	D/C: 0.3394	OK

Base Plate Design

ECCENTRIC WELD GROUP (RECTANGULAR SECTION, MULTI-AXIAL)

B =	4	in	Weld Pattern Width
H =	4	in	Weld Pattern Height (Strong Axis)
S =	0.25	in	Fillet weld size
V_y =	1.43	kip	In Plane Force
V_x =	0	kip	In Plane Force
P_z =	0	kip	Out-of-Plane Force
$M_{x\text{-}x}$ =	103.27	kip-in	Weld Group Moment
$M_{y\text{-}y}$ =	0	kip-in	Weld Group Moment
$M_{z\text{-}z}$ =	0	kip-in	Weld Group Torsion
F_{EXX} =	70	ksi	Weld tensile strength
$F_{n,w}$ =	42	ksi	Nominal weld strength
φ =	0.75	-	Strength Reduction Factor
$\varphi F_{n,w}$ =	31.5	ksi	Weld design capacity





a =	0.1768	in	Weld Throat
A_w =	2.828	in^2	Fillet weld area
$I_{w,x\text{-}x}$ =	7.5413	in	Weld Moment of Inertia
$c_{x\text{-}x}$ =	2	in	Center of Gravity
$I_{w,y\text{-}y}$ =	7.5413	in	Weld Moment of Inertia
$c_{y\text{-}y}$ =	2	in	Center of Gravity
J =	15.083	in	Polar Moment of Inertia
c_J =	5.6569	in	Radial Distance
$\tau_{v,y}$ =	0.5052	ksi	Weld stress due to V_y
$\tau_{v,x}$ =	0	ksi	Weld stress due to V_x
$\sigma_{v,z}$ =	0	ksi	Weld stress due to P_z
$\sigma_{b,x\text{-}x}$ =	27.388	ksi	Weld force due to $M_{x\text{-}x}$
$\sigma_{b,y\text{-}y}$ =	0	ksi	Weld force due to $M_{y\text{-}y}$
$\tau_{z\text{-}z}$ =	0	ksi	Torsion shear stress due to $M_{z\text{-}z}$
$\tau_{z\text{-}z,y}$ =	0	ksi	Torsion shear stress Y-component
$\tau_{z\text{-}z,x}$ =	0	ksi	Torsion shear stress X-component

Mild Steel		
F_{EXX} =	70000	psi

COMPARATIVE STRESSES

τ_x =	0	ksi	Total shear stress in X			
τ_y =	0.5052	ksi	Total shear stress in Y			
σ_z =	27.388	ksi	Total normal stress in Z			
f_{RES} =	27.392	ksi	Resultant Stress	D/C:	0.8696	OK

STEEL PLATE — SQUARE POST, SQUARE PLATE — UNIAXIAL BENDING

Base Plate

M_{u1} =	8.61	k-ft	Design Moment
b =	12	in	Plate width
edge =	1.5	in	Bolt Edge Distance
d =	4	in	Tube Depth
s_b =	9	in	Fastener spacing
arm =	2.5	in	Plate Bending Arm
b_{eff} =	6	in	Bending Width Override
b_{eff} =	6	in	Effective Bending Width

T_{grp1} =	11.474	kip	Group Tension
$M_{u,pl}$ =	28.686	k-in	Plate Moment
Z =	1.5	in^3	Plastic Section Modulus
M_n =	54	k-in	Nominal Yield Moment
φ_b =	0.9	-	Strength Reduction Factor
φM_n =	48.6	k-in	Yield Moment Capacity
D/C:	0.5902	OK	

F_y =	36	ksi	Plate Yield
t =	1.00	in	Plate thickness
n =	1	-	# T fasteners

Fastener T_u = 11.5 $kip_{(LRFD)}$
x 0.6 = 6.9 $kip_{(ASD)}$

Anchor Design

F1554 ANCHOR ROD (STEEL) CHECK

T_u =	11.5	kip	Rod Tension	N_{sa} =	16.95	kip		Nominal Tensile Capacity
V_u =	1.43	kip	Rod Shear	φ =	0.75	-		
d_{rod} =	0.625	in	Rod diameter	φN_{sa} =	12.713	kip		
GR. :	55	-	Rod Grade	D/C :	0.9026		OK	Tension Capacity Ratio
	x		Grout Pad? (V_n x 0.80)	V_{sa} =	10.17	kip		Nominal Shear Capacity
n_T =	11	-	Threads per inch (TPI)	φ =	0.65	-		
F_{ya} =	55	ksi	Yield Strength	φV_{sa} =	5.2884			
F_{uta} =	75	ksi	Ultimate Strength	D/C :	0.2701		OK	Shear Capacity Ratio
F_{uta} =	75	ksi	Min(F_{uta}, 1.9f_{ya}, 125)					
$A_{se,N}$ =	0.226	in^2	Effective Coss-Sectional Area	D/C :	0.9559		OK	Combined Capacity Ratio

Reference Table

Diam.	TPI
0.25	20
0.375	16
0.5	13
0.625	11
0.75	10
0.875	9
1	8
1.125	7
1.25	7
1.5	6

1.Project information

Project description:
Location:
Design name: Design

Comment:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB_H
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 18.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 20.13
C_{min} (inch): 3.75
S_{min} (inch): 3.75

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 13.78
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
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): 12.00 x 8.00 x 1.00
Yield stress: 36000 psi

Profile type/size: 4X4X3/16

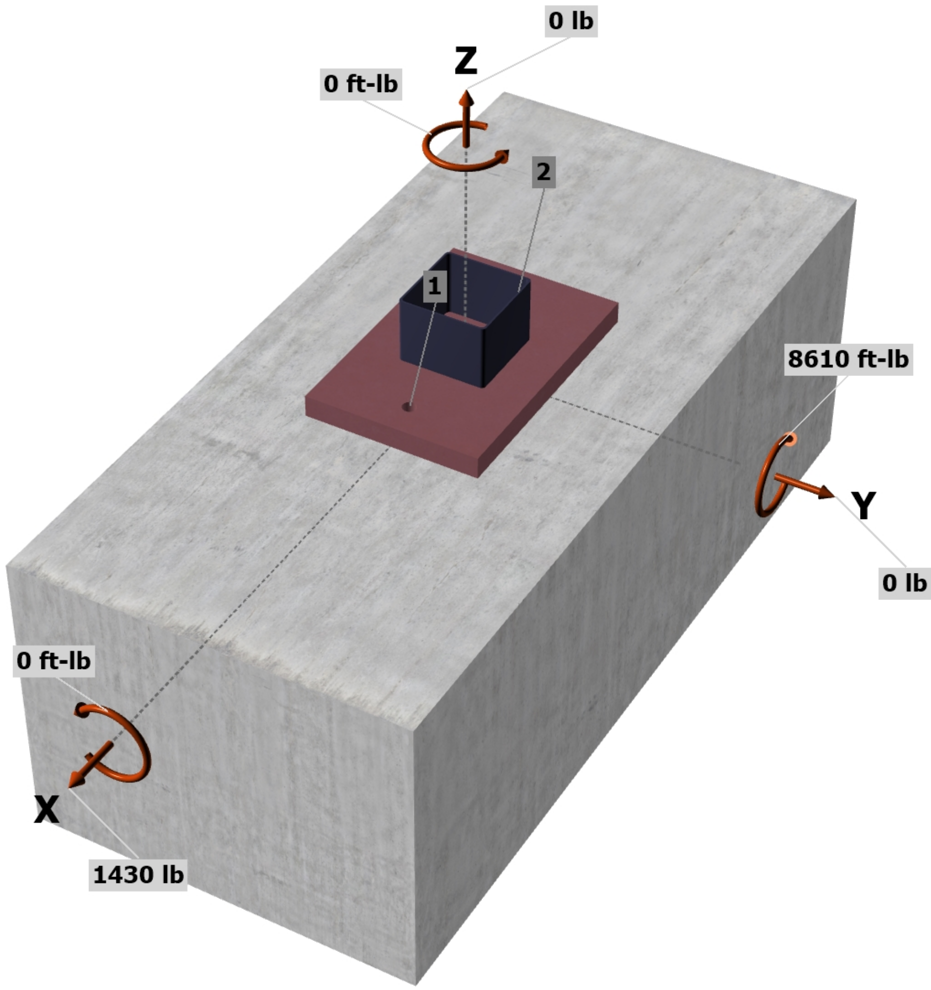
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB5H (5/8"Ø)

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

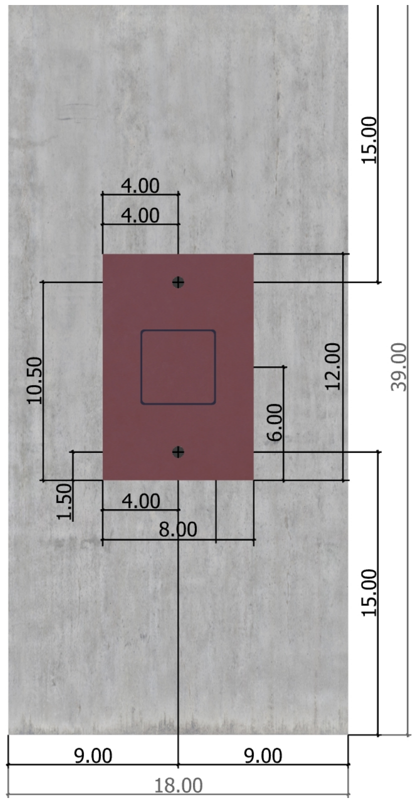
Strength level loads:

N_{ua} [lb]:	0
V_{uax} [lb]:	1430
V_{uay} [lb]:	0
M_{ux} [ft-lb]:	0
M_{uy} [ft-lb]:	-8610
M_{uz} [ft-lb]:	0

<Figure 1>



<Figure 2>

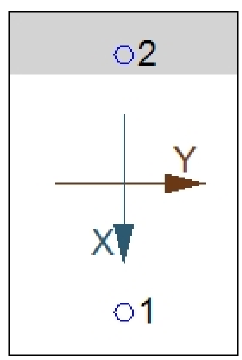


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	10573.7	715.0	0.0	715.0
2	0.0	715.0	0.0	715.0
Sum	10573.7	1430.0	0.0	1430.0

Maximum concrete compression strain (‰): 0.28
Maximum concrete compression stress (psi): 1209
Resultant tension force (lb): 10574
Resultant compression force (lb): 10574
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
27120	0.75	20340

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = 16\lambda_a\sqrt{f'_c}h_{ef}^{5/3}$ (Eq. 17.4.2.2b)

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	2500	16.000	81275

$\phi N_{cb} = \phi\,(A_{Nc} / A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
702.00	2304.00	9.00	0.813	1.00	1.000	81275	0.70	14084

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi\Psi_{c,P}N_p = \phi\Psi_{c,P}8A_{brg}f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	2.10	2500	0.70	29372

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
16270	1.0	0.65	10576

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in x-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. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
5.00	0.625	1.00	2500	16.00	26841

$\phi V_{cbx} = \phi\,(A_{Vc} / A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1 & Eq. 17.5.2.1a)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbx} (lb)
432.00	1152.00	0.813	1.000	1.000	26841	0.70	5725

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. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
5.00	0.625	1.00	2500	9.00	11324

$\phi V_{cbgy} = \phi\ (2)(A_{Vc} / A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

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)
486.00	364.50	1.000	1.000	1.000	1.000	11324	0.70	21138

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc} / A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

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	702.00	900.00	1.000	0.880	1.000	1.000	81275	0.70	35683

11. Results

Interaction of Tensile and Shear Forces (Sec. R17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	10574	20340	0.52	Pass
Concrete breakout	**10574**	**14084**	**0.75**	**Pass (Governs)**
Pullout	10574	29372	0.36	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status
Steel	715	10576	0.07	Pass
T Concrete breakout x+	**1430**	**5725**	**0.25**	**Pass (Governs)**
|| Concrete breakout y+	**1430**	**21138**	**0.07**	**Pass (Governs)**
Pryout	1430	35683	0.04	Pass

Interaction check	$(N_{ua}/\phi N_{ua})^{5/3}$	$(V_{ua}/\phi V_{ua})^{5/3}$	Utilization Ratio	Permissible	Status
Sec. R17.6	0.62	0.10	71.9%	1.0	Pass

PAB5H (5/8"Ø) with hef = 18.000 inch meets the selected design criteria.

12. Warnings

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