STRUCTURAL CALCULATIONS FOR

SF-60F PEDESTAL DESIGN
P-759 Base Main Gate and Entrance

Naval Air Station, North Island
San Diego, CA

REV. 2
10/27/2023

REGISTERED PROFESSIONAL ENGINEER
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Table of Contents

DESIGN CRITERIA

SH-60 Pedestal, NAS North Island

$H_{HELI} := 14\ in$ $H_{plate} := 1.5\ in$

WBDG Structural Load Data:

UNITED STATES - CALIFORNIA - NAS NORTH ISLAND, CORONADO

Latitude / Longitude 32.7015195, -117.2076912

WIND SPEED (MPH)					WIND SPEED (KM/H)				
RISK CATEGORY					RISK CATEGORY				
I	II	III	IV	V	I	II	III	IV	V
90	97	103	107	140	145	156	166	172	225

SNOW LOADING			
GROUND SNOW (PSF)	FROST PENETRATION (IN)	GROUND SNOW (KPA)	FROST PENETRATION (MM)
0	0	0	0

SEISMIC DATA (SITE CLASS D)				
PGA (%G)	S_S (%G)	S_1 (%G)	S_{MS} (%G)	S_{M1} (%G)
62.4	139.3	47.1	139.3	0

Wind Speed: 97 mph

Seismic Load: (Strengtn Level)

Risk Category II

Seismic Importance Factor $I_p := 1.0$

Site Class D

Seismic Design Category D

Minimum Analysis = Equivalent Lateral Force Procedure

Per UFC 3-301-01, *Structural Engineering*, 11 April 2023 & WBDG Structural Loads Tool

Latitude = 32.7015° N $S_s := 1.393$ $F_a := 1.0$

Longitude = -117.2077° E $S_1 := 0.471$ $F_v := 1.83$

$S_{MS} := S_s \cdot F_a$ $S_{MS} = 1.393$

$S_{M1} := S_1 \cdot F_v$ $S_{M1} = 0.862$

$S_{DS} := \frac{2}{3} \cdot S_{MS}$ $S_{DS} = 0.93$

$S_{D1} := \frac{2}{3} \cdot S_{M1}$ $S_{D1} = 0.575$

Seismic Force

Non-Structural Components:

Appendages and Ornamentations: $a_p := 2.5$ $R_p := 2.5$

Height of attachment: $z := 15.5 \cdot ft$ $h := 15.5 \cdot ft$

$$F_p := min\left(1.6 \cdot S_{DS} \cdot I_p, \max\left(\frac{0.4 \cdot a_p \cdot S_{DS}}{\left(\frac{R_p}{I_p}\right)} \cdot \left(1 + 2 \cdot \frac{z}{h}\right), 0.3 \cdot S_{DS} \cdot I_p\right)\right)$$

(Eq 13.3-2) **(Eq 13.3-1)** **(Eq 13.3-3)**

$F_p = 1.114$

Lateral seismic force is F_p time aircraft weight applied at aircraft center of gravity

$W := 12143 \cdot lbf = 12.143\ kip$ $DL_{SH60F} := W = 12.143\ kip$

$F_{pv} := 0.2 \cdot S_{DS} = 0.186$

$H_{HELI} = 14\ in$ $H_{plate} = 1.5\ in$

Concurent vertical force is +/- F_{pv} times aircraft weight applied at the aircraft center of gravity (shown in Government provided images below).

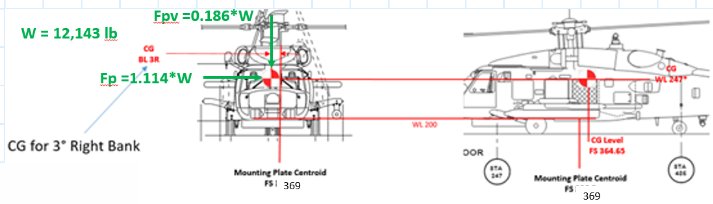
Assume 5deg pitch down configuration.

$F_p := 1.114 \cdot W = 13.527\ kip$ $H_v := 247\ in - 200\ in + H_{HELI} + H_{plate} = 62.5\ in$

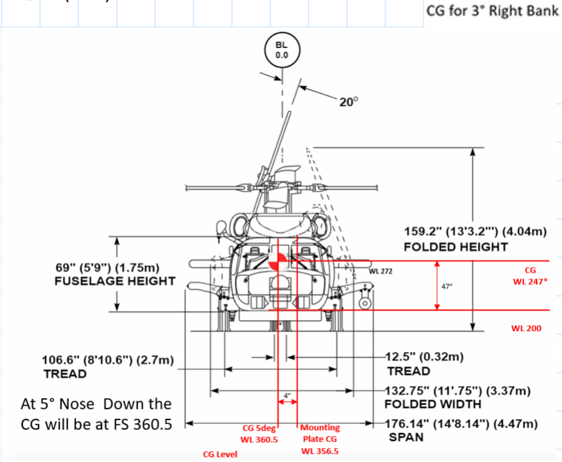
$F_{pv} := 0.2 \cdot S_{DS} \cdot W = 2.255\ kip$ $H_h := (362.1\ in - 359\ in) = 3.1\ in$

$F_p \cdot H_v + F_{pv} \cdot H_h = 71.037\ kip \cdot ft$

$M_z := F_p \cdot (H_v) = 70.455\ kip \cdot ft$



$M := F_p \cdot H_h = 3.495\ kip \cdot ft$



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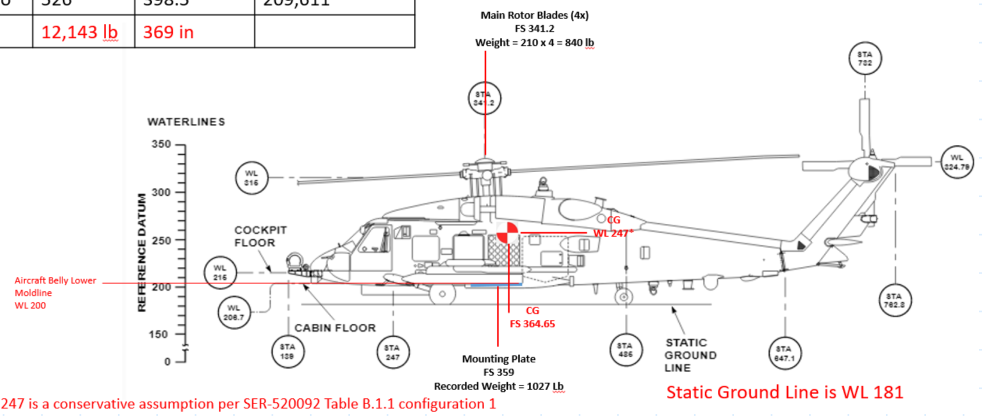
	Net Weight	Arm	Moment
MLG	6,100 lb	298.5 in	1,820,850 in-lb
TLG	3,650	482.9	1,762,585
Rotors	840	341.2	286,608
Plate	1,027	356.5	366,191
Torpedo	526	398.5	209,611
Total	12,143 lb	369 in	

SH-60F Aircraft with Main Rotors and Mounting Plate at Level



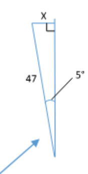
* CG WL 247 is a conservative assumption per SER-520092 Table B.1.1 configuration 1

SH-60F Aircraft with Main Rotors and Mounting Plate at Various Nose Down Pitch Angles

$$Sin\ 5 = \frac{X}{47}$$

X= 47(sin 5)

X = 4.09



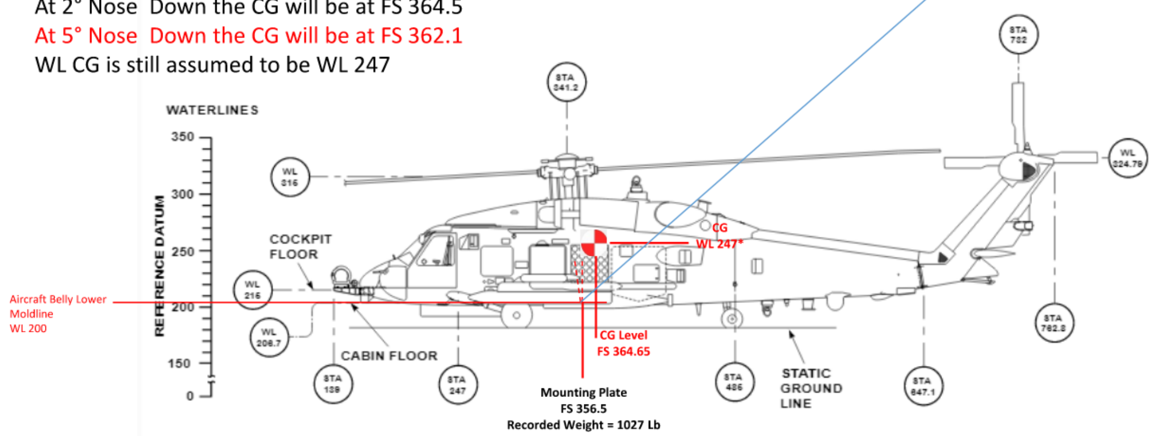
Each 1° tilt down will result in approximately 0.81-inch shift of CG in Fwd direction.

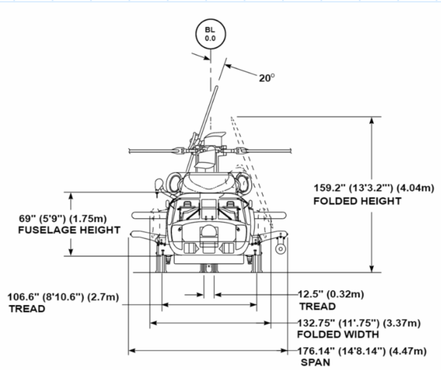
At 0° Nose Down the CG will be at FS 366.1

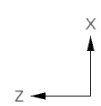
At 2° Nose Down the CG will be at FS 364.5

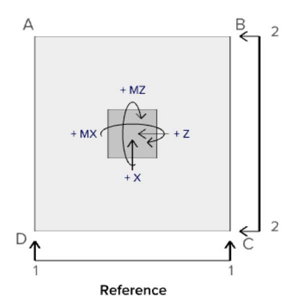
At 5° Nose Down the CG will be at FS 362.1

WL CG is still assumed to be WL 247









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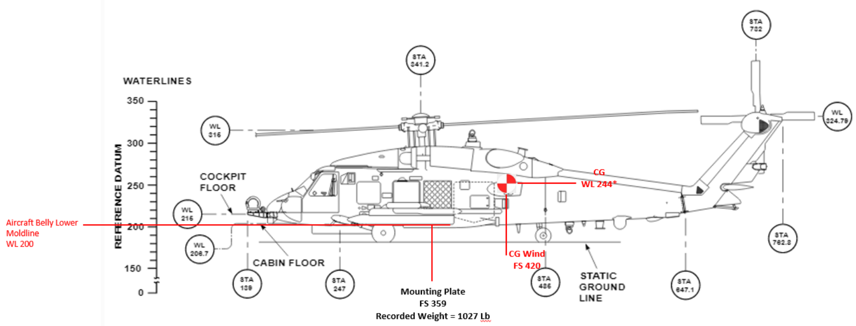
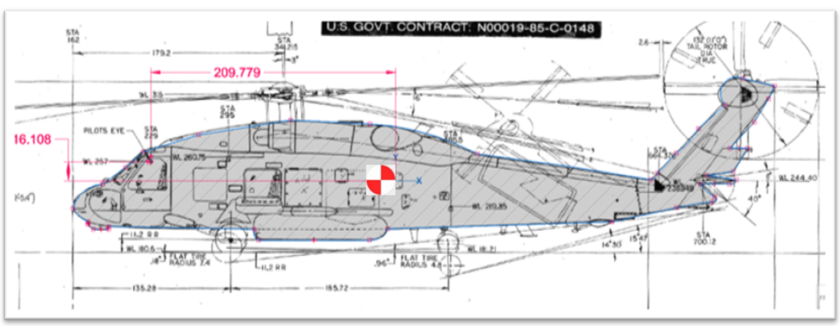
Wind Force

Loading Induced by Wind



From rough trace of SH-60F general assembly the measured center of area is
STA 420
WL 244
Total Area = 39,290 in^2

Loading Induced by Wind



From rough trace of SH-60F general assembly the measured center of area is
STA 439
WL 240
Total Area = 39,290 in^2

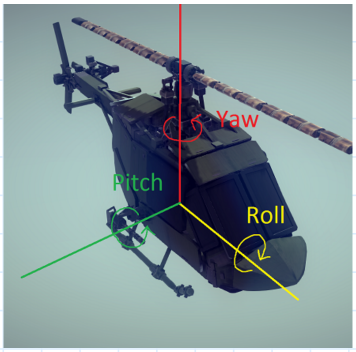
Estimated Induced Loading on aircraft based on 100 MPH wind = 10,500 lb

$P_{wind} := 10.5\ kip$ $\qquad H_{v\_wind} := (244\ in - 200\ in) + H_{HELI} + H_{plate} = 4.958\ ft$

$x := 360.5\ in - 359\ in = 1.5\ in$ $\qquad H_{h\_wind} := 420\ in - 359\ in = 61\ in$

$M_{wind\_x} := P_{wind} \cdot H_{v\_wind} = 52.063\ kip \cdot ft$ $\qquad M_{wind\_z} := P_{wind} \cdot x = 1.313\ kip \cdot ft$

$M_{wind\_torsion} := P_{wind} \cdot H_{h\_wind} = 53.375\ kip \cdot ft$



Loads on Anchor Bolts

Dead Load:

$P_D := W = 12.143\ kip$

Wind Load:

$M_{wind\_roll} := M_{wind\_x} = 52.063\ kip \cdot ft$ $M_{wind\_yaw} := M_{wind\_torsion} = 53.375\ kip \cdot ft$

$M_{wind\_pitch} := M_{wind\_z} = 1.313\ kip \cdot ft$ $P_{wind} = 10.5\ kip$

Seismic Load:

$F_{pv} = 2.255\ kip$ $F_p = 13.527\ kip$

$M_{seismic\_roll} := M_z = 70.455\ kip \cdot ft$ $M_{seismic\_yaw} := M = 3.495\ kip \cdot ft$

Load Combinations - Roll

LC 1: 1.4D	$1.4 \cdot P_D = 17\ kip$	
LC 2: 1.2D	$1.2 \cdot P_D = 14.572\ kip$	
LC 3: 1.2D+0.5W	$1.2 \cdot P_D = 14.572\ kip$	$0.5 \cdot M_{wind\_roll} = 26.031\ kip \cdot ft$
LC 4: 1.2D-0.5W	$1.2 \cdot P_D = 14.572\ kip$	$-0.5 \cdot M_{wind\_roll} = -26.031\ kip \cdot ft$
LC 5: 1.2D+W	$1.2 \cdot P_D = 14.572\ kip$	$M_{wind\_roll} = 52.063\ kip \cdot ft$
LC 6: 1.2D-W	$1.2 \cdot P_D = 14.572\ kip$	$-M_{wind\_roll} = -52.063\ kip \cdot ft$
LC 7: 0.9D+W	$0.9 \cdot P_D = 10.929\ kip$	$M_{wind\_roll} = 52.063\ kip \cdot ft$
LC 8: 0.9D-W	$0.9 \cdot P_D = 10.929\ kip$	$-M_{wind\_roll} = -52.063\ kip \cdot ft$
LC 9: 1.2D+E	$1.2 \cdot P_D + F_{pv} = 16.827\ kip$	$M_{seismic\_roll} = 70.455\ kip \cdot ft$
LC 10: 1.2D-E	$1.2 \cdot P_D - F_{pv} = 12.316\ kip$	$-M_{seismic\_roll} = -70.455\ kip \cdot ft$
LC 11: 0.9D+E	$0.9 \cdot P_D + F_{pv} = 13.184\ kip$	$M_{seismic\_roll} = 70.455\ kip \cdot ft$
LC 12: 0.9D-E	$0.9 \cdot P_D - F_{pv} = 8.673\ kip$	$-M_{seismic\_roll} = -70.455\ kip \cdot ft$

Governing Load Case: LC 9 = 1.2D+E (ROLL)

$P := 1.2 \cdot P_D + F_{pv} = 16.827\ kip$ $V := F_p = 13.527\ kip$

$M_{seismic\_roll} = 70.455\ kip \cdot ft$ $M_{seismic\_yaw} := M = 3.495\ kip \cdot ft$

Load Combinations - Yaw

LC 1: 1.4D	$1.4 \cdot P_D = 17\ kip$	
LC 2: 1.2D	$1.2 \cdot P_D = 14.572\ kip$	
LC 3: 1.2D+0.5W	$1.2 \cdot P_D = 14.572\ kip$	$0.5 \cdot M_{wind\_yaw} = 26.688\ kip \cdot ft$
LC 4: 1.2D-0.5W	$1.2 \cdot P_D = 14.572\ kip$	$-0.5 \cdot M_{wind\_yaw} = -26.688\ kip \cdot ft$
LC 5: 1.2D+W	$1.2 \cdot P_D = 14.572\ kip$	$M_{wind\_yaw} = 53.375\ kip \cdot ft$
LC 6: 1.2D-W	$1.2 \cdot P_D = 14.572\ kip$	$-M_{wind\_yaw} = -53.375\ kip \cdot ft$
LC 7: 0.9D+W	$0.9 \cdot P_D = 10.929\ kip$	$M_{wind\_yaw} = 53.375\ kip \cdot ft$
LC 8: 0.9D-W	$0.9 \cdot P_D = 10.929\ kip$	$-M_{wind\_yaw} = -53.375\ kip \cdot ft$
LC 9: 1.2D+E	$1.2 \cdot P_D + F_{pv} = 16.827\ kip$	$M_{seismic\_yaw} = 3.495\ kip \cdot ft$
LC 10: 1.2D-E	$1.2 \cdot P_D - F_{pv} = 12.316\ kip$	$-M_{seismic\_yaw} = -3.495\ kip \cdot ft$
LC 11: 0.9D+E	$0.9 \cdot P_D + F_{pv} = 13.184\ kip$	$M_{seismic\_yaw} = 3.495\ kip \cdot ft$
LC 12: 0.9D-E	$0.9 \cdot P_D - F_{pv} = 8.673\ kip$	$-M_{seismic\_yaw} = -3.495\ kip \cdot ft$

Governing Load LC 5: 1.2D+W

$1.2 \cdot P_D = 14.572\ kip$ $\quad P_{wind} = 10.5\ kip$

$M_{wind\_roll} = 52.063\ kip \cdot ft$ $\quad M_{wind\_pitch} = 1.313\ kip \cdot ft$

$M_{wind\_yaw} = 53.375\ kip \cdot ft$

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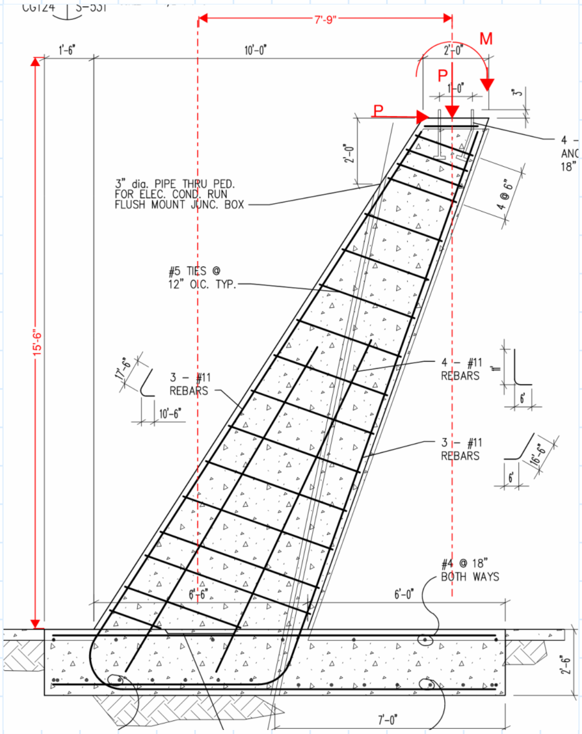
Pedestal Loading:



$H_{ped} := 15.5\ ft$

$x := 7.75\ ft$

Dead Load:

$P_D = 12.143\ kip$

$M := P_D \cdot x = 94.108\ kip \cdot ft$

Seismic Load:

$F_p = 13.527\ kip$ $\quad$ $H_v := 247\ in - 200\ in + H_{ped} + 15.5\ in = 20.7083\ ft$

$F_{pv} = 2.255\ kip$ $\quad$ $H_h := 362.1\ in - 359\ in = 3.1\ in$

$M_x := F_p \cdot (H_v) = 280.128\ kip \cdot ft$ $\quad$ $M := F_p \cdot H_h = 3.495\ kip \cdot ft$

Wind Load:

$P_{wind} = 10.5\ kip$ $\quad$ $H_{v\_wind} := 240\ in - 200\ in + H_{ped} + 15.5\ in = 20.125\ ft$

$M_{wind} := P_{wind} \cdot H_{v\_wind} = 211.313\ kip \cdot ft$ $\quad$ $M_{wind\_torsion} = 53.375\ kip \cdot ft$

SPREAD FOOTING & PEDESTAL DESIGN

PROPERTIES

GEOMETRY

FOOTING

Length Design Method	Explicit	**Design Length**	14 ft	**Width Design Method**	Explicit
Design Width	12 ft	**Force Square**	No	**Thickness Design Method**	Explicit
Design Thickness	30 in				

PEDESTAL

Shape	Rectangular	**Length**	24 in	**Width**	18 in
Height	186 in	**eX**	0 in	**eZ**	27 in

MATERIAL PROPERTIES

Design Code	ACI 318-19	**Material Type**	3000 NW	**Concrete Weight**	0.145 k/ft^3
Concrete f'c	3 ksi	**Check Concrete Bearing**	Yes	**Steel Fy**	60 ksi

SOIL PROPERTIES

Passive Sliding Resistance	0 k	**Coefficient of Friction**	0.3	**Allowable Bearing**	1.337 ksf
Gross/Net	Gross				

REINFORCEMENT PROPERTIES

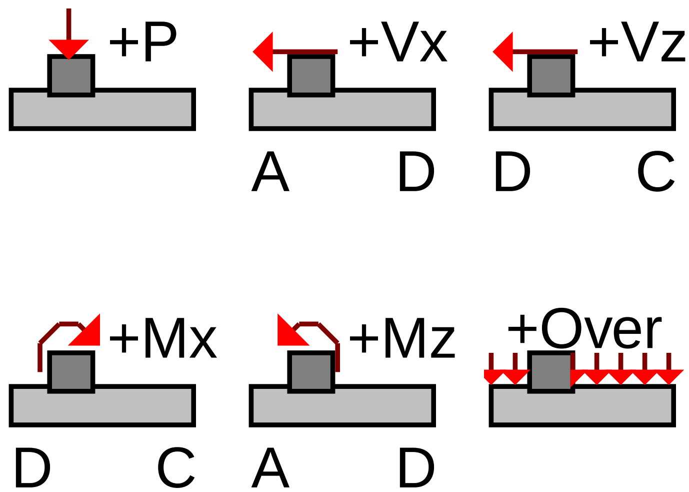
Footing Top Cover	1.5 in	**Footing Bottom Cover**	3 in	**Footing Design**	Explicit
Top Bar Size	#4	**Top Bar Quantity (X dir.)**	14	**Top Bar Quantity (Z dir.)**	12
Bottom Bar Size	#7	**Bottom Bar Quantity (X dir.)**	16	**Bottom Bar Quantity (Z dir.)**	12
Force Top Bars?	Yes	**Pedestal Side Cover**	1.5 in	**Pedestal Bar Size**	#11
Pedestal Shear Ties	#5				

DESIGN PROPERTIES

Overturning/Sliding SF	Varies	**Φ for Flexure**	0.9	**Φ for Shear**	0.75
Φ for Bearing	0.65				

LOADS

Load Category	Magnitude (k, k-ft)	Direction
DL	12.143	P
EL	2.255	P
EL	262.655	Mz
WL	197.75	Mz
Overburden (ksf)	0	



LOAD COMBINATIONS

Load combinations include self weight.

STRENGTH

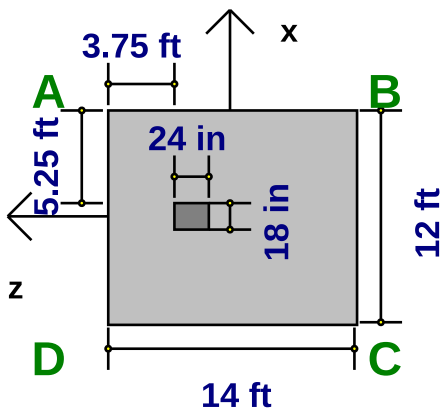
Equation	Code	ABIF	SF
1. 1.4 DL	IBC 21/ASCE Strength 1	1	1
2. 1.2 DL + 1.6 HL	IBC 21/ASCE Strength 2 (a)	1	1
3. 1.2 DL + 1.6 HL + 0.5 WL	IBC 21/ASCE Strength 3 (b)	1	1
4. 1.2 DL + 1 WL + 1.6 HL	IBC 21/ASCE Strength 4 (a)	1	1
5. 0.9 DL + 1 WL + 1.6 HL	IBC 21/ASCE Strength 5 (a)	1	1
6. 0.9 DL + 1 WL + 0.9 HL	IBC 21/ASCE Strength 5 (b)	1	1
7. 1.2 DL + 1 EL + 1.6 HL	IBC 21/ASCE Strength 6	1	1
8. 0.9 DL + 1 EL + 1.6 HL	IBC 21/ASCE Strength 7 (a)	1	1
9. 0.9 DL + 1 EL + 0.9 HL	IBC 21/ASCE Strength 7 (b)	1	1

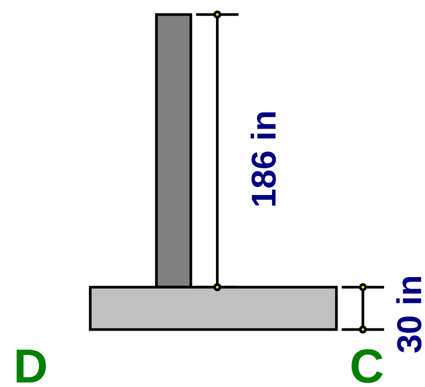
ASD

Equation	Code	ABIF	SF
10. 1 DL	IBC 21/ASCE 1	1	1.5
11. 1 DL + 1 HL	IBC 21/ASCE 2	1	1.5
12. 1 DL + 1 HL + 0.6 WL	IBC 21/ASCE 5 (a)	1	1.5
13. 1 DL + 1 HL + 0.45 WL	IBC 21/ASCE 6 (a)	1	1.5
14. 0.6 DL + 1 HL + 0.6 WL	IBC 21/ASCE 7 (a)	1	1
15. 0.6 DL + 0.6 HL + 0.6 WL	IBC 21/ASCE 7 (b)	1	1
16. 1 DL + 1 HL + 0.7 EL	IBC 21/ASCE 8	1	1.5
17. 1 DL + 1 HL + 0.525 EL	IBC 21/ASCE 9	1	1.5
18. 0.6 DL + 1 HL + 0.7 EL	IBC 21/ASCE 10 (a)	1	1
19. 0.6 DL + 0.6 HL + 0.7 EL	IBC 21/ASCE 10 (b)	1	1

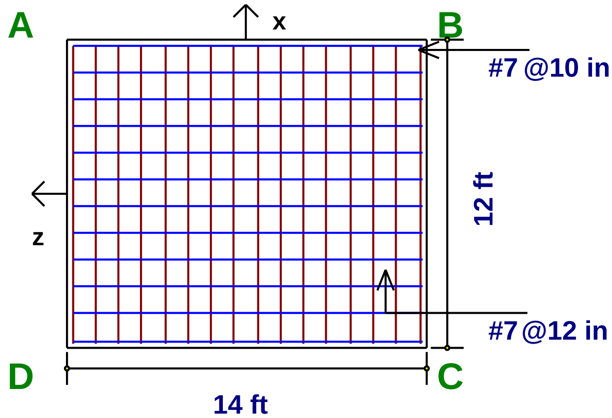
DIAGRAMS

SKETCH





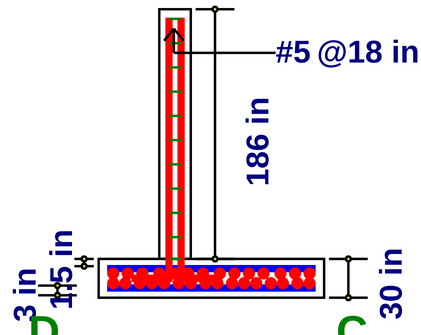
DETAILS



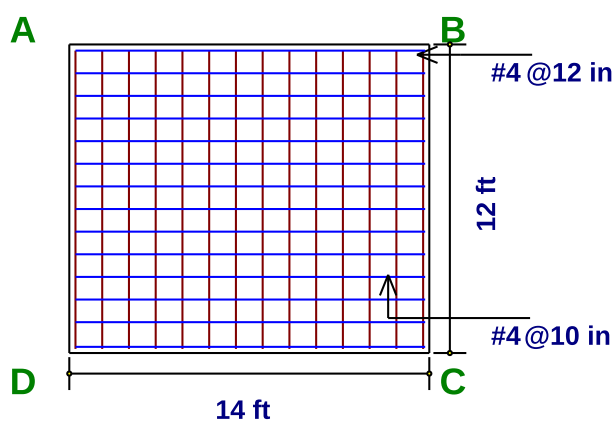
x Dir. Steel: 9.62 in^2 (16 #7)

z Dir. Steel: 7.22 in^2 (12 #7)

Bottom Rebar Plan



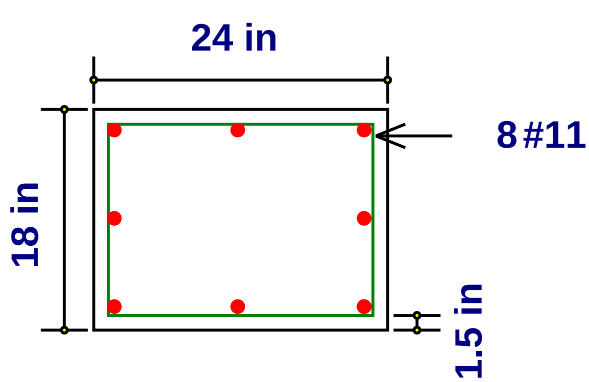
Footing Elevation



x Dir. Steel: 2.75 in^2 (14 #4)

z Dir. Steel: 2.75 in^2 (14 #4)

Top Rebar Plan



Pedestal Rebar Plan

CALCULATIONS

Limit State	Required	Available	Unity Check	Result
Footing Flexure Design (Bottom Bars)	148.851 k-ft	1101.914 k-ft	0.135	PASS
Footing Flexure Design (Top Bars)	69.66 k-ft	-	-	
Footing Shear Check	35.013 k	187.369 k	0.187	PASS
Pedestal Design Shear	0 k	65.235 k	0	PASS
Pedestal Design Bending	-	-	0.731	PASS
Concrete Bearing Check (Vertical Loads Only)	26.44 k	1432.08 k	0.018	PASS
Overturning Check (Service)	183.858 k-ft	296.699 k-ft	1.614	PASS
Sliding Check (Service)	0 k	14.361 k	N/A	PASS
Soil Bearing	1.169ksf	1.337ksf	0.874	PASS

Footing Flexure Design (Bottom Bars)

As-min x-dir (Top Flexure)	0 in^2	**As-min z-dir (Top Flexure)**	0 in^2	**As-min x-dir (T & S)**	9.072 in^2
As-min z-dir (T & S)	7.776 in^2	**As-min x-dir (Bot Flexure)**	9.072 in^2	**As-min z-dir (Bot Flexure)**	7.776 in^2

Description	Categories and Factors	Mu-xx UC Max	Mu-xx (k-ft)	z-Dir As Required (in^2)	z-Dir As Provided (in^2)	Mu-zz UC Max	Mu-zz (k-ft)	x-Dir As Required (in^2)	x-Dir As Provided (in^2)
IBC 21/ASCE..	1.4DL	0.03213	26.64	0.227	7.216	0.02756	30.36	0.258	9.621
IBC 21/ASCE..	1.2DL+1.6HL	0.02754	22.84	0.194	7.216	0.02362	26.03	0.222	9.621
IBC 21/ASCE..	1.2DL+1.6HL+ 0.5WL	0.02754	22.84	0.194	7.216	0.06012	66.24	0.564	9.621
IBC 21/ASCE..	1.2DL+1WL+1.6 HL	0.02782	23.07	0.196	7.216	0.09699	106.88	0.911	9.621
IBC 21/ASCE..	0.9DL+1WL+1. 6HL	0.02097	17.38	0.148	7.216	0.09454	104.17	0.888	9.621
IBC 21/ASCE..	0.9DL+1WL+0. 9HL	0.02097	17.38	0.148	7.216	0.09454	104.17	0.888	9.621
IBC 21/ASCE..	1.2DL+1EL+1.6 HL	0.03077	25.51	0.217	7.216	0.12729	140.26	1.197	9.621
IBC 21/ASCE..	0.9DL+1EL+1.6 HL	0.02378	19.72	0.168	7.216	0.13508	148.85	1.27	9.621
IBC 21/ASCE..	0.9DL+1EL+0. 9HL	0.02378	19.72	0.168	7.216	0.13508	148.85	1.27	9.621

Footing Flexure Design (Top Bars)

Description	Categories and Factors	Mu-xx (k-ft)	z Dir As (in^2)	Mu-zz (k-ft)	x Dir As (in^2)
SW+OB	1SW+1OB-(IBC 21/A..,IBC 21/A..)	0	0	69.66	0.554

Moment Capacity of Plain Concrete Section Along xx and zz= 257.649k-ft,300.59k-ft Per Chapter 22 of ACI 318.

Footing Shear Check

Two Way (Punching) Vc: 802.785 k **One Way (x Dir. Cut) Vc:** 204.814 k **One Way (z Dir. Cut) Vc:** 249.825 k

Description	Categories and Factors	Punching Vu(k)	Punching Vu/(φVc)	x Dir. Cut Vu(k)	x Dir. Cut Vu/(φVc)	z Dir. Cut Vu(k)	z Dir. Cut Vu/(φVc)
IBC 21/ASCE S..	1.4DL	23.273	0.039	5.513	0.036	6.771	0.036
IBC 21/ASCE S..	1.2DL+1.6HL	19.948	0.033	4.726	0.031	5.803	0.031
IBC 21/ASCE S..	1.2DL+1.6HL+0.5WL	19.948	0.033	4.726	0.031	15.221	0.081
IBC 21/ASCE S..	1.2DL+1WL+1.6HL	19.959	0.033	4.761	0.031	24.741	0.132
IBC 21/ASCE S..	0.9DL+1WL+1.6HL	15.259	0.025	3.593	0.023	24.234	0.129
IBC 21/ASCE S..	0.9DL+1WL+0.9HL	15.259	0.025	3.593	0.023	24.234	0.129
IBC 21/ASCE S..	1.2DL+1EL+1.6HL	22.245	0.037	5.272	0.034	32.591	0.174
IBC 21/ASCE S..	0.9DL+1EL+1.6HL	19.414	0.032	4.075	0.027	35.013	0.187
IBC 21/ASCE S..	0.9DL+1EL+0.9HL	19.414	0.032	4.075	0.027	35.013	0.187

Pedestal Design Shear

	Vc (k)	Vs (k)	Vu (k)	Vu/Φ*Vn	Φ	Gov LC
Shear Along x Direction:	41.735	31.027	0	0	0.75	1
Shear Along z Direction:	43.681	43.299	0	0	0.75	1

Pedestal Ties #5 @ 18 in

Pedestal Design Bending

Unity Check:		**0.731**		**Pu**	13.184 k
Pn	20.047 k	**Governing LC**	8	**Pedestal Bars**	8 #11
Φ	0.9	**Mux**	0 k-ft	**Mnx**	NC
Mnox	NC	**% Steel**	2.8916	**Parme β**	0.65
Muz	262.655 k-ft	**Mnz**	399.392 k-ft	**Mnoz**	NC

Concrete Bearing Check (Vertical Loads Only)

Bearing Bc : 2203.2 k

Description	Categories and Factors	Bearing Bu (k)	Bearing Bu/(Φ Bc)
IBC 21/ASCE S..	1.4DL	26.44	0.018
IBC 21/ASCE S..	1.2DL+1.6HL	22.663	0.016
IBC 21/ASCE S..	1.2DL+1.6HL+0.5 WL	22.663	0.016
IBC 21/ASCE S..	1.2DL+1WL+1.6HL	22.663	0.016
IBC 21/ASCE S..	0.9DL+1WL+1.6H L	16.997	0.012
IBC 21/ASCE S..	0.9DL+1WL+0.9H L	16.997	0.012
IBC 21/ASCE S..	1.2DL+1EL+1.6HL	24.918	0.017
IBC 21/ASCE S..	0.9DL+1EL+1.6HL	19.252	0.013
IBC 21/ASCE S..	0.9DL+1EL+0.9HL	19.252	0.013

Overturning Check (Service)

Description	Categories and Factors	Mo-xx (k-ft)	Ms-xx (k-ft)	Mo-zz (k-ft)	Ms-zz (k-ft)	OSF-xx	OSF-zz
IBC 21/ASCE 1	1DL	0	516.006	0	478.713	NA	NA
IBC 21/ASCE 2	1DL+1HL	0	516.006	0	478.713	NA	NA
IBC 21/ASCE 5..	1DL+1HL+0.6WL	0	516.006	118.65	478.713	NA	4.035
IBC 21/ASCE 6..	1DL+1HL+0.45WL	0	516.006	88.987	478.713	NA	5.38
IBC 21/ASCE 7..	0.6DL+1HL+0.6W L	0	309.604	118.65	287.228	NA	2.421
IBC 21/ASCE 7..	0.6DL+0.6HL+0.6 WL	0	309.604	118.65	287.228	NA	2.421
IBC 21/ASCE 8	1DL+1HL+0.7EL	0	523.504	183.858	488.184	NA	2.655

IBC 21/ASCE 9	1DL+1HL+0.525EL	0	521.63	137.894	485.816	NA	3.523
IBC 21/ASCE 1..	0.6DL+1HL+0.7EL	0	317.102	183.858	296.699	NA	1.614
IBC 21/ASCE 1..	0.6DL+0.6HL+0.7EL	0	317.102	183.858	296.699	NA	1.614

Mo-xx: Governing Overturning Moment about AD or BC

Ms-xx: Governing Stablizing Moment about AD or BC

OSF-xx: Ratio of Ms-xx to Mo-xx

Sliding Check (Service)

Description	Categories and Factors	Va-xx (k)	Vr-xx (k)	Va-zz (k)	Vr-zz (k)	SR-xx	SR-zz
IBC 21/ASCE 1	1DL	0	23.936	0	23.936	NA	NA
IBC 21/ASCE 2	1DL+1HL	0	23.936	0	23.936	NA	NA
IBC 21/ASCE 5..	1DL+1HL+0.6WL	0	23.936	0	23.936	NA	NA
IBC 21/ASCE 6..	1DL+1HL+0.45WL	0	23.936	0	23.936	NA	NA
IBC 21/ASCE 7..	0.6DL+1HL+0.6WL	0	14.361	0	14.361	NA	NA
IBC 21/ASCE 7..	0.6DL+0.6HL+0.6WL	0	14.361	0	14.361	NA	NA
IBC 21/ASCE 8	1DL+1HL+0.7EL	0	24.409	0	24.409	NA	NA
IBC 21/ASCE 9	1DL+1HL+0.525EL	0	24.291	0	24.291	NA	NA
IBC 21/ASCE 1..	0.6DL+1HL+0.7EL	0	14.835	0	14.835	NA	NA
IBC 21/ASCE 1..	0.6DL+0.6HL+0.7EL	0	14.835	0	14.835	NA	NA

Va-xx: Applied Lateral Force to Cause Sliding Along xx Axis

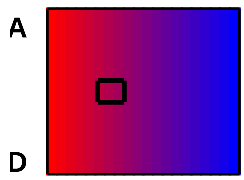
Vr-xx: Resisting Lateral Force Against Sliding Along xx Axis

SR-xx: Ratio of Vr-xx to Va-xx

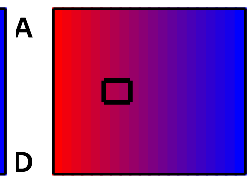
Soil Bearing

Description	Categories and Factors	Gross Allow.(ksf)	Max Bearing (ksf)	Max/Allowable Ratio
IBC 21/ASCE 1	1DL	1.337	0.583 (A)	0.436
IBC 21/ASCE 2	1DL+1HL	1.337	0.583 (A)	0.436

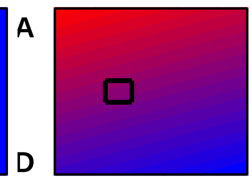
IBC 21/ASCE 5..	1DL+1HL+0.6WL	1.337	0.936 (A)	0.7
IBC 21/ASCE 6..	1DL+1HL+0.45WL	1.337	0.848 (A)	0.634
IBC 21/ASCE 7..	0.6DL+1HL+0.6WL	1.337	0.725 (A)	0.542
IBC 21/ASCE 7..	0.6DL+0.6HL+0.6WL	1.337	0.725 (A)	0.542
IBC 21/ASCE 8	1DL+1HL+0.7EL	1.337	1.168 (A)	0.873
IBC 21/ASCE 9	1DL+1HL+0.525EL	1.337	1.008 (A)	0.754
IBC 21/ASCE 1..	0.6DL+1HL+0.7EL	1.337	1.169 (A)	0.874
IBC 21/ASCE 1..	0.6DL+0.6HL+0.7EL	1.337	1.169 (A)	0.874



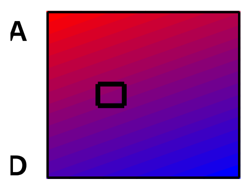
1DL
QA: 0.583 ksf
QB: 0.367 ksf
QC: 0.367 ksf
QD: 0.583 ksf
NAZ: 452.018 in



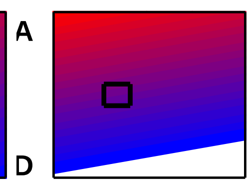
1DL+1HL
QA: 0.583 ksf
QB: 0.367 ksf
QC: 0.367 ksf
QD: 0.583 ksf
NAZ: 452.018 in



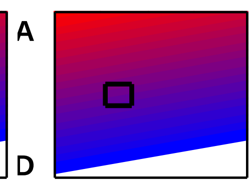
1DL+1HL+0.6WL
QA: 0.936 ksf
QB: 0.72 ksf
QC: 0.013 ksf
QD: 0.23 ksf
NAZ: 725.66 in



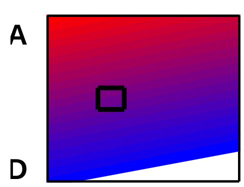
1DL+1HL+0.45WL
QA: 0.848 ksf
QB: 0.631 ksf
QC: 0.102 ksf
QD: 0.318 ksf
NAZ: 657.25 in



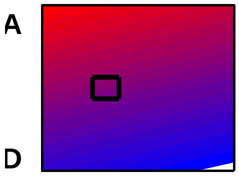
0.6DL+1HL+0.6W
QA: 0.725 ksf
QB: 0.576 ksf
QC: 0 ksf
QD: 0 ksf
NAZ: 816.826 in

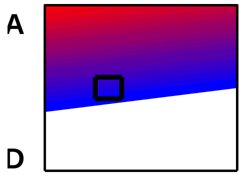


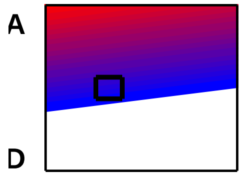
0.6DL+0.6HL+0..
QA: 0.725 ksf
QB: 0.576 ksf
QC: 0 ksf
QD: 0 ksf
NAZ: 816.826 in



1DL+1HL+0.7EL
QA: 1.168 ksf
QB: 0.914 ksf
QC: 0 ksf
QD: 0.044 ksf
NAZ: 772.554 in







	1DL+1HL+0.525E		0.6DL+1HL+0.7E		0.6DL+0.6HL+0..
QA:	1.008 ksf	QA:	1.169 ksf	QA:	1.169 ksf
QB:	0.777 ksf	QB:	0.906 ksf	QB:	0.906 ksf
QC:	0 ksf	QC:	0 ksf	QC:	0 ksf
QD:	0.187 ksf	QD:	0 ksf	QD:	0 ksf
NAZ:	733.661 in	NAZ:	747.799 in	NAZ:	747.799 in

General Footing

DESCRIPTION: Pedestal Footing_Final Design

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0	ksi
fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,155.92	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.750	

Analysis Settings

Min Steel % Bending Reinf.	=		
Min Allow % Temp Reinf.	=	0.00180	
Min. Overturning Safety Factor	=	1.0	: 1
Min. Sliding Safety Factor	=	1.0	: 1
Add Ftg Wt for Soil Pressure	:	Yes	
Use ftg wt for stability, moments & shears	:	Yes	
Add Pedestal Wt for Soil Pressure	:	No	
Use Pedestal wt for stability, mom & shear	:	No	

Soil Design Values

Allowable Soil Bearing	=	1.50	ksf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	200.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

Footing base depth below soil surface	=	2.0	ft
Allow press. increase per foot of depth	=		ksf
when footing base is below	=		ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=		ksf
when max. length or width is greater than	=		ft

Dimensions

Width parallel to X-X Axis	=	14.0	ft
Length parallel to Z-Z Axis	=	12.0	ft
Footing Thickness	=	30.0	in

Load location offset from footing center...

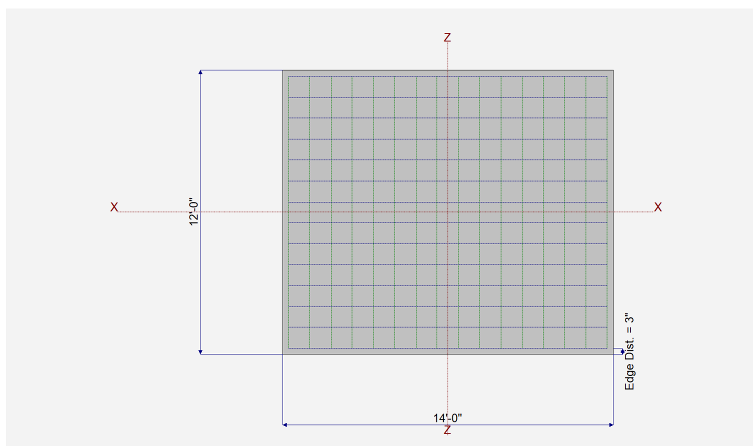
ex : Prll to X-X Axis	=	-27	in
	=		in

Pedestal dimensions...

px : parallel to X-X Axis	=	78.0	in
pz : parallel to Z-Z Axis	=	34.0	in
Height	=		in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0	in

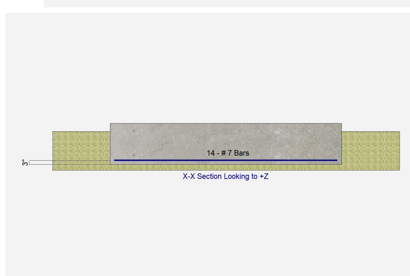


Reinforcing

Bars parallel to X-X Axis		
Number of Bars	=	14.0
Reinforcing Bar Size	=	# 7
Bars parallel to Z-Z Axis		
Number of Bars	=	16.0
Reinforcing Bar Size	=	# 7

Bandwidth Distribution Check (ACI 15.4.4.2)
Direction Requiring Closer Separation

	Bars along Z-Z Axis
# Bars required within zone	92.3 %
# Bars required on each side of zone	7.7 %



Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	12.143					2.255		k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=	94.108				197.750	262.655		k-ft
V-x	=								k
V-z	=								k

General Footing

DESCRIPTION: Pedestal Footing_Final Design

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.7747	Soil Bearing	1.162 ksf	1.50 ksf	+0.60D+0.70E about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	1.406	Overturning - Z-Z	240.323 k-ft	337.775 k-ft	+0.60D+0.70E
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.1806	Z Flexure (+X)	14.970 k-ft/ft	82.888 k-ft/ft	+0.90D+E
PASS	0.00590	Z Flexure (-X)	0.4890 k-ft/ft	82.888 k-ft/ft	+1.20D+W
PASS	0.01308	X Flexure (+Z)	1.063 k-ft/ft	81.240 k-ft/ft	+1.40D
PASS	0.01308	X Flexure (-Z)	1.063 k-ft/ft	81.240 k-ft/ft	+1.40D
PASS	0.1217	1-way Shear (+X)	10.002 psi	82.158 psi	+0.90D+E
PASS	0.0	1-way Shear (-X)	0.0 psi	0.0 psi	n/a
PASS	0.008667	1-way Shear (+Z)	0.7121 psi	82.158 psi	+1.40D
PASS	0.008667	1-way Shear (-Z)	0.7121 psi	82.158 psi	+1.40D
PASS	n/a	2-way Punching	2.239 psi	82.158 psi	+0.90D+E

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location: Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	1.50	n/a	0.0	0.4348	0.4348	n/a	n/a	0.290
X-X, +D+0.60W	1.50	n/a	0.0	0.4348	0.4348	n/a	n/a	0.290
X-X, +D-0.60W	1.50	n/a	0.0	0.4348	0.4348	n/a	n/a	0.290
X-X, +D+0.70E	1.50	n/a	0.0	0.4442	0.4442	n/a	n/a	0.296
X-X, +D-0.70E	1.50	n/a	0.0	0.4254	0.4254	n/a	n/a	0.284
X-X, +D+0.450W	1.50	n/a	0.0	0.4348	0.4348	n/a	n/a	0.290
X-X, +D-0.450W	1.50	n/a	0.0	0.4348	0.4348	n/a	n/a	0.290
X-X, +D+0.5250E	1.50	n/a	0.0	0.4418	0.4418	n/a	n/a	0.295
X-X, +D-0.5250E	1.50	n/a	0.0	0.4277	0.4277	n/a	n/a	0.285
X-X, +0.60D+0.60W	1.50	n/a	0.0	0.2609	0.2609	n/a	n/a	0.174
X-X, +0.60D-0.60W	1.50	n/a	0.0	0.2609	0.2609	n/a	n/a	0.174
X-X, +0.60D+0.70E	1.50	n/a	0.0	0.2703	0.2703	n/a	n/a	0.180
X-X, +0.60D-0.70E	1.50	n/a	0.0	0.2515	0.2515	n/a	n/a	0.168
Z-Z, D Only	1.50	10.972	n/a	n/a	n/a	0.2661	0.6034	0.402
Z-Z, +D+0.60W	1.50	30.465	n/a	n/a	n/a	0.0	0.9048	0.603
Z-Z, +D-0.60W	1.50	-8.521	n/a	n/a	n/a	0.5658	0.3038	0.377
Z-Z, +D+0.70E	1.50	39.735	n/a	n/a	n/a	0.0	1.117	0.745
Z-Z, +D-0.70E	1.50	-19.062	n/a	n/a	n/a	0.7121	0.1387	0.475
Z-Z, +D+0.450W	1.50	25.592	n/a	n/a	n/a	0.04137	0.8282	0.552
Z-Z, +D-0.450W	1.50	-3.647	n/a	n/a	n/a	0.4908	0.3787	0.327
Z-Z, +D+0.5250E	1.50	32.659	n/a	n/a	n/a	0.0	0.9586	0.639
Z-Z, +D-0.5250E	1.50	-11.430	n/a	n/a	n/a	0.6006	0.2549	0.400
Z-Z, +0.60D+0.60W	1.50	43.460	n/a	n/a	n/a	0.0	0.7157	0.477
Z-Z, +0.60D-0.60W	1.50	-21.516	n/a	n/a	n/a	0.4593	0.06242	0.306
Z-Z, +0.60D+0.70E	1.50	58.244	n/a	n/a	n/a	0.0	1.162	0.775
Z-Z, +0.60D-0.70E	1.50	-39.833	n/a	n/a	n/a	0.6336	0.0	0.422

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
X-X, D Only	None	0.0 k-ft	Infinity	OK
X-X, +D+0.60W	None	0.0 k-ft	Infinity	OK
X-X, +D+0.70E	None	0.0 k-ft	Infinity	OK
X-X, +D+0.450W	None	0.0 k-ft	Infinity	OK
X-X, +D+0.5250E	None	0.0 k-ft	Infinity	OK
X-X, +0.60D+0.60W	None	0.0 k-ft	Infinity	OK
X-X, +0.60D+0.70E	None	0.0 k-ft	Infinity	OK
Z-Z, D Only	94.108 k-ft	538.62 k-ft	5.723	OK
Z-Z, +D+0.60W	212.758 k-ft	538.62 k-ft	2.532	OK
Z-Z, +D+0.70E	277.967 k-ft	553.22 k-ft	1.990	OK

General Footing

DESCRIPTION: Pedestal Footing_Final Design

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Z-Z, +D+0.450W	183.096 k-ft	538.62 k-ft	2.942	OK
Z-Z, +D+0.5250E	232.002 k-ft	549.57 k-ft	2.369	OK
Z-Z, +0.60D+0.60W	175.115 k-ft	323.174 k-ft	1.846	OK
Z-Z, +0.60D+0.70E	240.323 k-ft	337.775 k-ft	1.406	OK

All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status

Footing Has NO Sliding

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	1.063	+Z	Bottom	0.6480	Min Temp %	0.6857	81.240	OK
X-X, +1.40D	1.063	-Z	Bottom	0.6480	Min Temp %	0.6857	81.240	OK
X-X, +1.20D	0.9110	+Z	Bottom	0.6480	Min Temp %	0.6857	81.240	OK
X-X, +1.20D	0.9110	-Z	Bottom	0.6480	Min Temp %	0.6857	81.240	OK
X-X, +1.20D+0.50W	0.9110	+Z	Bottom	0.6480	Min Temp %	0.6857	81.240	OK
X-X, +1.20D+0.50W	0.9110	-Z	Bottom	0.6480	Min Temp %	0.6857	81.240	OK
X-X, +1.20D+W	0.9110	+Z	Bottom	0.6480	Min Temp %	0.6857	81.240	OK
X-X, +1.20D+W	0.9110	-Z	Bottom	0.6480	Min Temp %	0.6857	81.240	OK
X-X, +1.20D+E	1.052	+Z	Bottom	0.6480	Min Temp %	0.6857	81.240	OK
X-X, +1.20D+E	1.052	-Z	Bottom	0.6480	Min Temp %	0.6857	81.240	OK
X-X, +0.90D+W	0.6833	+Z	Bottom	0.6480	Min Temp %	0.6857	81.240	OK
X-X, +0.90D+W	0.6833	-Z	Bottom	0.6480	Min Temp %	0.6857	81.240	OK
X-X, +0.90D+E	0.8242	+Z	Bottom	0.6480	Min Temp %	0.6857	81.240	OK
X-X, +0.90D+E	0.8242	-Z	Bottom	0.6480	Min Temp %	0.6857	81.240	OK
Z-Z, +1.40D	0.1352	-X	Top	0.6480	Min Temp %	0.70	82.888	OK
Z-Z, +1.40D	4.888	+X	Bottom	0.6480	Min Temp %	0.70	82.888	OK
Z-Z, +1.20D	0.1159	-X	Top	0.6480	Min Temp %	0.70	82.888	OK
Z-Z, +1.20D	4.190	+X	Bottom	0.6480	Min Temp %	0.70	82.888	OK
Z-Z, +1.20D+0.50W	0.3791	-X	Top	0.6480	Min Temp %	0.70	82.888	OK
Z-Z, +1.20D+0.50W	7.432	+X	Bottom	0.6480	Min Temp %	0.70	82.888	OK
Z-Z, +1.20D+W	0.4890	-X	Top	0.6480	Min Temp %	0.70	82.888	OK
Z-Z, +1.20D+W	11.073	+X	Bottom	0.6480	Min Temp %	0.70	82.888	OK
Z-Z, +1.20D+E	0.4890	-X	Top	0.6480	Min Temp %	0.70	82.888	OK
Z-Z, +1.20D+E	14.190	+X	Bottom	0.6480	Min Temp %	0.70	82.888	OK
Z-Z, +0.90D+W	0.3668	-X	Top	0.6480	Min Temp %	0.70	82.888	OK
Z-Z, +0.90D+W	10.860	+X	Bottom	0.6480	Min Temp %	0.70	82.888	OK
Z-Z, +0.90D+E	0.3668	-X	Top	0.6480	Min Temp %	0.70	82.888	OK
Z-Z, +0.90D+E	14.970	+X	Bottom	0.6480	Min Temp %	0.70	82.888	OK

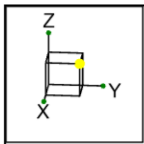
One Way Shear

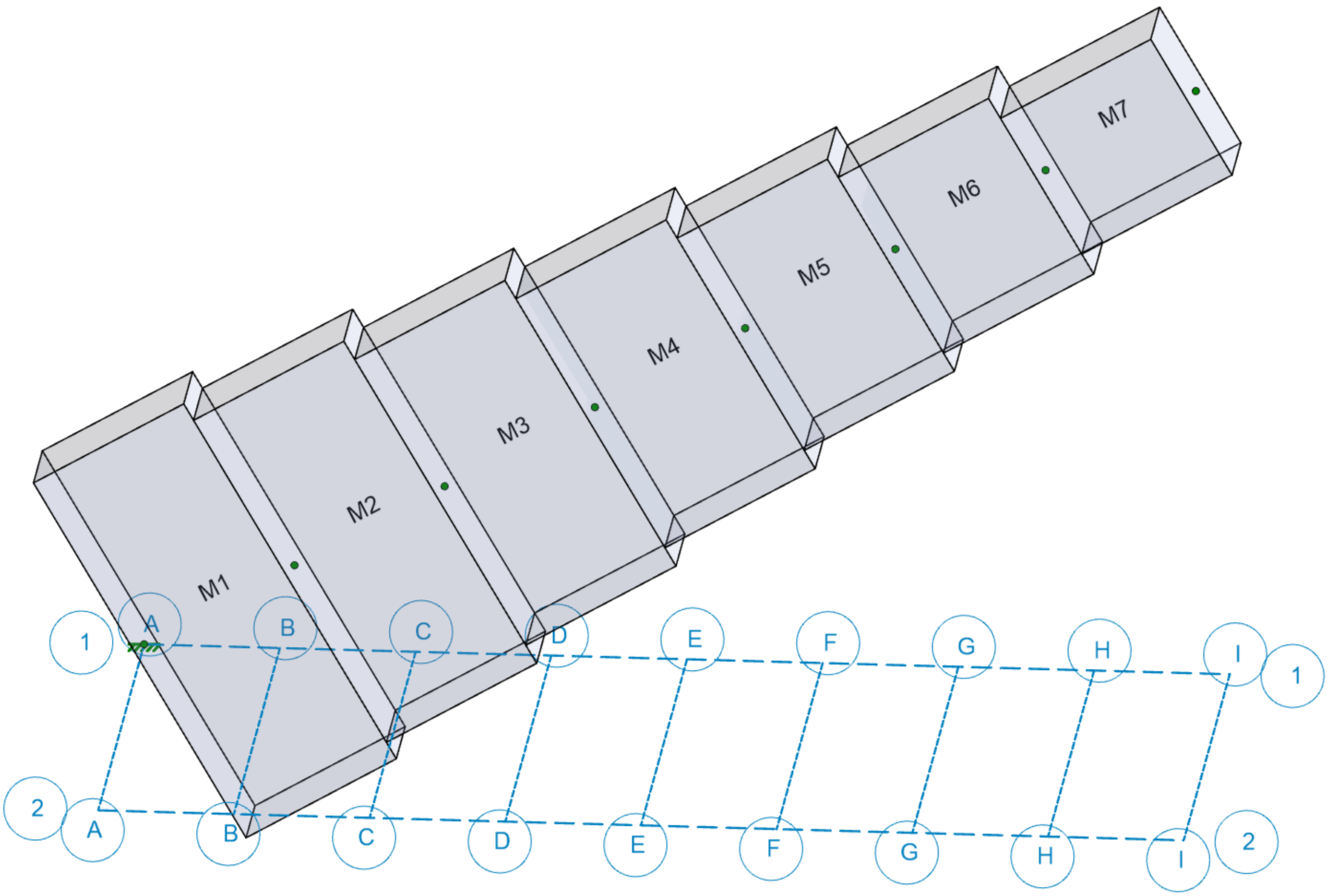
Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0.00 psi	3.21 psi	0.71 psi	0.71 psi	3.21 psi	82.16 psi	0.04	OK
+1.20D	0.00 psi	2.75 psi	0.61 psi	0.61 psi	2.75 psi	82.16 psi	0.03	OK
+1.20D+0.50W	0.00 psi	4.90 psi	0.61 psi	0.61 psi	4.90 psi	82.16 psi	0.06	OK
+1.20D+W	0.00 psi	7.32 psi	0.61 psi	0.61 psi	7.32 psi	82.16 psi	0.09	OK
+1.20D+E	0.00 psi	9.40 psi	0.70 psi	0.70 psi	9.40 psi	82.16 psi	0.11	OK
+0.90D+W	0.00 psi	7.21 psi	0.46 psi	0.46 psi	7.21 psi	82.16 psi	0.09	OK
+0.90D+E	0.00 psi	10.00 psi	0.55 psi	0.55 psi	10.00 psi	82.16 psi	0.12	OK

Two-Way "Punching" Shear

All units k

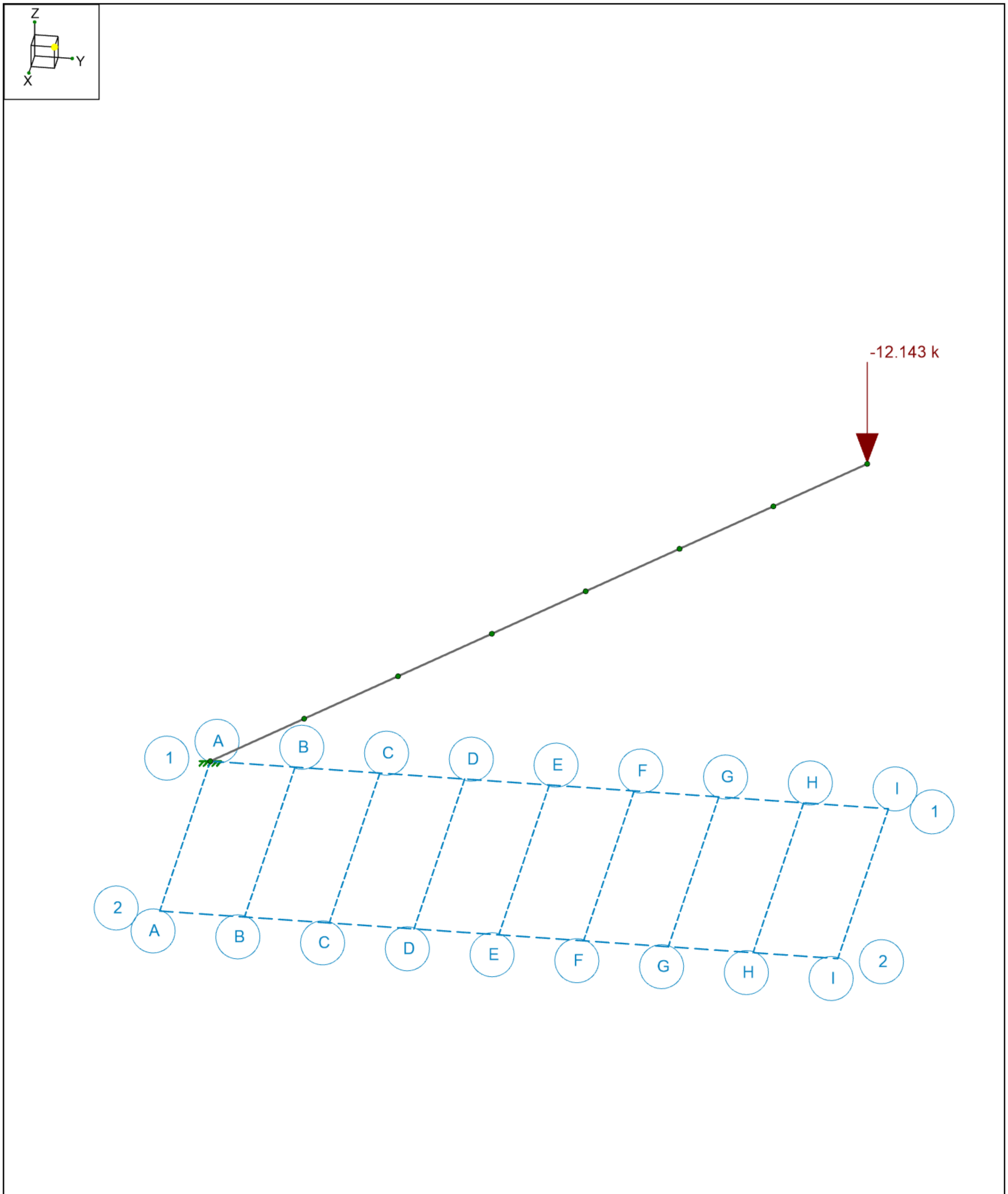
Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	1.40 psi	153.78 psi	0.009121	OK
+1.20D	1.20 psi	153.78 psi	0.007818	OK
+1.20D+0.50W	1.20 psi	153.78 psi	0.007818	OK
+1.20D+W	1.32 psi	153.78 psi	0.008609	OK
+1.20D+E	1.89 psi	153.78 psi	0.01228	OK
+0.90D+W	1.38 psi	153.78 psi	0.009001	OK
+0.90D+E	2.24 psi	153.78 psi	0.01456	OK





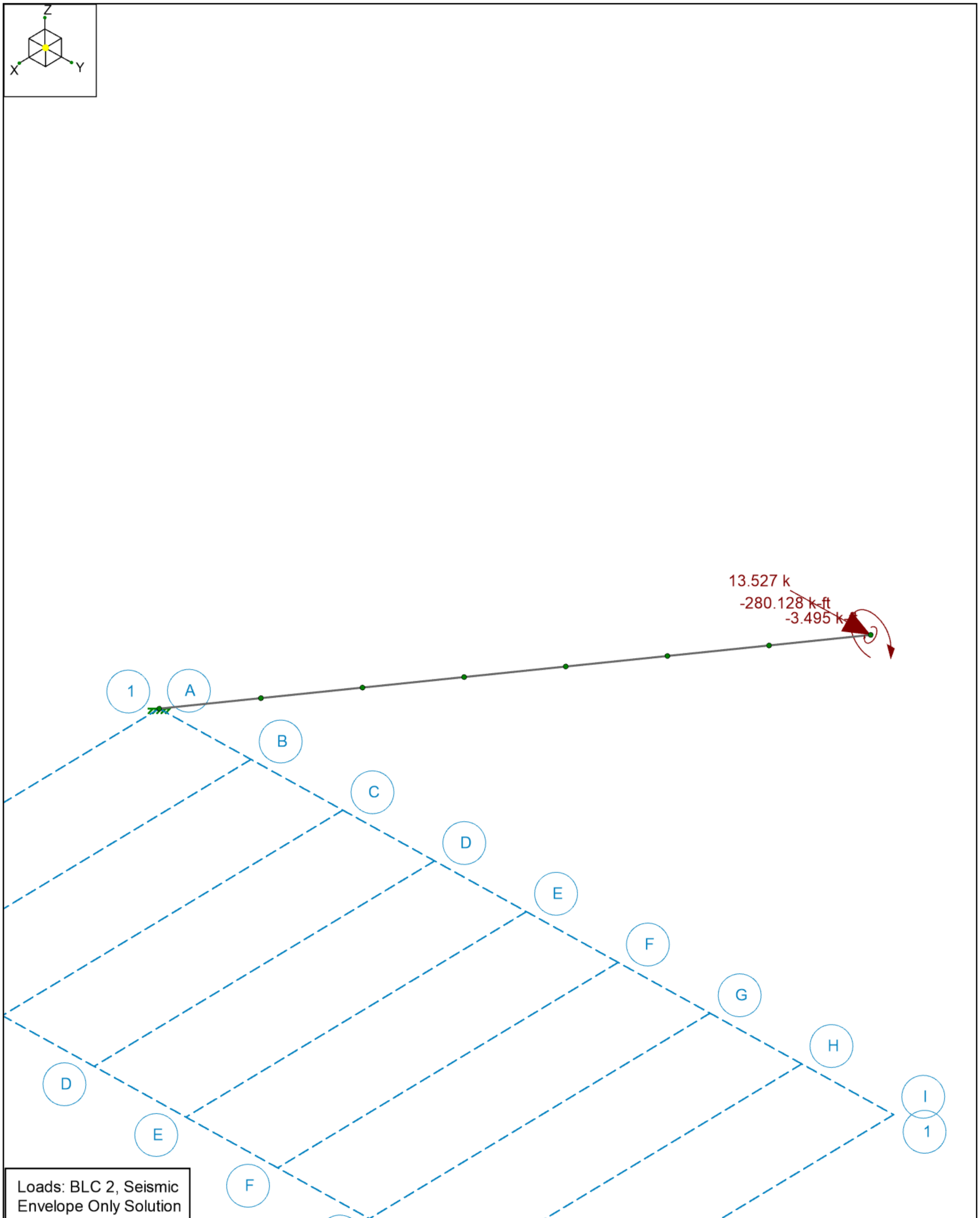
Envelope Only Solution

RISA A NEMETSCHEK COMPANY	Wildman & Morris	SH-60 Helicopter Pedestal	SK-1
	nicole.caudana		Oct 18, 2023 at 04:09 PM
	23026		SH-60 Helicopter Pedestal_1...



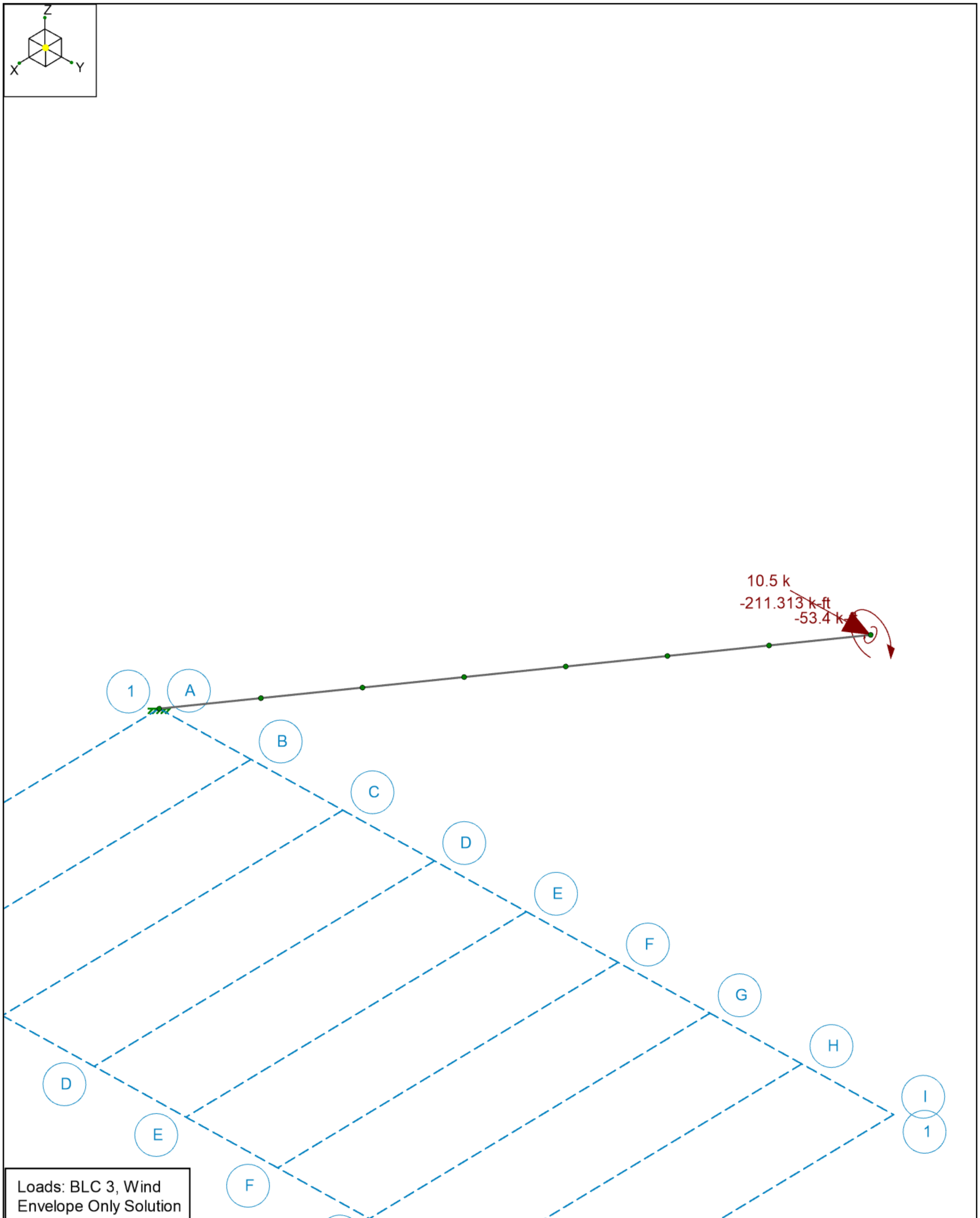
Loads: BLC 1, Dead
Envelope Only Solution

RISA A NEMETSCHEK COMPANY	Wildman & Morris	SH-60 Helicopter Pedestal	SK-1
	nicole.caudana		Oct 19, 2023 at 09:52 AM
	23026		SH-60 Helicopter Pedestal_1...



Loads: BLC 2, Seismic
Envelope Only Solution

RISA A NEMETSCHEK COMPANY	Wildman & Morris	SH-60 Helicopter Pedestal	SK-2
	nicole.caudana		Oct 19, 2023 at 10:28 AM
	23026		SH-60 Helicopter Pedestal_1...



Loads: BLC 3, Wind
Envelope Only Solution

RISA	Wildman & Morris	SH-60 Helicopter Pedestal	SK-3
	nicole.caudana		Oct 19, 2023 at 10:28 AM
	23026		SH-60 Helicopter Pedestal_1...

Node Loads and Enforced Displacements (BLC 1 : Dead)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]
1	N2	L	Z	-12.143

Node Loads and Enforced Displacements (BLC 2 : Seismic)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]
1	N2	L	MY	-3.495
2	N2	L	MX	-280.128
3	N2	L	Y	13.527

Node Loads and Enforced Displacements (BLC 3 : Wind)

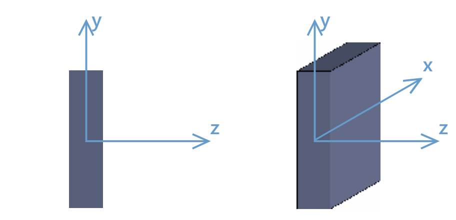
	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]
1	N2	L	Y	10.5
2	N2	L	MX	-211.313
3	N2	L	MY	-53.4

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	Deflection 1	Yes	Y	DL	1								
2	Deflection 2	Yes	Y	LL	1								
3	Deflection 3	Yes	Y	DL	1	LL	1						
4	IBC 21/ASCE Strength 1	Yes	Y	DL	1.4								
5	IBC 21/ASCE Strength 2 (a)	Yes	Y	DL	1.2	LL	1.6	LLS	1.6				
6	IBC 21/ASCE Strength 3 (b)	Yes	Y	DL	1.2	WL	0.5						
7	IBC 21/ASCE Strength 4 (a)	Yes	Y	DL	1.2	WL	1	LL	0.5	LLS	1		
8	IBC 21/ASCE Strength 5	Yes	Y	DL	0.9	WL	1						
9	IBC 21/ASCE Strength 6 (a)	Yes	Y	DL	1.2	Sds*DL	0.2	EL	1	LL	0.5	LLS	1
10	IBC 21/ASCE Strength 6 (b)	Yes	Y	DL	1.2	Sds*DL	0.2	EL	-1	LL	0.5	LLS	1
11	IBC 21/ASCE Strength 7 (a)	Yes	Y	DL	0.9	Sds*DL	-0.2	EL	1				
12	IBC 21/ASCE Strength 7 (b)	Yes	Y	DL	0.9	Sds*DL	-0.2	EL	-1				

Detail Report: M1

Load Combination: Envelope **Code check:** 0.268 (LC 9)



Input Data

Shape:	CRECT74.875X18	**I Node:**	N1
Member Type:	Column	**J Node:**	N3
Length (ft):	2.56	**I Release:**	Fixed
Material Type:	Concrete	**J Release:**	Fixed
Design Rule:	Typical	**I Offset:**	N/A
Internal Sections:	97	**J Offset:**	N/A
Design Code:	ACI 318-19	**T/C Only:**	Both Way

Material Properties

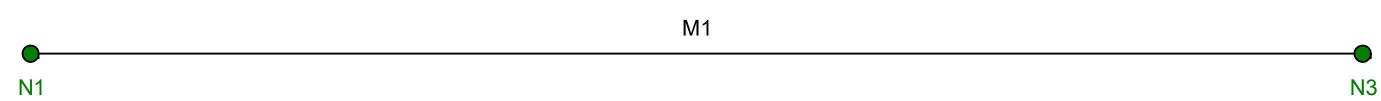
Material:	Conc4000NW	**Therm. Coeff. ($1e^5$ $F^o e^{-1}$):**	0.6	**Flex Steel (ksi):**	60
E (ksi):	3644	**Density (k/ft^3):**	0.145	**Shear Steel (ksi):**	60
G (ksi):	1584	**f'_c (ksi):**	4		
Nu:	0.15	**λ:**	1		

Shape Properties

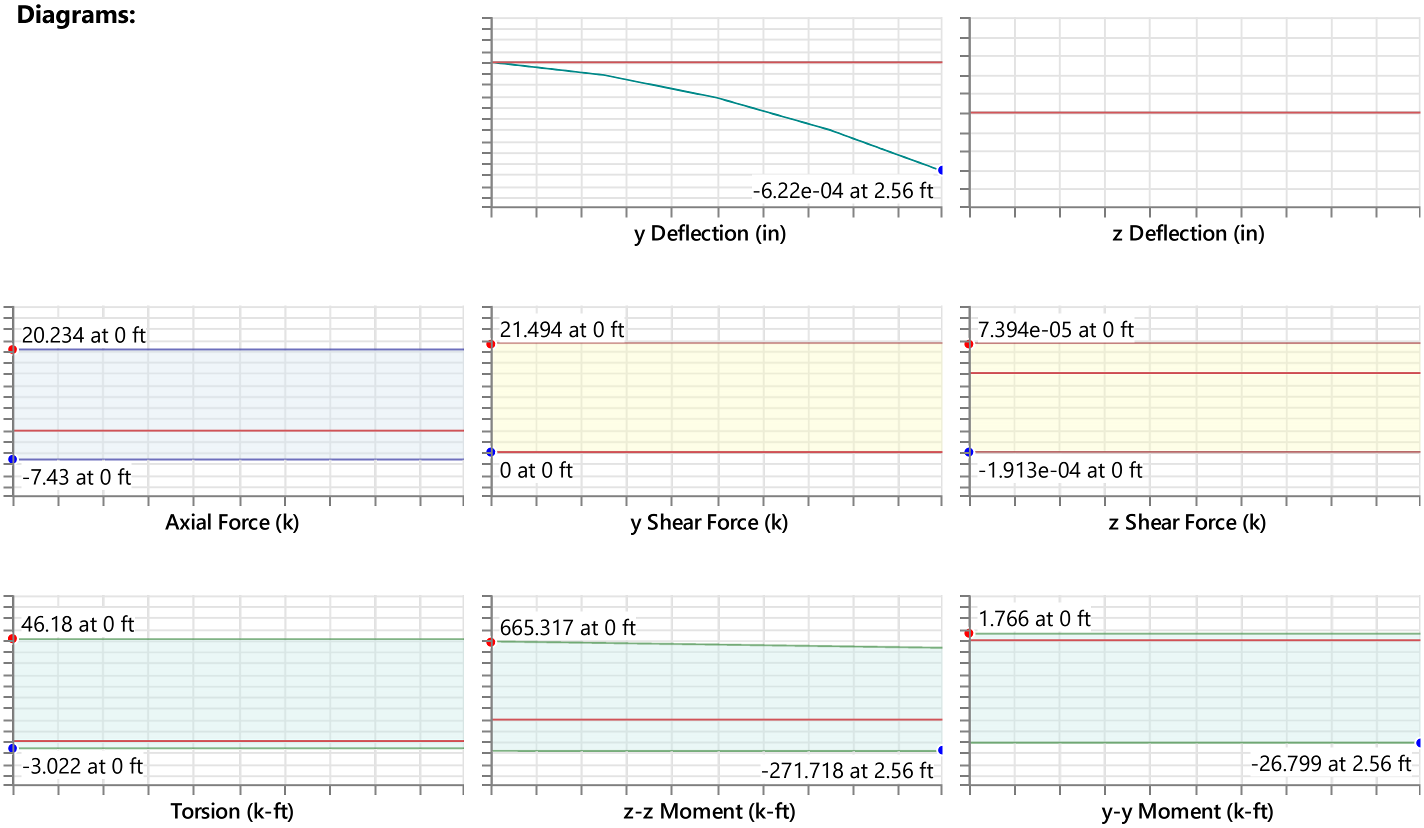
D (in):	74.875	**W (in):**	18

Design Properties

Cm_{y-y}:	N/A	**z sway:**	No	**Effective "I"(Service) (in^4):**	6.303e+5
Cm_{z-z}:	N/A	**Concrete Stress Block:**	Rectangular	**Flex Rebar Set:**	ASTM A615
K_{y-y}:	1	**Cracked Sections Used:**	Yes	**Shear Rebar Set:**	ASTM A615
K_{z-z}:	1	**Cracked "I" Factor:**	0.7	**Side Cover (in):**	1.5
y sway:	No	**Effective "I" (in^4):**	4.408e+5	**Legs/Stirrup:**	2



Diagrams:



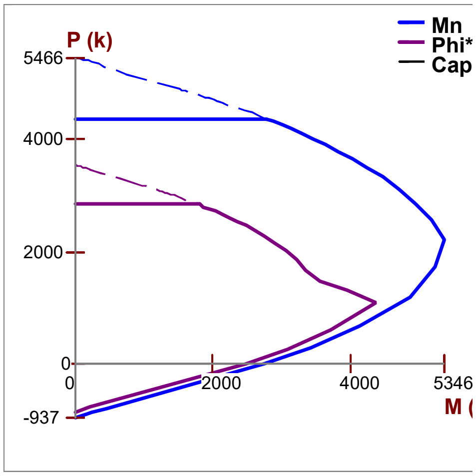
WARNING: Detail Report Based On Less Than 10 Sections!

ACI 318-19 Code Check

Column Design does not consider any Torsional Moments
Warning: Exact Integration selected but PCA method used
Factored torsional moment Tu exceeds the threshold torsion per ACI 318-19 22.7.4.1

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	-	-	-	-	-
Flexural Reinforcement	9	13.477 in^2	15.615 in^2	-	**PASS**
Axial Capacity	-	0 k	-	-	**PASS**
Bending Unity Check	9	665.317 k-ft	2482.766 k-ft	**0.268**	**PASS**
y Shear Design Strength	9	21.494 k	294.131 k	**0.073**	**PASS**
z Shear Design Strength	8	0.0001913 k	107.867 k	**1.774e-6**	**PASS**
Threshold Torsion	-	3.022 k-ft	39.785 k-ft	**0.076**	**PASS**

Column Interaction Diagram



Span Information

Span	Span Length (ft)	I-Face Dist. (in)	J-Face Dist. (in)
1	0 - 2.6	0	0

Column Steel

Span	Main Bars	UC Max	Gov LC	Loc (ft)	Pu (k)	Muy (k-ft)	Muz (k-ft)
1	10 #11	0.268	9	0 ft	0	0	665.317

Axial Span Results

Span	Phi$_{e}$ff	Pn (k)	Po (k)	Rho Gross	As Prvd (in^2)
1	0.9	-1	-936.87	0.0116	15.615

Bending Span Results

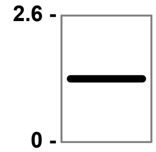
Span	ecc. y (ft)	ecc. z (ft)	NA y-y (ft)	NA z-z (ft)	Mny (k-ft)	Mnz (k-ft)	Mnoy (k-ft)	Mnoz (k-ft)
1	0	0	-2.494			2758.629		

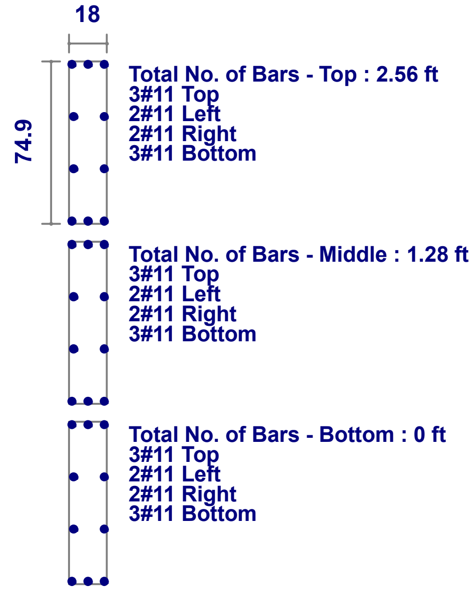
Shear Steel

Bars Provided: 2 #5 @12in

Rebar Detailing

Region(ft) | Shear Steel | Flexural Layout

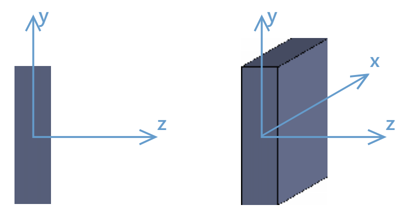




Span 1

Detail Report: M2

Load Combination: Envelope **Code check:** 0.273 (LC 9)



Input Data

Shape:	CRECT67.938X18	**I Node:**	N3
Member Type:	Column	**J Node:**	N4
Length (ft):	2.56	**I Release:**	Fixed
Material Type:	Concrete	**J Release:**	Fixed
Design Rule:	Typical	**I Offset:**	N/A
Internal Sections:	97	**J Offset:**	N/A
Design Code:	ACI 318-19	**T/C Only:**	Both Way

Material Properties

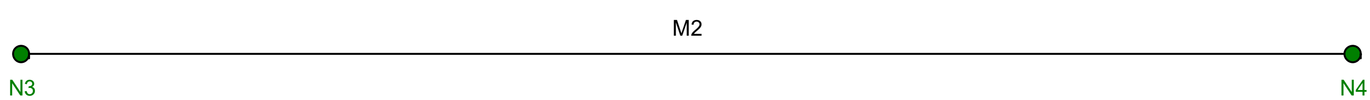
Material:	Conc4000NW	**Therm. Coeff. (1e^{5} ^{o}F e^{-1}):**	0.6	**Flex Steel (ksi):**	60
E (ksi):	3644	**Density (k/ft^{3}):**	0.145	**Shear Steel (ksi):**	60
G (ksi):	1584	**f'_c (ksi):**	4		
Nu:	0.15	**λ:**	1		

Shape Properties

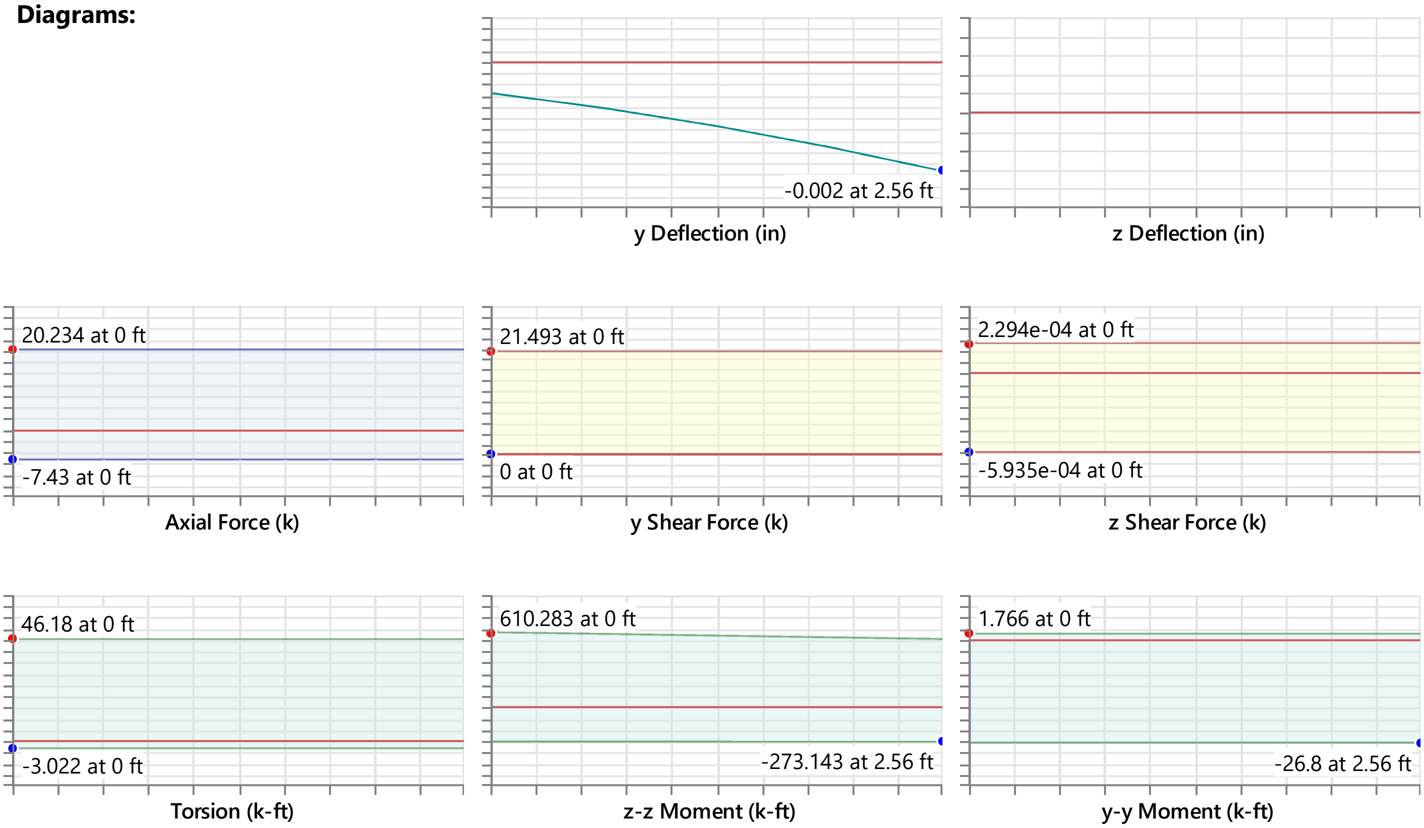
D (in):	67.938	**W (in):**	18

Design Properties

$C_{m\ y\text{-}y}$:	N/A	**z sway:**	No	**Effective "I"(Service) (in^{4}):**	4.708e+5
$C_{m\ z\text{-}z}$:	N/A	**Concrete Stress Block:**	Rectangular	**Flex Rebar Set:**	ASTM A615
$K_{y\text{-}y}$:	1	**Cracked Sections Used:**	Yes	**Shear Rebar Set:**	ASTM A615
$K_{z\text{-}z}$:	1	**Cracked "I" Factor:**	0.7	**Side Cover (in):**	1.5
y sway:	No	**Effective "I" (in^{4}):**	3.293e+5	**Legs/Stirrup:**	2



Diagrams:



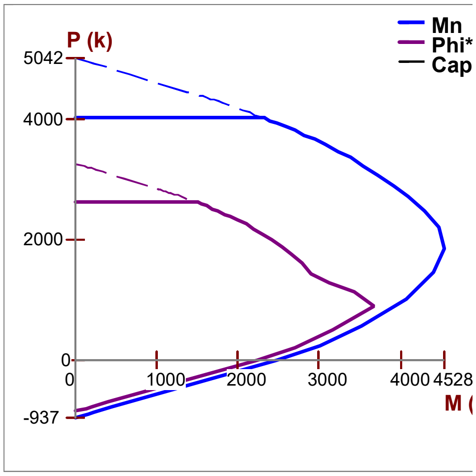
WARNING: Detail Report Based On Less Than 10 Sections!

ACI 318-19 Code Check

Column Design does not consider any Torsional Moments
Warning: Exact Integration selected but PCA method used
Factored torsional moment Tu exceeds the threshold torsion per ACI 318-19 22.7.4.1

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	-	-	-	-	-
Flexural Reinforcement	9	12.229 in^2	15.615 in^2	-	**PASS**
Axial Capacity	-	0 k	-	-	**PASS**
Bending Unity Check	9	610.283 k-ft	2239.051 k-ft	**0.273**	**PASS**
y Shear Design Strength	9	21.493 k	266.323 k	**0.081**	**PASS**
z Shear Design Strength	8	0.0005935 k	103.48 k	**5.736e-6**	**PASS**
Threshold Torsion	-	3.022 k-ft	35.5 k-ft	**0.085**	**PASS**

Column Interaction Diagram



Span Information

Span	Span Length (ft)	I-Face Dist. (in)	J-Face Dist. (in)
1	0 - 2.6	0	0

Column Steel

Span	Main Bars	UC Max	Gov LC	Loc (ft)	Pu (k)	Muy (k-ft)	Muz (k-ft)
1	10 #11	0.273	9	0 ft	0	0	610.283

Axial Span Results

Span	Phi$_e$ff	Pn (k)	Po (k)	Rho Gross	As Prvd (in^2)
1	0.9	-1	-936.87	0.0128	15.615

Bending Span Results

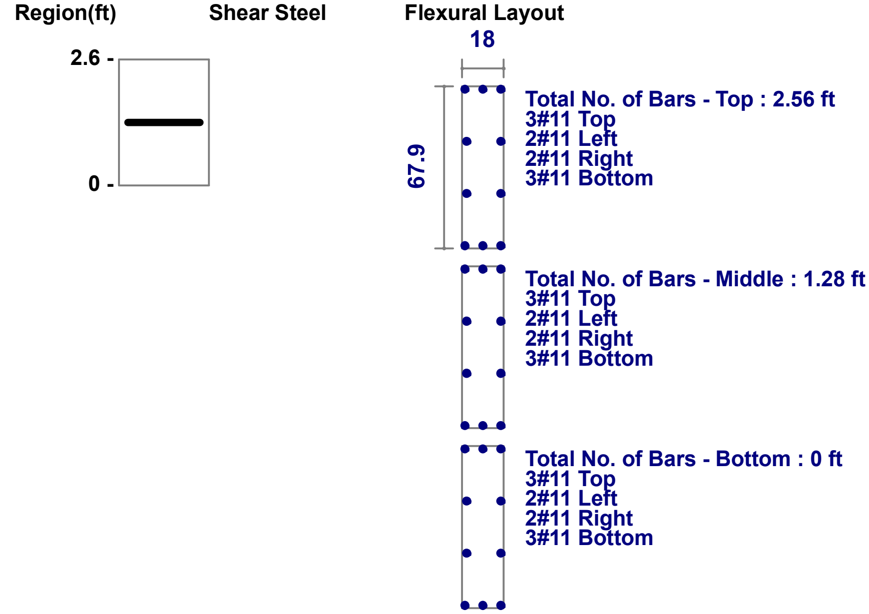
Span	ecc. y (ft)	ecc. z (ft)	NA y-y (ft)	NA z-z (ft)	Mny (k-ft)	Mnz (k-ft)	Mnoy (k-ft)	Mnoz (k-ft)
1	0	0	-2.205			2487.835		

Shear Steel

Bars Provided: 2 #5 @12in

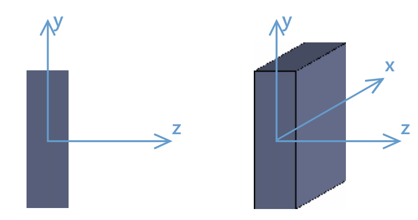
Rebar Detailing

Span 1



Detail Report: M3

Load Combination: Envelope **Code check:** 0.282 (LC 9)



Input Data

Shape:	CRECT60.25X18	**I Node:**	N4
Member Type:	Column	**J Node:**	N5
Length (ft):	2.56	**I Release:**	Fixed
Material Type:	Concrete	**J Release:**	Fixed
Design Rule:	Typical	**I Offset:**	N/A
Internal Sections:	97	**J Offset:**	N/A
Design Code:	ACI 318-19	**T/C Only:**	Both Way

Material Properties

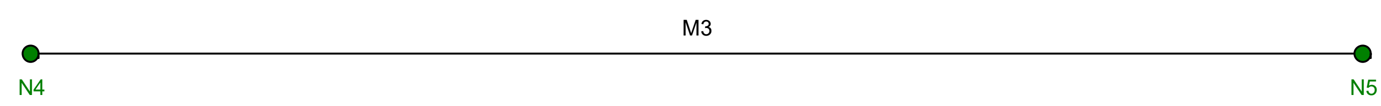
Material:	Conc4000NW	**Therm. Coeff. ($1e^5$ F^oe^{-1}):**	0.6	**Flex Steel (ksi):**	60
E (ksi):	3644	**Density (k/ft^3):**	0.145	**Shear Steel (ksi):**	60
G (ksi):	1584	**f'_c (ksi):**	4		
Nu:	0.15	**λ:**	1		

Shape Properties

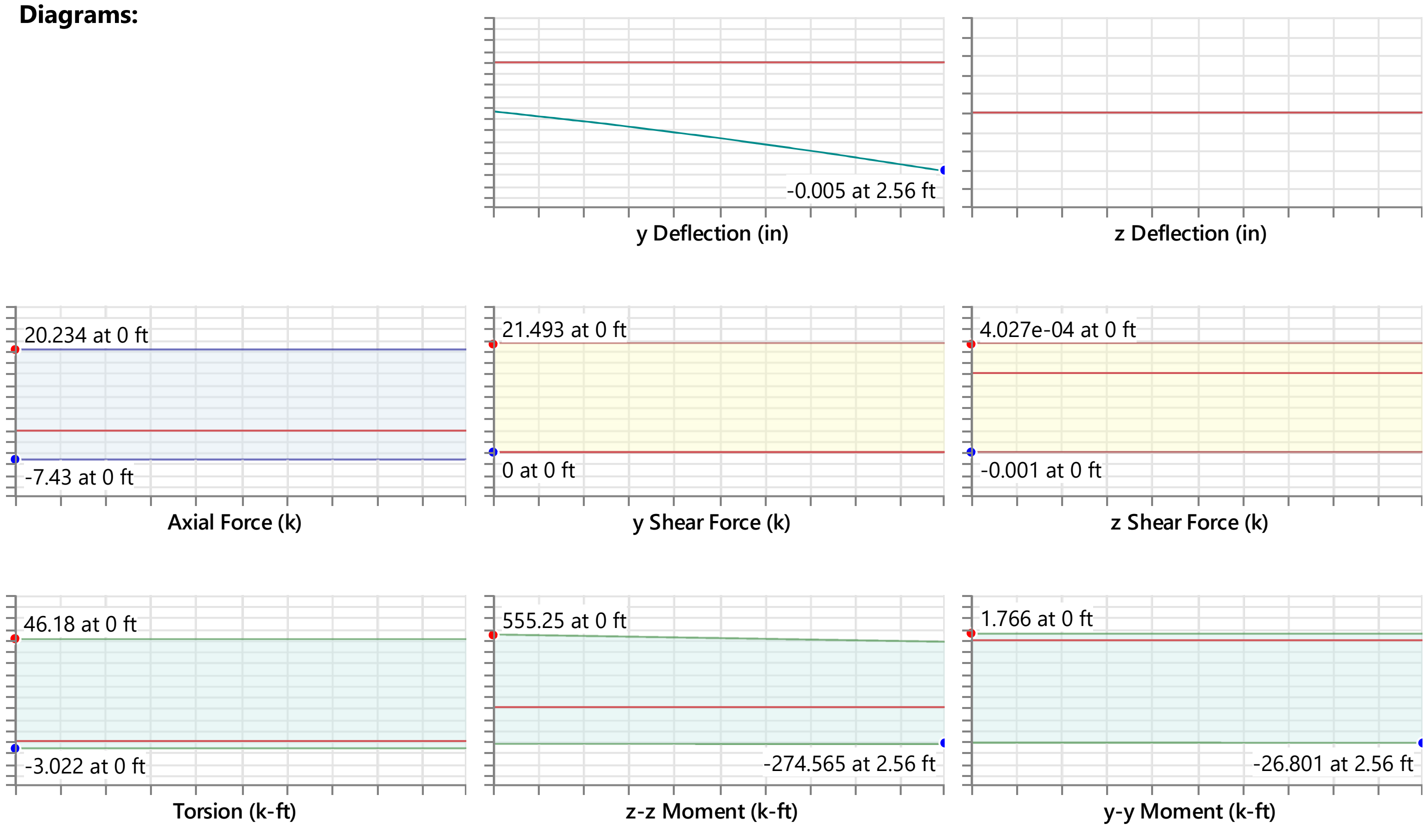
D (in):	60.25	**W (in):**	18

Design Properties

Cm_{y-y}:	N/A	**z sway:**	No	**Effective "I"(Service) (in^4):**	3.284e+5
Cm_{z-z}:	N/A	**Concrete Stress Block:**	Rectangular	**Flex Rebar Set:**	ASTM A615
K_{y-y}:	1	**Cracked Sections Used:**	Yes	**Shear Rebar Set:**	ASTM A615
K_{z-z}:	1	**Cracked "I" Factor:**	0.7	**Side Cover (in):**	1.5
y sway:	No	**Effective "I" (in^4):**	2.296e+5	**Legs/Stirrup:**	2



Diagrams:



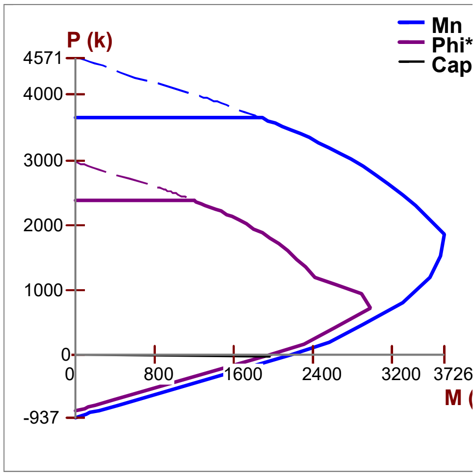
WARNING: Detail Report Based On Less Than 10 Sections!

ACI 318-19 Code Check

Column Design does not consider any Torsional Moments
Warning: Exact Integration selected but PCA method used
Factored torsional moment Tu exceeds the threshold torsion per ACI 318-19 22.7.4.1

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	-	-	-	-	-
Flexural Reinforcement	9	10.845 in^2	15.615 in^2	-	**PASS**
Axial Capacity	-	0 k	-	-	**PASS**
Bending Unity Check	9	555.25 k-ft	1968.952 k-ft	**0.282**	**PASS**
y Shear Design Strength	9	21.493 k	235.505 k	**0.091**	**PASS**
z Shear Design Strength	8	0.001 k	132.277 k	**7.879e-6**	**PASS**
Threshold Torsion	-	3.022 k-ft	30.783 k-ft	**0.098**	**PASS**

Column Interaction Diagram



Span Information

Span	Span Length (ft)	I-Face Dist. (in)	J-Face Dist. (in)
1	0 - 2.6	0	0

Column Steel

Span	Main Bars	UC Max	Gov LC	Loc (ft)	Pu (k)	Muy (k-ft)	Muz (k-ft)
1	10 #11	0.282	9	0 ft	0	0	555.25

Axial Span Results

Span	Phi$_e$ff	Pn (k)	Po (k)	Rho Gross	As Prvd (in^2)
1	0.9	-1	-936.87	0.0144	15.615

Bending Span Results

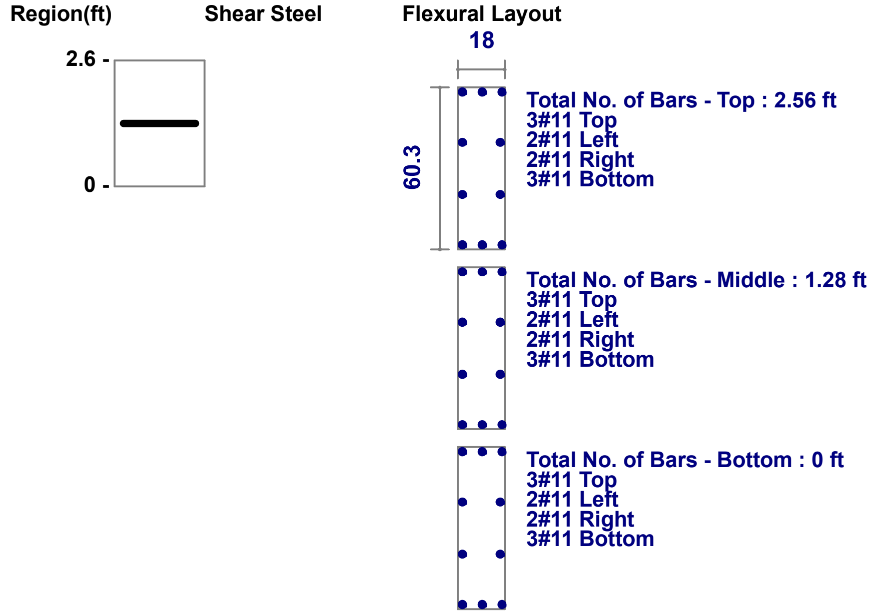
Span	ecc. y (ft)	ecc. z (ft)	NA y-y (ft)	NA z-z (ft)	Mny (k-ft)	Mnz (k-ft)	Mnoy (k-ft)	Mnoz (k-ft)
1	0	0	-1.885			2187.724		

Shear Steel

Bars Provided: 2 #5 @12in

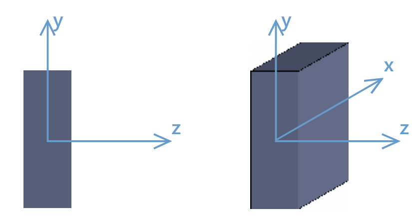
Rebar Detailing

Span 1



Detail Report: M4

Load Combination: Envelope **Code check:** 0.295 (LC 9)



Input Data

Shape:	CRECT52.5X18	**I Node:**	N5
Member Type:	Column	**J Node:**	N6
Length (ft):	2.56	**I Release:**	Fixed
Material Type:	Concrete	**J Release:**	Fixed
Design Rule:	Typical	**I Offset:**	N/A
Internal Sections:	97	**J Offset:**	N/A
Design Code:	ACI 318-19	**T/C Only:**	Both Way

Material Properties

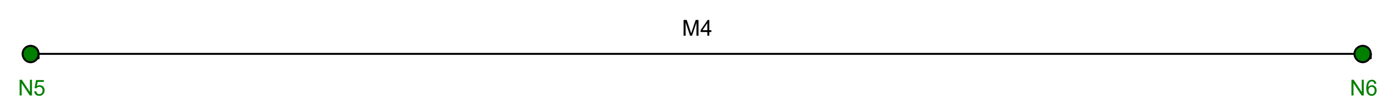
Material:	Conc4000NW	**Therm. Coeff. ($1e^5$ $F^o e^{-1}$):**	0.6	**Flex Steel (ksi):**	60
E (ksi):	3644	**Density (k/ft^3):**	0.145	**Shear Steel (ksi):**	60
G (ksi):	1584	**f'_c (ksi):**	4		
Nu:	0.15	**λ:**	1		

Shape Properties

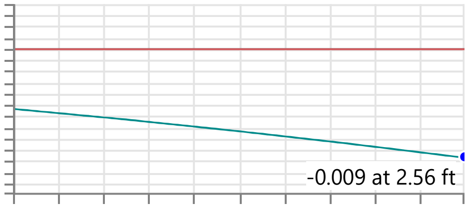
D (in):	52.5	**W (in):**	18

Design Properties

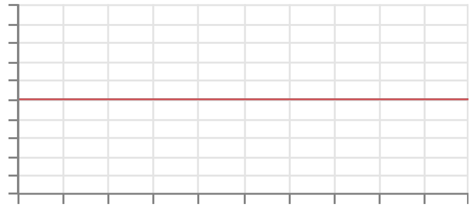
Cm_{y-y}:	N/A	**z sway:**	No	**Effective "I"(Service) (in^4):**	2.173e+5
Cm_{z-z}:	N/A	**Concrete Stress Block:**	Rectangular	**Flex Rebar Set:**	ASTM A615
K_{y-y}:	1	**Cracked Sections Used:**	Yes	**Shear Rebar Set:**	ASTM A615
K_{z-z}:	1	**Cracked "I" Factor:**	0.7	**Side Cover (in):**	1.5
y sway:	No	**Effective "I" (in^4):**	1.519e+5	**Legs/Stirrup:**	2



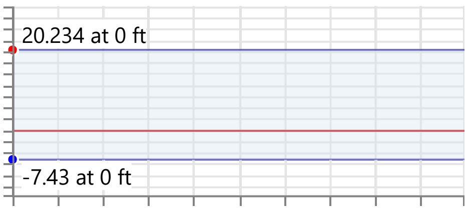
Diagrams:



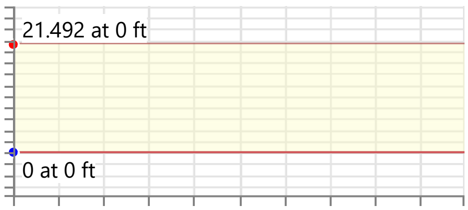
y Deflection (in)



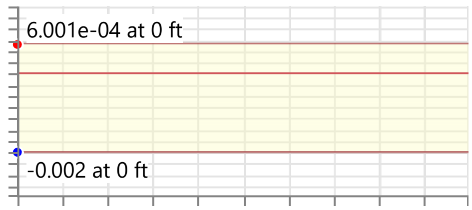
z Deflection (in)



Axial Force (k)

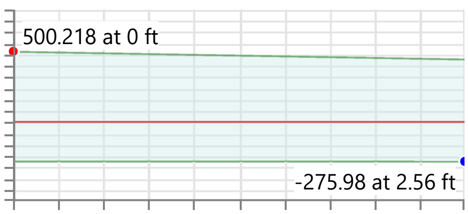


y Shear Force (k)

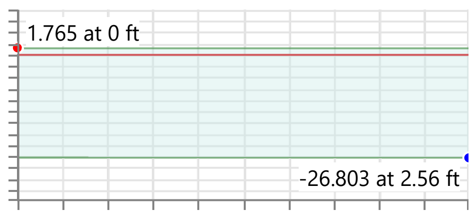


z Shear Force (k)

Torsion (k-ft)



z-z Moment (k-ft)



y-y Moment (k-ft)

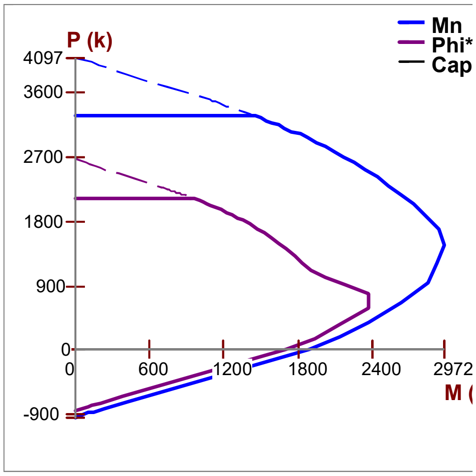
WARNING: Detail Report Based On Less Than 10 Sections!

ACI 318-19 Code Check

Column Design does not consider any Torsional Moments
Warning: Exact Integration selected but PCA method used
Factored torsional moment Tu exceeds the threshold torsion per ACI 318-19 22.7.4.1

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	-	-	-	-	-
Flexural Reinforcement	9	9.45 in^2	15.615 in^2	-	PASS
Axial Capacity	-	0 k	-	-	PASS
Bending Unity Check	9	500.218 k-ft	1696.674 k-ft	**0.295**	PASS
y Shear Design Strength	9	21.492 k	204.439 k	**0.105**	PASS
z Shear Design Strength	8	0.002 k	120.146 k	**1.292e-5**	PASS
Threshold Torsion	-	3.022 k-ft	26.073 k-ft	**0.116**	PASS

Column Interaction Diagram



Span Information

Span	Span Length (ft)	I-Face Dist. (in)	J-Face Dist. (in)
1	0 - 2.6	0	0

Column Steel

Span	Main Bars	UC Max	Gov LC	Loc (ft)	Pu (k)	Muy (k-ft)	Muz (k-ft)
1	10 #11	0.295	9	0 ft	0	0	500.218

Axial Span Results

Span	Phi$_{e}$ff	Pn (k)	Po (k)	Rho Gross	As Prvd (in^2)
1	0.9	-1	-936.87	0.0165	15.615

Bending Span Results

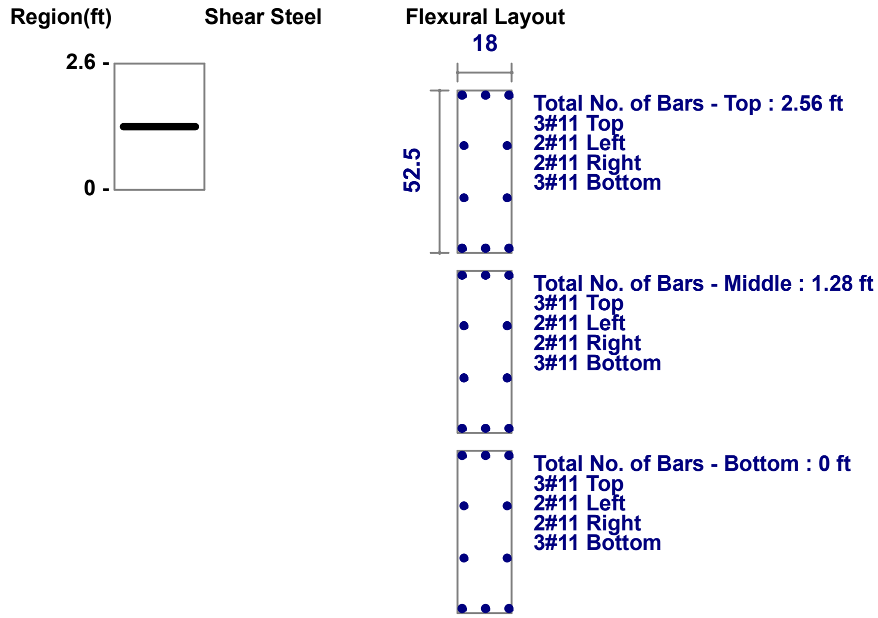
Span	ecc. y (ft)	ecc. z (ft)	NA y-y (ft)	NA z-z (ft)	Mny (k-ft)	Mnz (k-ft)	Mnoy (k-ft)	Mnoz (k-ft)
1	0	0	-1.562			1885.193		

Shear Steel

Bars Provided: 2 #5 @12in

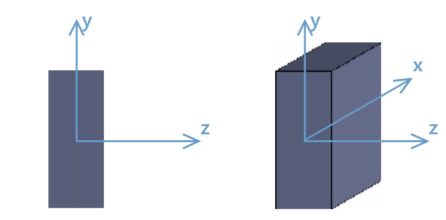
Rebar Detailing

Span 1



Detail Report: M5

Load Combination: Envelope **Code check:** 0.380 (LC 9)



Input Data

Shape:	CRECT44.75X18	**I Node:**	N6
Member Type:	Column	**J Node:**	N7
Length (ft):	2.56	**I Release:**	Fixed
Material Type:	Concrete	**J Release:**	Fixed
Design Rule:	Typical	**I Offset:**	N/A
Internal Sections:	97	**J Offset:**	N/A
Design Code:	ACI 318-19	**T/C Only:**	Both Way

Material Properties

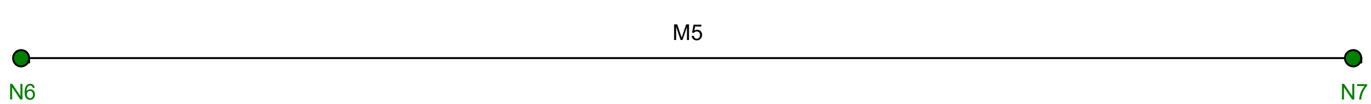
Material:	Conc4000NW	**Therm. Coeff. (1e^{5} F^{o} e^{-1}):**	0.6	**Flex Steel (ksi):**	60
E (ksi):	3644	**Density (k/ft^{3}):**	0.145	**Shear Steel (ksi):**	60
G (ksi):	1584	**f'_c (ksi):**	4		
Nu:	0.15	**λ:**	1		

Shape Properties

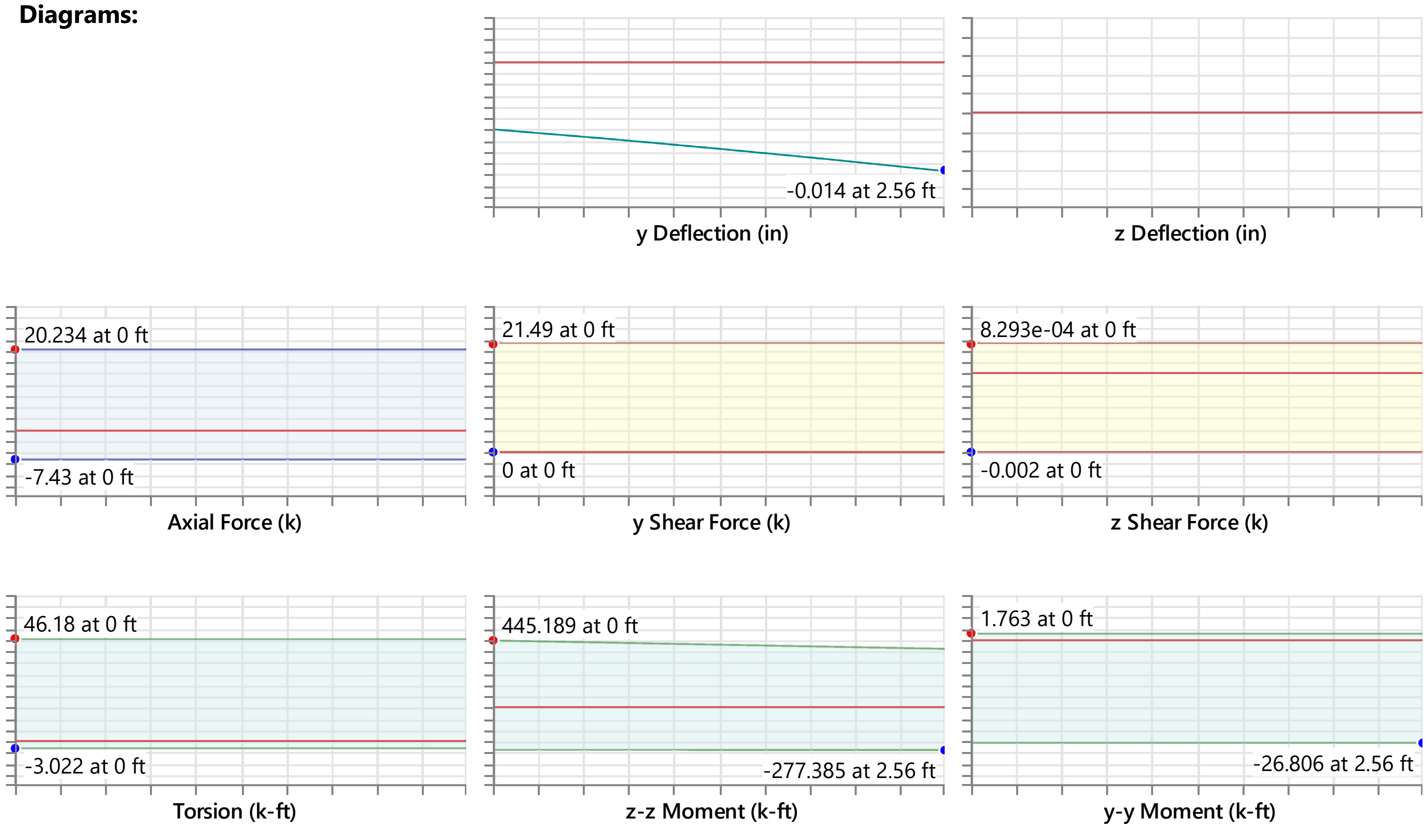
D (in):	44.75	**W (in):**	18

Design Properties

$C_{m\ y\text{-}y}$:	N/A	**z sway:**	No	**Effective "I"(Service) (in^{4}):**	1.346e+5
$C_{m\ z\text{-}z}$:	N/A	**Concrete Stress Block:**	Rectangular	**Flex Rebar Set:**	ASTM A615
$K_{y\text{-}y}$:	1	**Cracked Sections Used:**	Yes	**Shear Rebar Set:**	ASTM A615
$K_{z\text{-}z}$:	1	**Cracked "I" Factor:**	0.7	**Side Cover (in):**	1.5
y sway:	No	**Effective "I" (in^{4}):**	94095.406	**Legs/Stirrup:**	2



Diagrams:



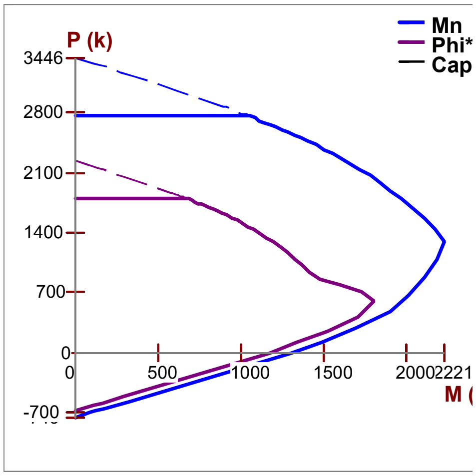
WARNING: Detail Report Based On Less Than 10 Sections!

ACI 318-19 Code Check

Column Design does not consider any Torsional Moments
Warning: Exact Integration selected but PCA method used
Factored torsional moment Tu exceeds the threshold torsion per ACI 318-19 22.7.4.1

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	-	-	-	-	-
Flexural Reinforcement	9	8.055 in^2	12.492 in^2	-	PASS
Axial Capacity	-	0 k	-	-	PASS
Bending Unity Check	9	445.189 k-ft	1171.671 k-ft	**0.38**	PASS
y Shear Design Strength	9	21.49 k	173.372 k	**0.124**	PASS
z Shear Design Strength	8	0.002 k	108.014 k	**1.987e-5**	PASS
Threshold Torsion	-	3.022 k-ft	21.427 k-ft	**0.141**	PASS

Column Interaction Diagram



Span Information

Span	Span Length (ft)	I-Face Dist. (in)	J-Face Dist. (in)
1	0 - 2.6	0	0

Column Steel

Span	Main Bars	UC Max	Gov LC	Loc (ft)	Pu (k)	Muy (k-ft)	Muz (k-ft)
1	8 #11	0.380	9	0 ft	0	0	445.189

Axial Span Results

Span	$Phi_{e}ff$	Pn (k)	Po (k)	Rho Gross	As Prvd (in^2)
1	0.9	-1	-749.496	0.0155	12.492

Bending Span Results

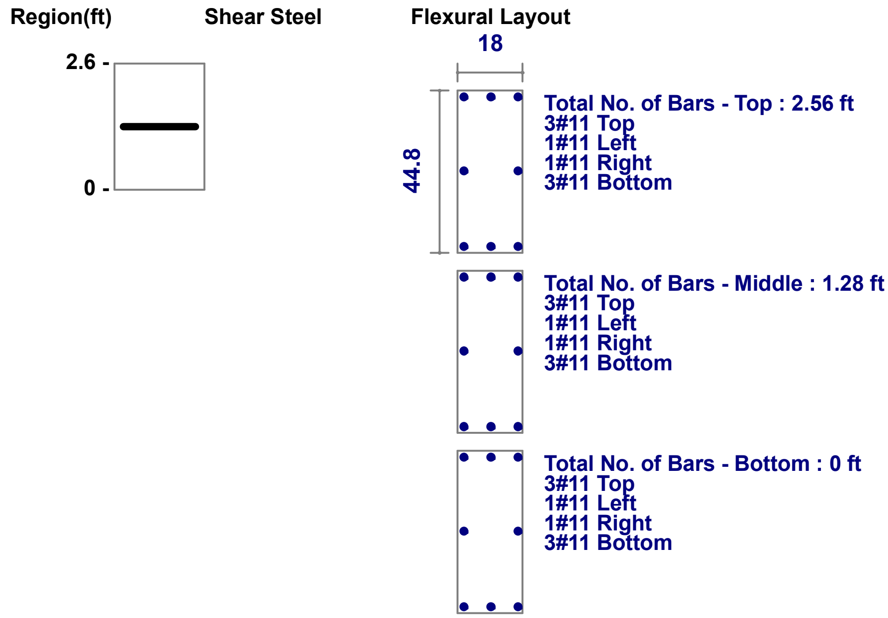
Span	ecc. y (ft)	ecc. z (ft)	NA y-y (ft)	NA z-z (ft)	Mny (k-ft)	Mnz (k-ft)	Mnoy (k-ft)	Mnoz (k-ft)
1	0	0	-1.511			1301.857		

Shear Steel

Bars Provided: 2 #5 @12in

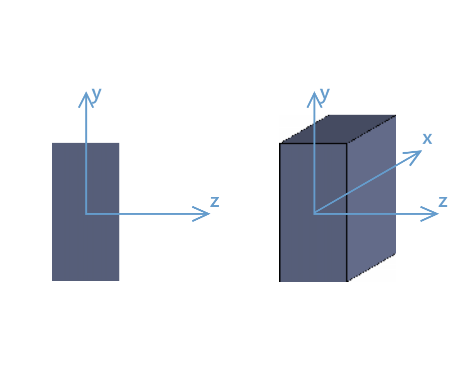
Rebar Detailing

Span 1



Detail Report: M6

Load Combination: Envelope **Code check:** 0.408 (LC 9)



Input Data

Shape:	CRECT37.062X18	**I Node:**	N7
Member Type:	Column	**J Node:**	N8
Length (ft):	2.56	**I Release:**	Fixed
Material Type:	Concrete	**J Release:**	Fixed
Design Rule:	Typical	**I Offset:**	N/A
Internal Sections:	97	**J Offset:**	N/A
Design Code:	ACI 318-19	**T/C Only:**	Both Way

Material Properties

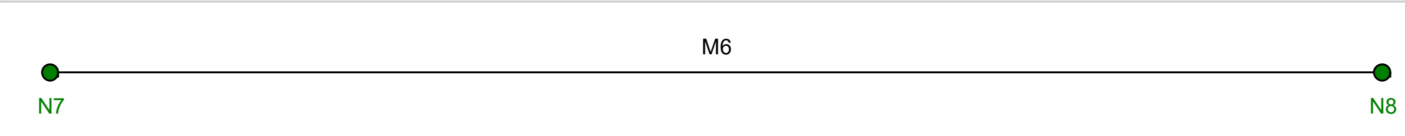
Material:	Conc4000NW	**Therm. Coeff. ($1e^5$ $F^o e^{-1}$):**	0.6	**Flex Steel (ksi):**	60
E (ksi):	3644	**Density (k/ft^3):**	0.145	**Shear Steel (ksi):**	60
G (ksi):	1584	**f'_c (ksi):**	4		
Nu:	0.15	**λ:**	1		

Shape Properties

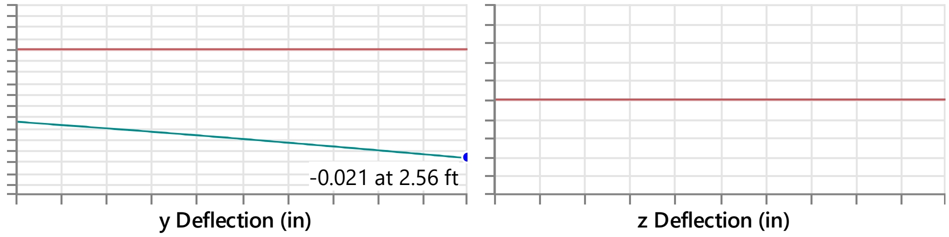
D (in):	37.062	**W (in):**	18

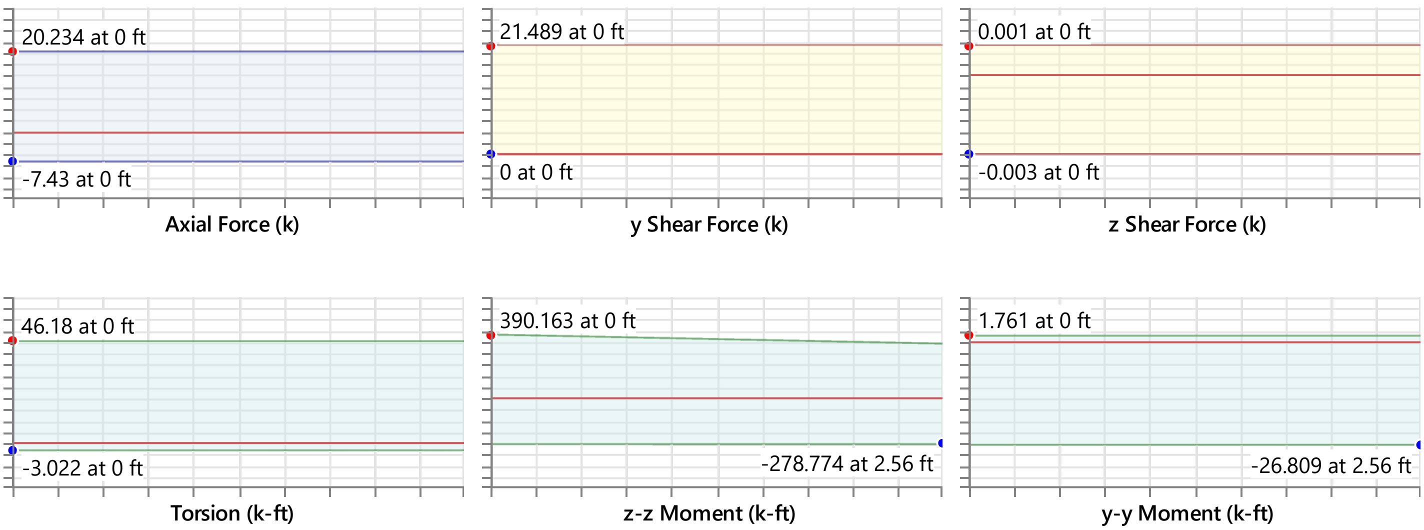
Design Properties

$C_{m\ y-y}$:	N/A	**z sway:**	No	**Effective "I"(Service) (in^4):**	76438.453
$C_{m\ z-z}$:	N/A	**Concrete Stress Block:**	Rectangular	**Flex Rebar Set:**	ASTM A615
K_{y-y}:	1	**Cracked Sections Used:**	Yes	**Shear Rebar Set:**	ASTM A615
K_{z-z}:	1	**Cracked "I" Factor:**	0.7	**Side Cover (in):**	1.5
y sway:	No	**Effective "I" (in^4):**	53453.465	**Legs/Stirrup:**	2



Diagrams:





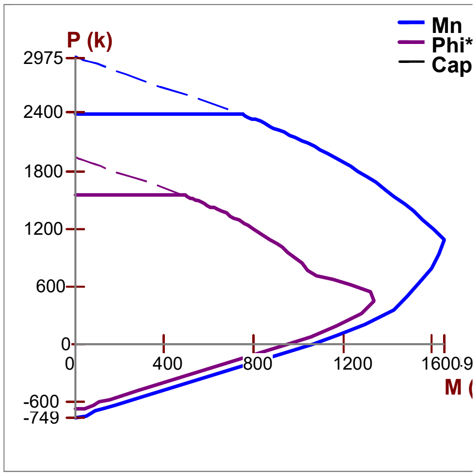
WARNING: Detail Report Based On Less Than 10 Sections!

ACI 318-19 Code Check

Column Design does not consider any Torsional Moments
Warning: Exact Integration selected but PCA method used
Factored torsional moment Tu exceeds the threshold torsion per ACI 318-19 22.7.4.1

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	-	-	-	-	-
Flexural Reinforcement	9	6.671 in^2	12.492 in^2	-	**PASS**
Axial Capacity	-	0 k	-	-	**PASS**
Bending Unity Check	9	390.163 k-ft	955.592 k-ft	**0.408**	**PASS**
y Shear Design Strength	9	21.489 k	142.554 k	**0.151**	**PASS**
z Shear Design Strength	8	0.003 k	95.98 k	**2.972e-5**	**PASS**
Threshold Torsion	-	3.022 k-ft	16.905 k-ft	**0.179**	**PASS**

Column Interaction Diagram



Span Information

Span	Span Length (ft)	I-Face Dist. (in)	J-Face Dist. (in)
1	0 - 2.6	0	0

Column Steel

Span	Main Bars	UC Max	Gov LC	Loc (ft)	Pu (k)	Muy (k-ft)	Muz (k-ft)
1	8 #11	0.408	9	0 ft	0	0	390.163

Axial Span Results

Span	$Phi_e ff$	Pn (k)	Po (k)	Rho Gross	As Prvd (in^2)
1	0.9	-1	-749.496	0.0187	12.492

Bending Span Results

Span	ecc. y (ft)	ecc. z (ft)	NA y-y (ft)	NA z-z (ft)	Mny (k-ft)	Mnz (k-ft)	Mnoy (k-ft)	Mnoz (k-ft)
1	0	0	-1.19			1061.769		

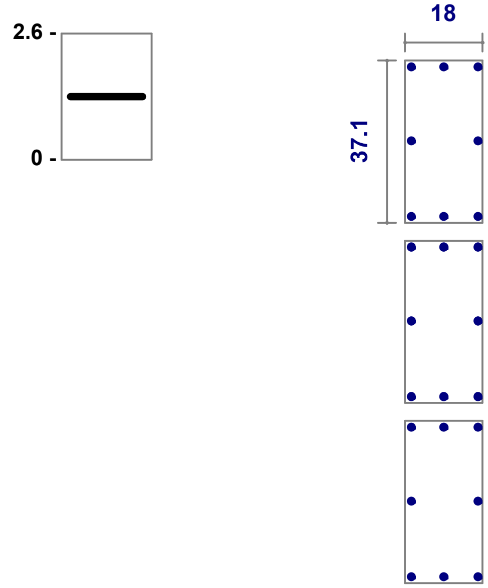
Shear Steel

Bars Provided: 2 #5 @12in

Rebar Detailing

Region(ft) | Shear Steel | Flexural Layout

Span 1



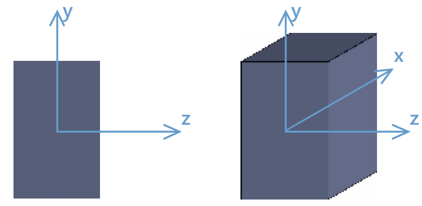
Total No. of Bars - Top : 2.56 ft
3#11 Top
1#11 Left
1#11 Right
3#11 Bottom

Total No. of Bars - Middle : 1.28 ft
3#11 Top
1#11 Left
1#11 Right
3#11 Bottom

Total No. of Bars - Bottom : 0 ft
3#11 Top
1#11 Left
1#11 Right
3#11 Bottom

Detail Report: M7

Load Combination: Envelope **Code check:** 0.466 (LC 9)



Input Data

Shape:	CRECT28.625X18	**I Node:**	N8
Member Type:	Column	**J Node:**	N2
Length (ft):	2.56	**I Release:**	Fixed
Material Type:	Concrete	**J Release:**	Fixed
Design Rule:	Typical	**I Offset:**	N/A
Internal Sections:	97	**J Offset:**	N/A
Design Code:	ACI 318-19	**T/C Only:**	Both Way

Material Properties

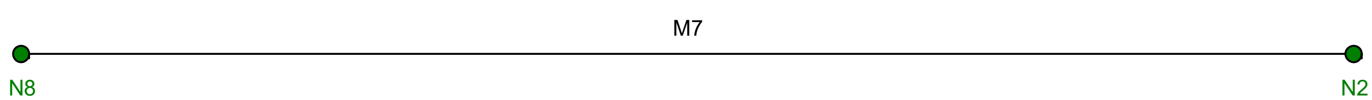
Material:	Conc4000NW	**Therm. Coeff. (1e^{5} F^{o}e^{-1}):**	0.6	**Flex Steel (ksi):**	60
E (ksi):	3644	**Density (k/ft^{3}):**	0.145	**Shear Steel (ksi):**	60
G (ksi):	1584	**f'$_{c}$ (ksi):**	4		
Nu:	0.15	**λ:**	1		

Shape Properties

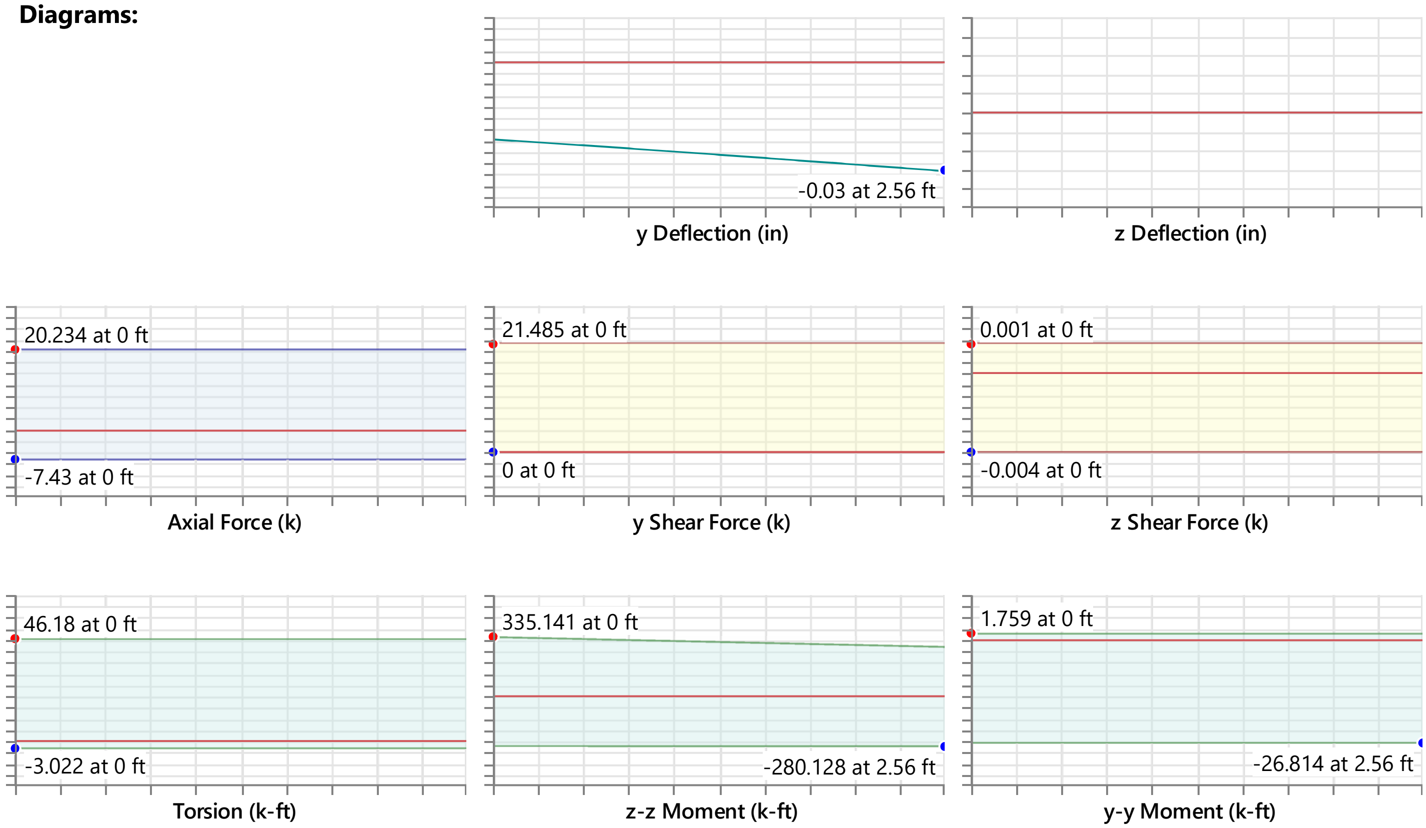
D (in):	28.625	**W (in):**	18

Design Properties

$C_{m\ y-y}$:	N/A	**z sway:**	No	**Effective "I"(Service) (in^{4}):**	35217.766
$C_{m\ z-z}$:	N/A	**Concrete Stress Block:**	Rectangular	**Flex Rebar Set:**	ASTM A615
K_{y-y}:	1	**Cracked Sections Used:**	Yes	**Shear Rebar Set:**	ASTM A615
K_{z-z}:	1	**Cracked "I" Factor:**	0.7	**Side Cover (in):**	1.5
y sway:	No	**Effective "I" (in^{4}):**	24627.809	**Legs/Stirrup:**	2



Diagrams:



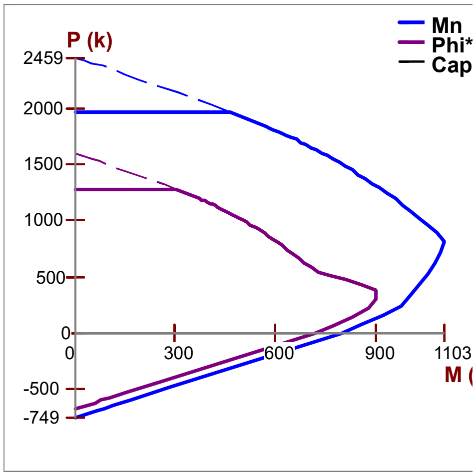
WARNING: Detail Report Based On Less Than 10 Sections!

ACI 318-19 Code Check

Column Design does not consider any Torsional Moments
Warning: Exact Integration selected but PCA method used
Factored torsional moment Tu exceeds the threshold torsion per ACI 318-19 22.7.4.1

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	-	-	-	-	-
Flexural Reinforcement	9	5.153 in^2	12.492 in^2	-	**PASS**
Axial Capacity	-	0 k	-	-	**PASS**
Bending Unity Check	9	335.141 k-ft	718.461 k-ft	**0.466**	**PASS**
y Shear Design Strength	9	21.485 k	206.099 k	**0.104**	**PASS**
z Shear Design Strength	8	0.004 k	142.001 k	**2.633e-5**	**PASS**
Threshold Torsion	-	3.022 k-ft	12.096 k-ft	**0.25**	**PASS**

Column Interaction Diagram



Span Information

Span	Span Length (ft)	I-Face Dist. (in)	J-Face Dist. (in)
1	0 - 2.6	0	0

Column Steel

Span	Main Bars	UC Max	Gov LC	Loc (ft)	Pu (k)	Muy (k-ft)	Muz (k-ft)
1	8 #11	0.466	9	0 ft	0	0	335.141

Axial Span Results

Span	$Phi_{e}ff$	Pn (k)	Po (k)	Rho Gross	As Prvd (in^2)
1	0.9	-1	-749.496	0.0242	12.492

Bending Span Results

Span	ecc. y (ft)	ecc. z (ft)	NA y-y (ft)	NA z-z (ft)	Mny (k-ft)	Mnz (k-ft)	Mnoy (k-ft)	Mnoz (k-ft)
1	0	0	-0.839			798.289		

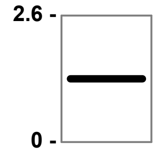
Shear Steel

Bars Provided: 10 #4 @3in

Rebar Detailing

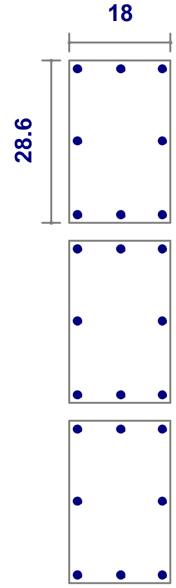
Span 1

Region(ft)



Shear Steel

Flexural Layout



Total No. of Bars - Top : 2.56 ft
3#11 Top
1#11 Left
1#11 Right
3#11 Bottom

Total No. of Bars - Middle : 1.28 f
3#11 Top
1#11 Left
1#11 Right
3#11 Bottom

Total No. of Bars - Bottom : 0 ft
3#11 Top
1#11 Left
1#11 Right
3#11 Bottom

ANCHOR BOLTS DESIGN

SIMPSON Strong-Tie®

Anchor Designer™
Software
Version 3.1.2303.1

Company:	Wildman & Morris	Date:	9/8/2022
Engineer:	B. Miller	Page:	1/6
Project:	SH-60 Helicopter Pedestal		
Address:	405 Maple Street, Suite B-10, Ramona, CA 92065		
Phone:	760-789-3305		
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB_H
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 7.500
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 10.13
C_{min} (inch): 1.88
S_{min} (inch): 4.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 48.00
State: Cracked
Compressive strength, f'_c (psi): 5000
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement present
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: Yes
Ignore 6do requirement: Yes
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 18.00 x 19.00 x 1.50
Yield stress: 36000 psi

Profile type/size: HSS10.750X0.500

Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB8H (1"Ø)

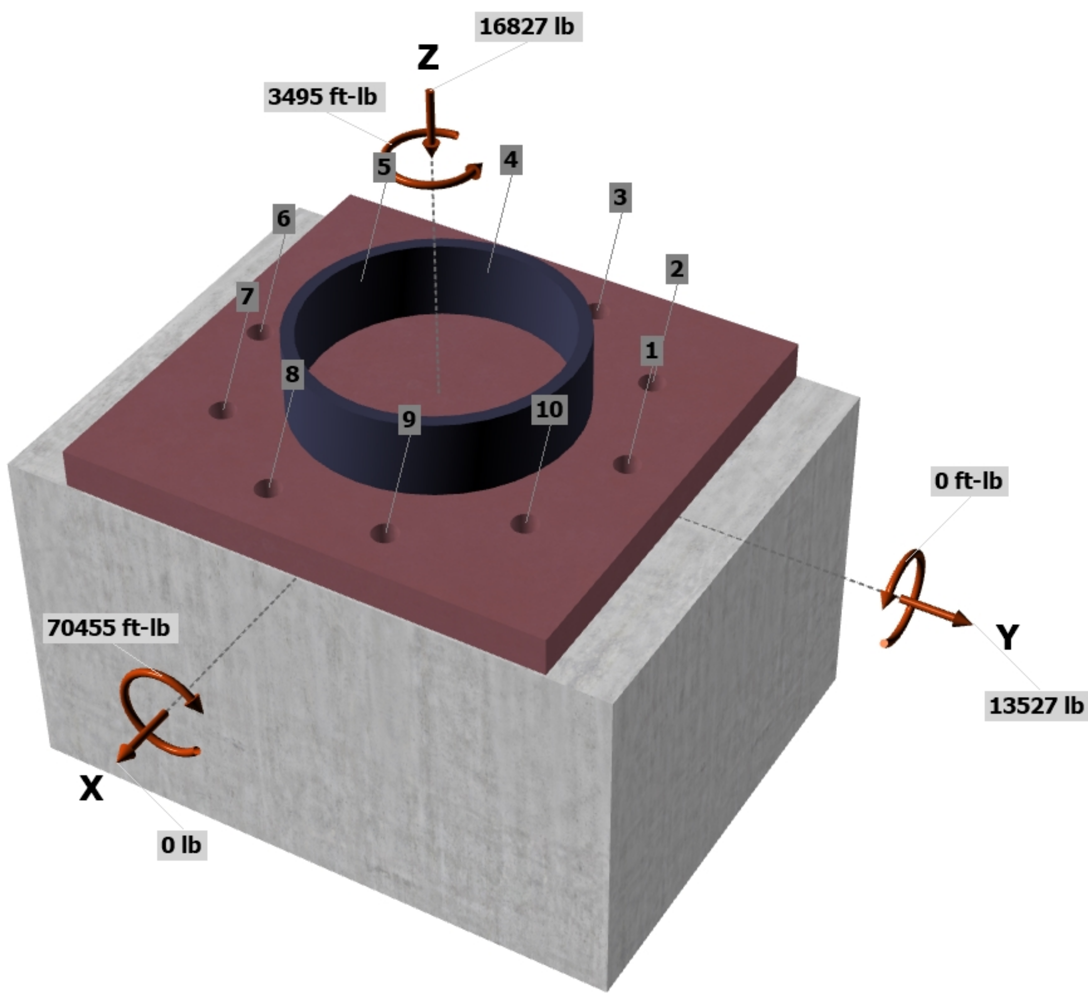
NOTE:
Simpson used to determine anchor bolt forces

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.10.5.3 (a) (iii)-(vi) is satisfied
Ductility section for shear: 17.10.6.3 (a) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

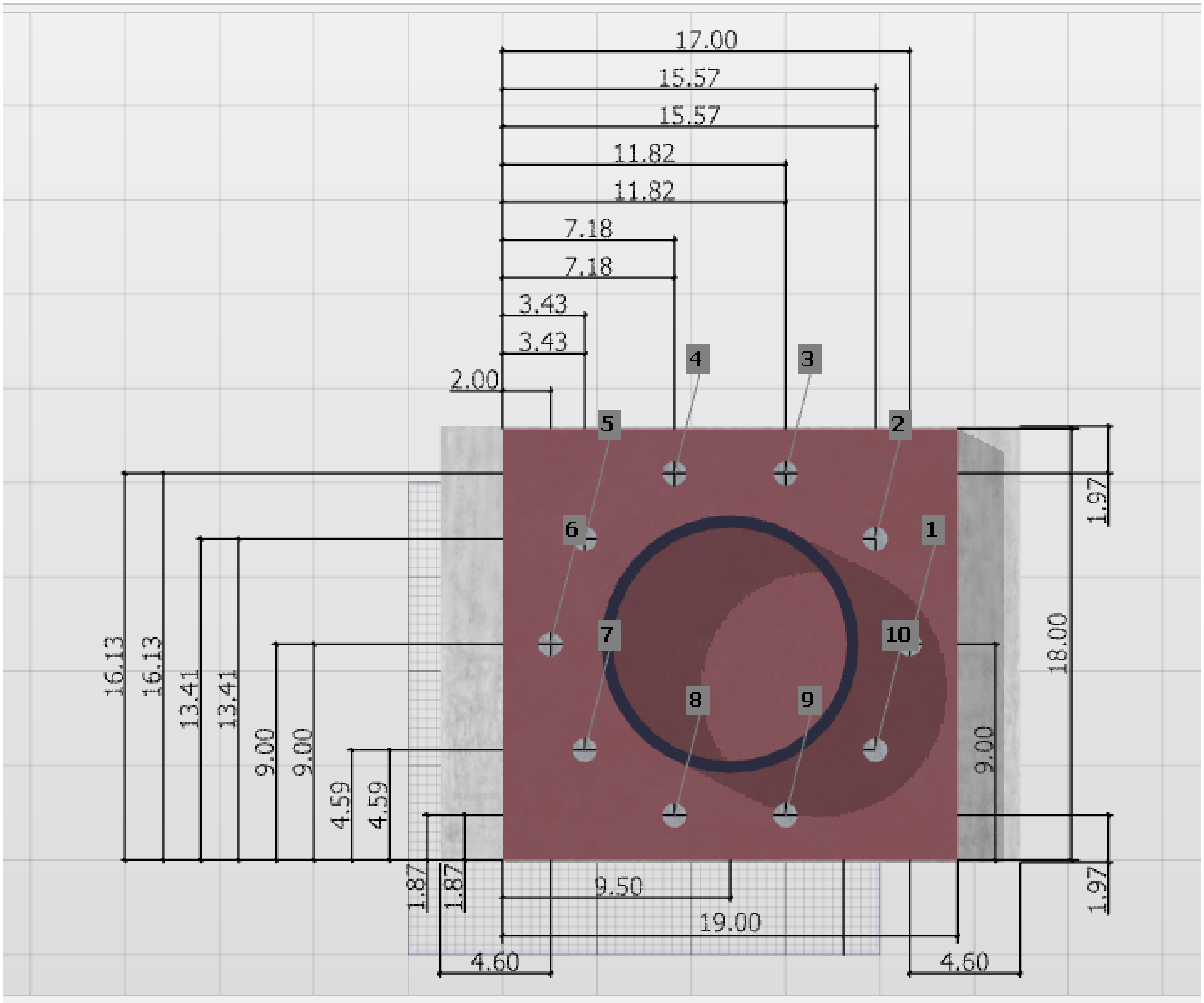
Strength level loads:

N_{ua} [lb]: -16827
V_{uax} [lb]: 0
V_{uay} [lb]: 13527
M_{ux} [ft-lb]: -70455
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 3495

<Figure 1>



<Figure 2>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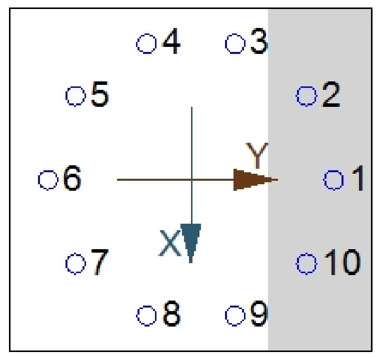
15.57
11.82
11.82
7.18
7.18
3.43
3.43
2.00
1
2
3
4
5
6
7
8
9
10
1.97
18.00
9.00
1.97
16.13
16.13
13.41
13.41
9.00
9.00
4.59
4.59
1.87
1.87
9.50
19.00
4.60
4.60

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	0.0	-559.2	1352.7	1463.7
2	0.0	-452.4	1024.0	1119.5
3	2048.0	-172.8	820.9	838.9
4	7605.6	172.8	820.9	838.9
5	12101.6	452.4	1024.0	1119.5
6	13818.8	559.2	1352.7	1463.7
7	12101.6	452.4	1681.4	1741.2
8	7605.6	172.8	1884.5	1892.4
9	2048.0	-172.8	1884.5	1892.4
10	0.0	-452.4	1681.4	1741.2
Sum	57329.3	0.0	13527.0	14111.4

Maximum concrete compression strain (‰): 0.35
Maximum concrete compression stress (psi): 1505
Resultant tension force (lb): 0
Resultant compression force (lb): 74156
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): 1.55

<Figure 3>



1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB_H
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 7.500
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 10.13
C_{min} (inch): 1.88
S_{min} (inch): 4.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 48.00
State: Cracked
Compressive strength, f'_c (psi): 5000
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement present
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: Yes
Ignore 6do requirement: Yes
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 18.00 x 19.00 x 1.50
Yield stress: 36000 psi

Profile type/size: HSS10.750X0.500

Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB8H (1"Ø)

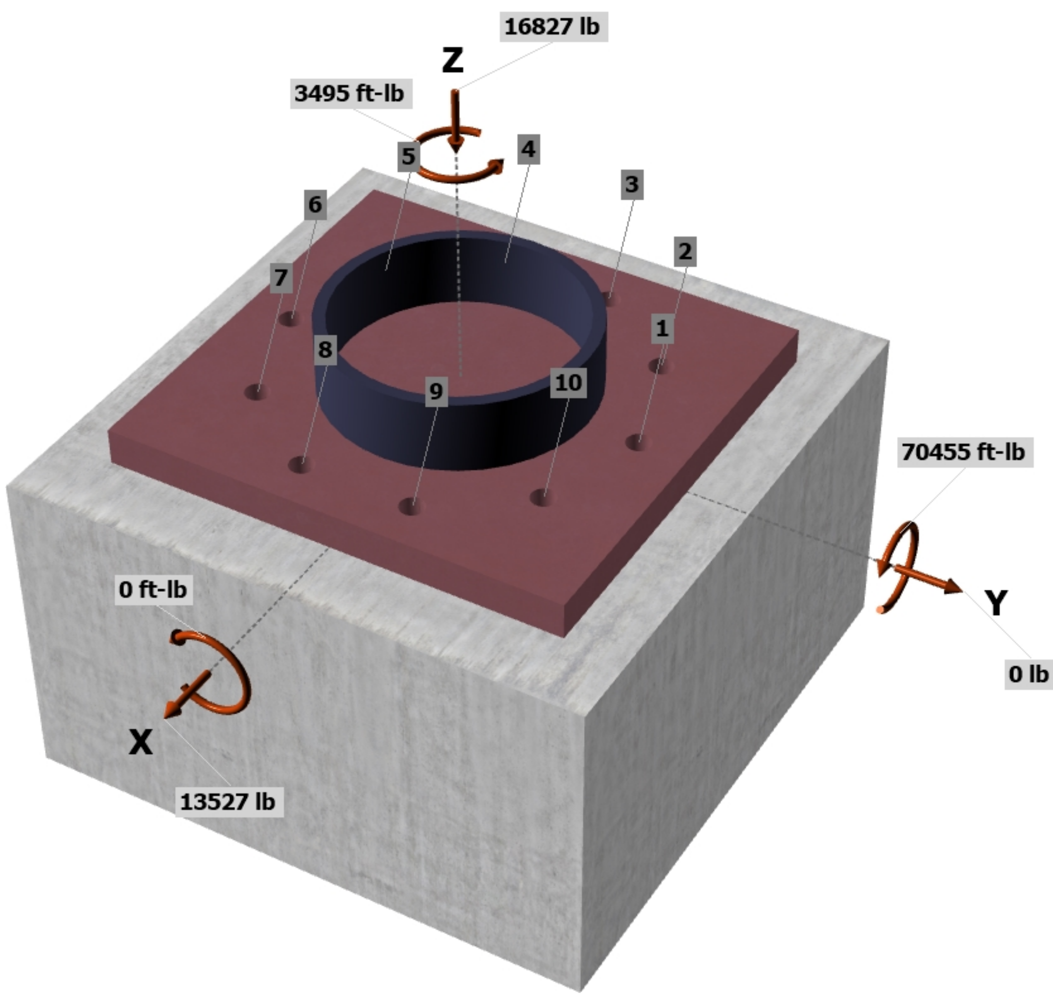
NOTE:
Simpson used to determine anchor bolt forces

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.10.5.3 (a) (iii)-(vi) is satisfied
Ductility section for shear: 17.10.6.3 (a) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

N_{ua} [lb]: -16827
V_{uax} [lb]: 13527
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 70455
M_{uz} [ft-lb]: 3495

<Figure 1>



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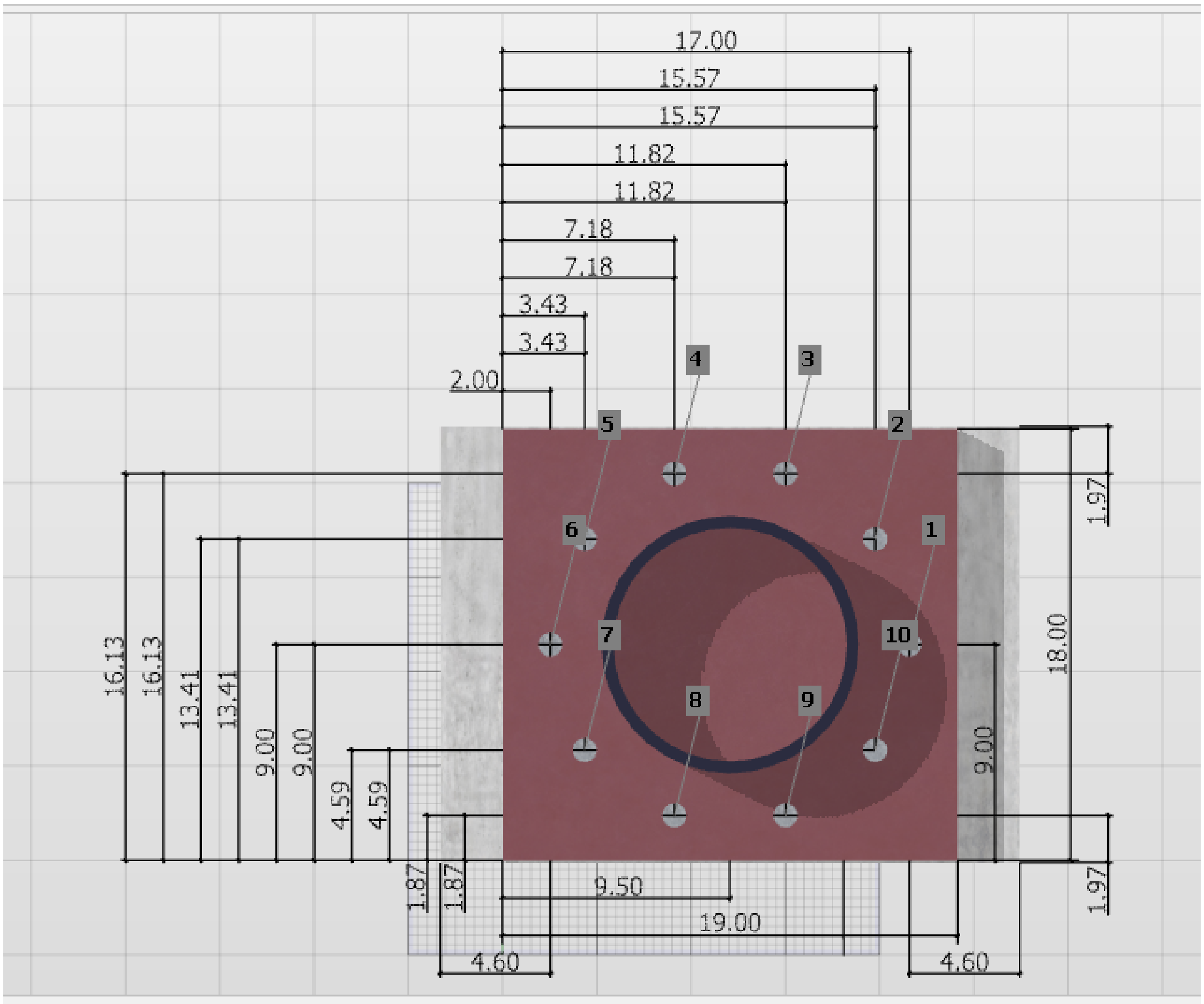
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15.57
15.57
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11.82
7.18
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3.43
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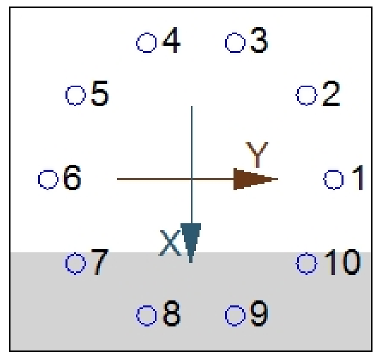
16.13
16.13
13.41
13.41
9.00
9.00
4.59
4.59
1.87
1.87
9.50
19.00
4.60
4.60
1.97
18.00
9.00
1.97

3. Resulting Anchor Forces

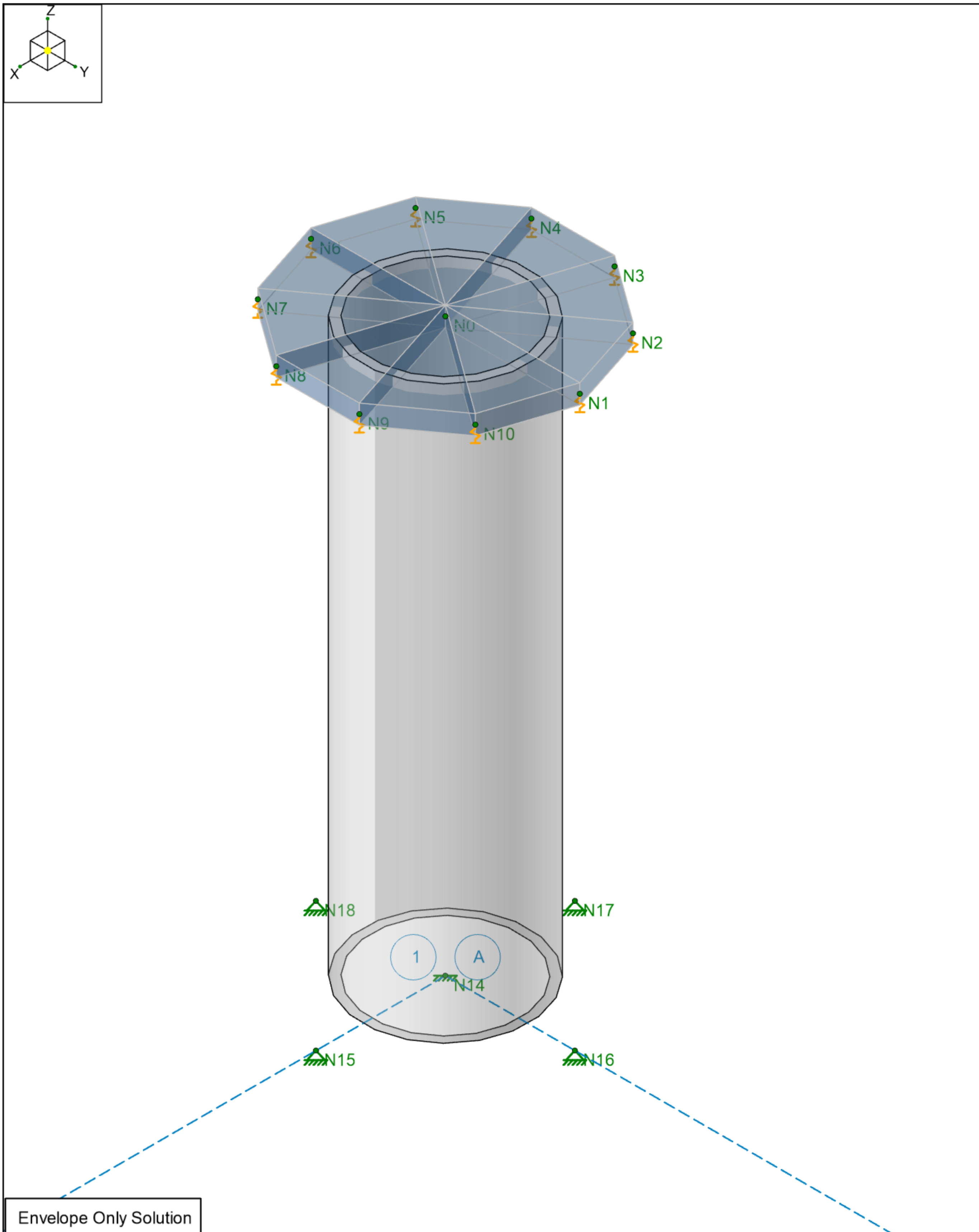
Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	4891.2	793.5	0.0	793.5
2	10567.2	900.3	-328.7	958.4
3	14075.6	1179.9	-531.8	1294.2
4	14075.6	1525.5	-531.8	1615.5
5	10567.2	1805.1	-328.7	1834.8
6	4891.2	1911.9	0.0	1911.9
7	0.0	1805.1	328.7	1834.8
8	0.0	1525.5	531.8	1615.5
9	0.0	1179.9	531.8	1294.2
10	0.0	900.3	328.7	958.4
Sum	59067.9	13527.0	0.0	14111.3

Maximum concrete compression strain (‰): 0.35
Maximum concrete compression stress (psi): 1536
Resultant tension force (lb): 0
Resultant compression force (lb): 75895
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): 1.55
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>

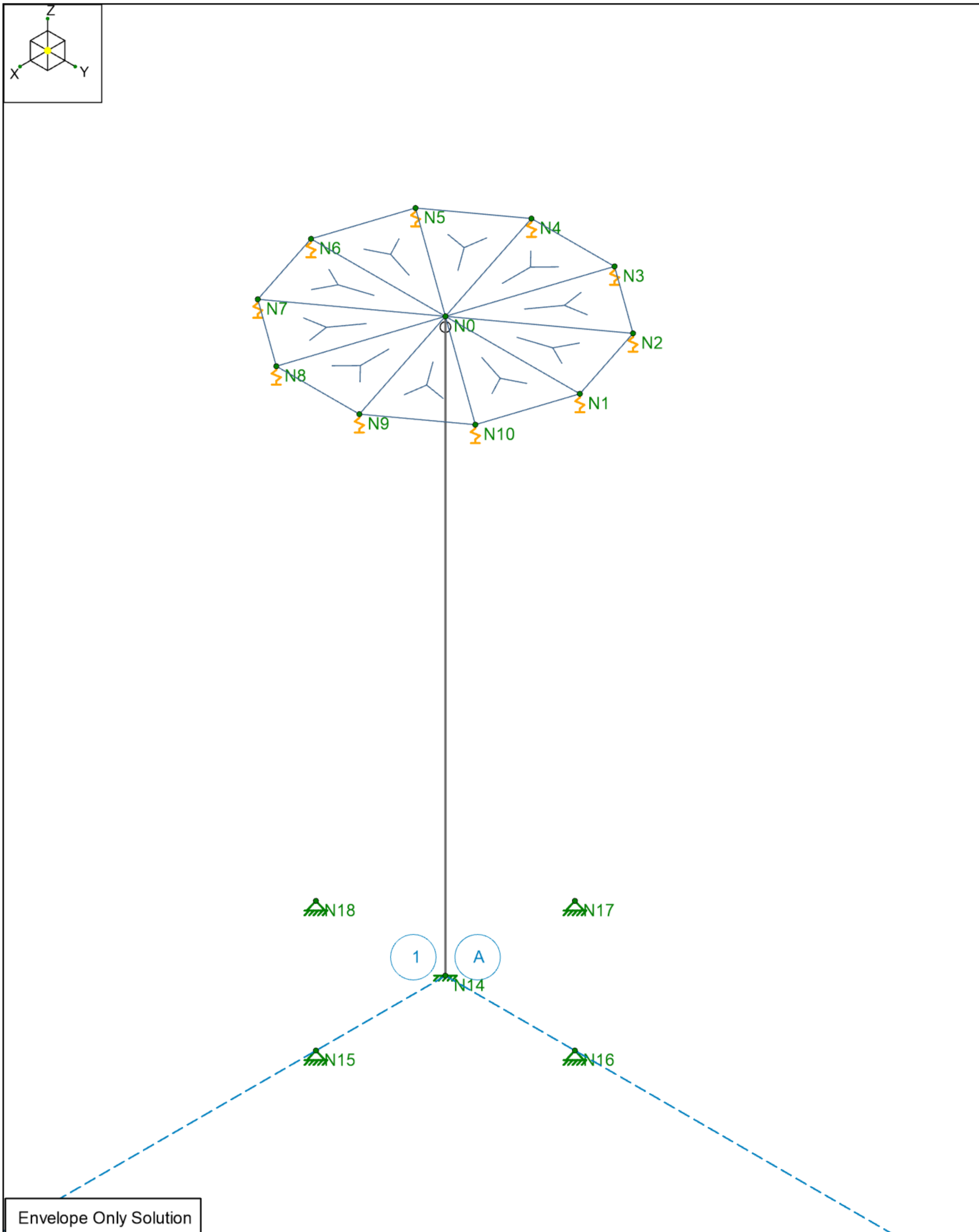


RISA TORSION
ANALYSIS & DESIGN OF
EMBEDDED PIPE



Envelope Only Solution

RISA A NEMETSCHEK COMPANY	Wildman & Morris	SH-60 Helicopter Pedestal	SK-6
	nicole.caudana		Oct 18, 2023 at 03:26 PM
	23026		SH-60 Helicopter Pipe_Des...



Envelope Only Solution

RISA A NEMETSCHEK COMPANY	Wildman & Morris	SH-60 Helicopter Pedestal	SK-7
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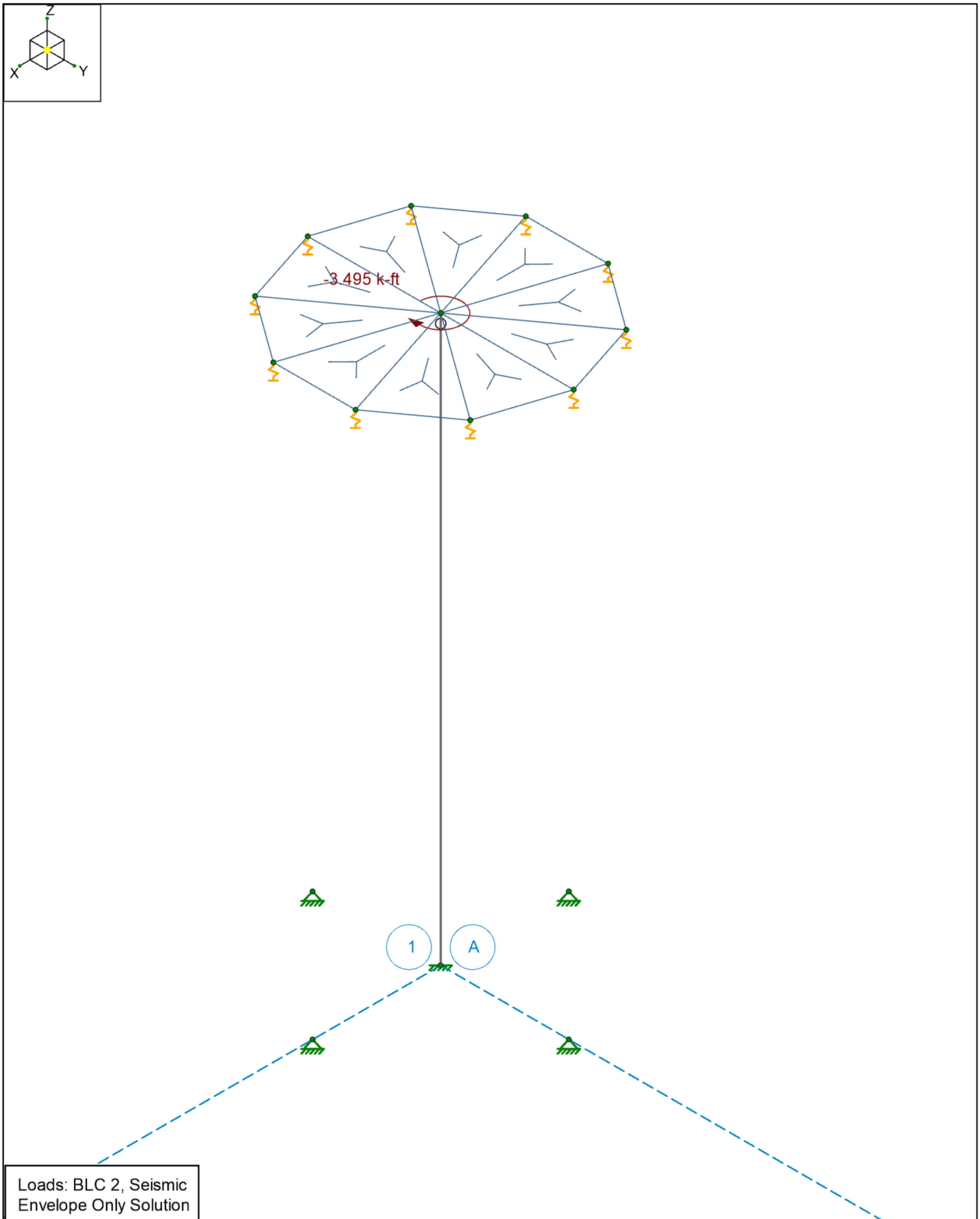

Loads: BLC 2, Seismic
Envelope Only Solution

RISA A NEMETSCHEK COMPANY	Wildman & Morris	SH-60 Helicopter Pedestal	SK-4
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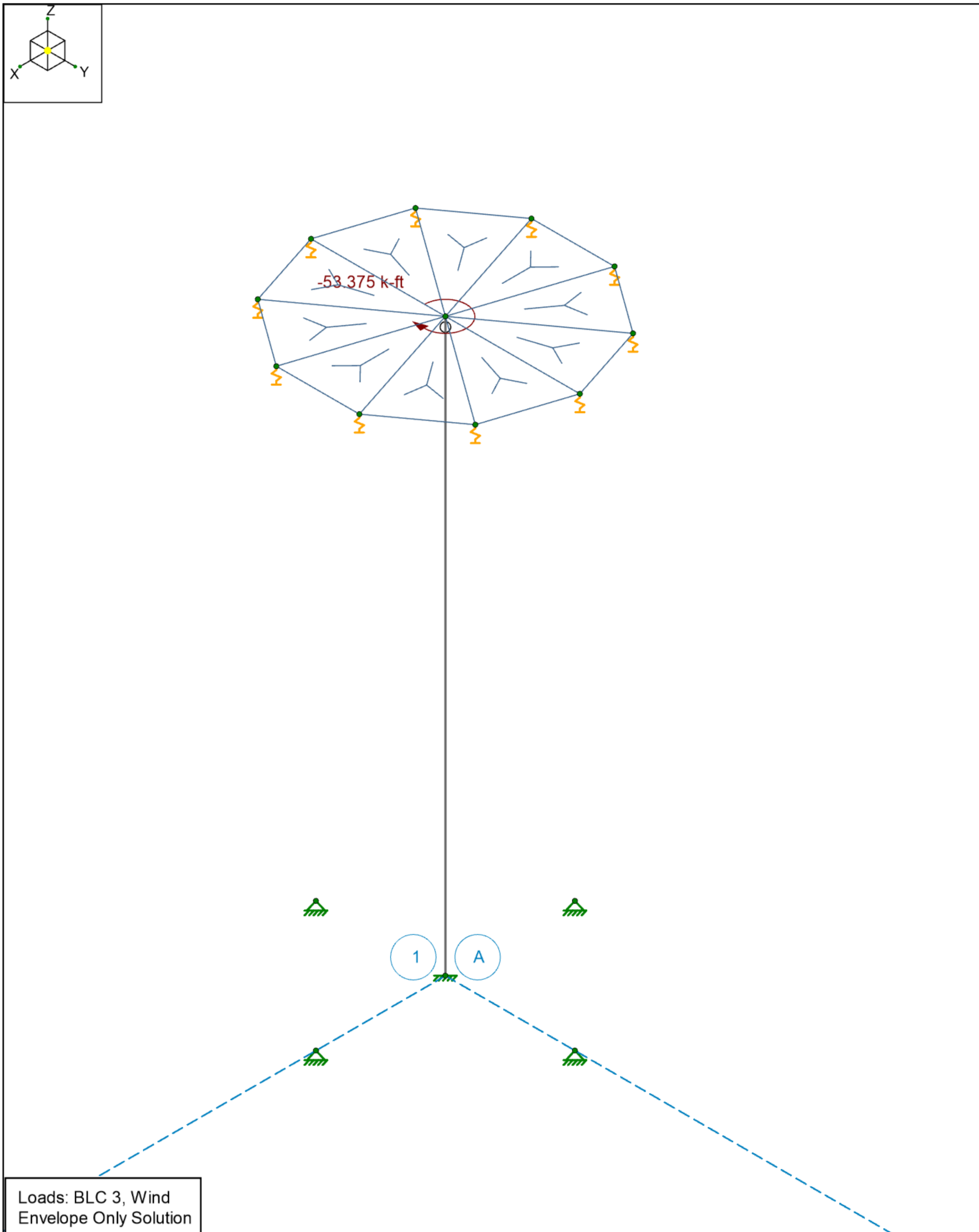


Loads: BLC 3, Wind
Envelope Only Solution

RISA A NEMETSCHEK COMPANY	Wildman & Morris	SH-60 Helicopter Pedestal	SK-5
	nicole.caudana		Oct 18, 2023 at 03:22 PM
	23026		SH-60 Helicopter Pipe_Des...

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N0	0	-0.00025	2.479333	
2	N14	0	-0.000286	0.000286	
3	N9	0.554562	0.18058	2.479333	
4	N10	0.342361	0.472008	2.479333	
5	N1	-0.000611	0.58305	2.479333	
6	N2	-0.343349	0.47129	2.479333	
7	N3	-0.55494	0.179419	2.479333	
8	N4	-0.554562	-0.18108	2.479333	
9	N5	-0.342361	-0.472508	2.479333	
10	N6	0.000611	-0.58355	2.479333	
11	N7	0.343349	-0.47179	2.479333	
12	N8	0.55494	-0.179919	2.479333	
13	N15	0.5625	-0.000286	0.000286	
14	N16	0	0.562214	0.000286	
15	N17	-0.5625	-0.000286	0.000286	
16	N18	0	-0.562786	0.000286	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]	Y Rot [k-ft/rad]	Z Rot [k-ft/rad]
1	N9			CS1e+7			
2	N8			CS1e+7			
3	N5			CS1e+7			
4	N6			CS1e+7			
5	N7			CS1e+7			
6	N3			CS1e+7			
7	N4			CS1e+7			
8	N2			CS1e+7			
9	N1			CS1e+7			
10	N10			CS1e+7			
11	N14	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
12	N15	Reaction	Reaction	Reaction			
13	N16	Reaction	Reaction	Reaction			
14	N17	Reaction	Reaction	Reaction			
15	N18	Reaction	Reaction	Reaction			

Node Loads and Enforced Displacements (BLC 2 : Seismic)

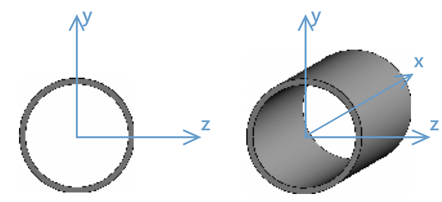
	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]
1	N0	L	MZ	-3.495

Node Loads and Enforced Displacements (BLC 3 : Wind)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]
1	N0	L	MZ	-53.375

Detail Report: M7

Load Combination: Envelope **Code check:** 0.838 (LC 8)



Input Data

Shape:	HSS8.625X0.500	**I Node:**	N14
Member Type:	Column	**J Node:**	N0
Length (ft):	2.479	**I Release:**	Fixed
Material Type:	Hot Rolled Steel	**J Release:**	BenPIN
Design Rule:	Typical	**I Offset:**	N/A
Internal Sections:	97	**J Offset:**	N/A
Design Code:	AISC 15th (360-16): ASD	**T/C Only:**	Both Way

Material Properties

Material:	A500 Gr.C RND	**Therm. Coeff. (/1E5 F):**	0.65	**F_u (ksi):**	62
E (ksi):	29000	**Density (k/ft^3):**	0.527	**R_t:**	1.3
G (ksi):	11154	**F_y (ksi):**	46		
Nu:	0.3	**R_y:**	1.4		

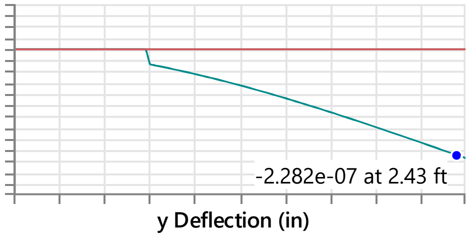
Shape Properties

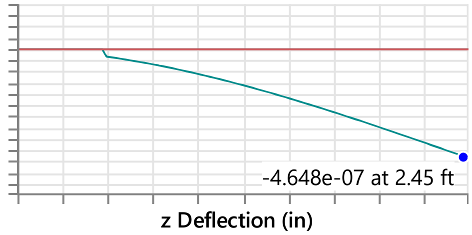
d (in):	8.63	**Area (in^2):**	11.9	**I_{zz} (in^4):**	100
t (in):	0.465	**J (in^4):**	199		
Z (in^3):	31	**I_{yy} (in^4):**	100		

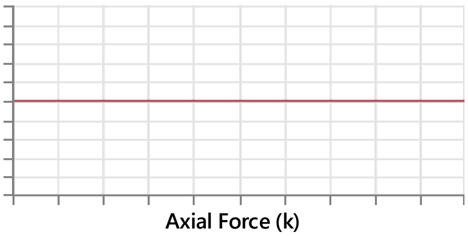
Design Properties

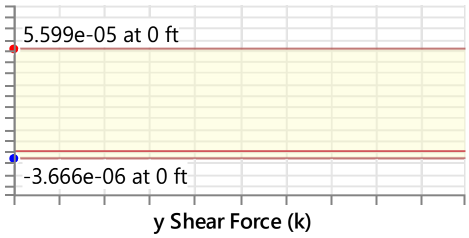
$L_{b\ y\text{-}y}$ (ft):	2.479	**$K_{y\text{-}y}$:**	1	**Seismic DR:**	None
$L_{b\ z\text{-}z}$ (ft):	2.479	**$K_{z\text{-}z}$:**	1	**Max Defl Ratio:**	L/10000
$L_{comp\ top}$:	Lbyy	**y sway:**	No	**Max Defl Location:**	0
$L_{comp\ bot}$ (ft):	2.479	**z sway:**	No	**Span:**	N/A
L_{torque} (ft):	2.479	**Function:**	Lateral	**τ_b:**	1

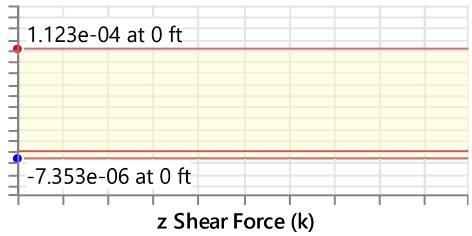
Diagrams:

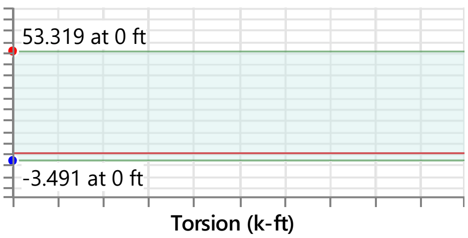


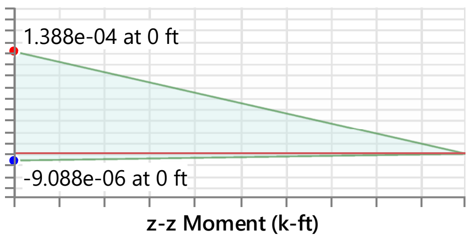


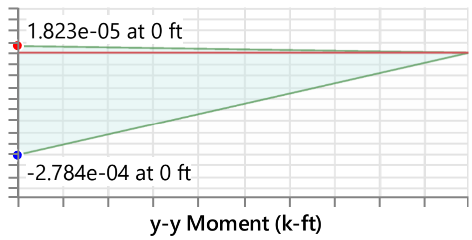


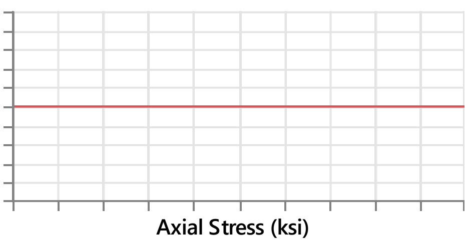


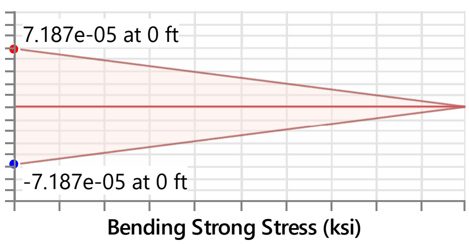


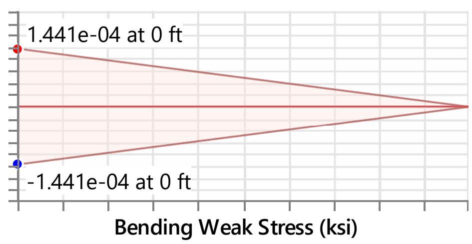








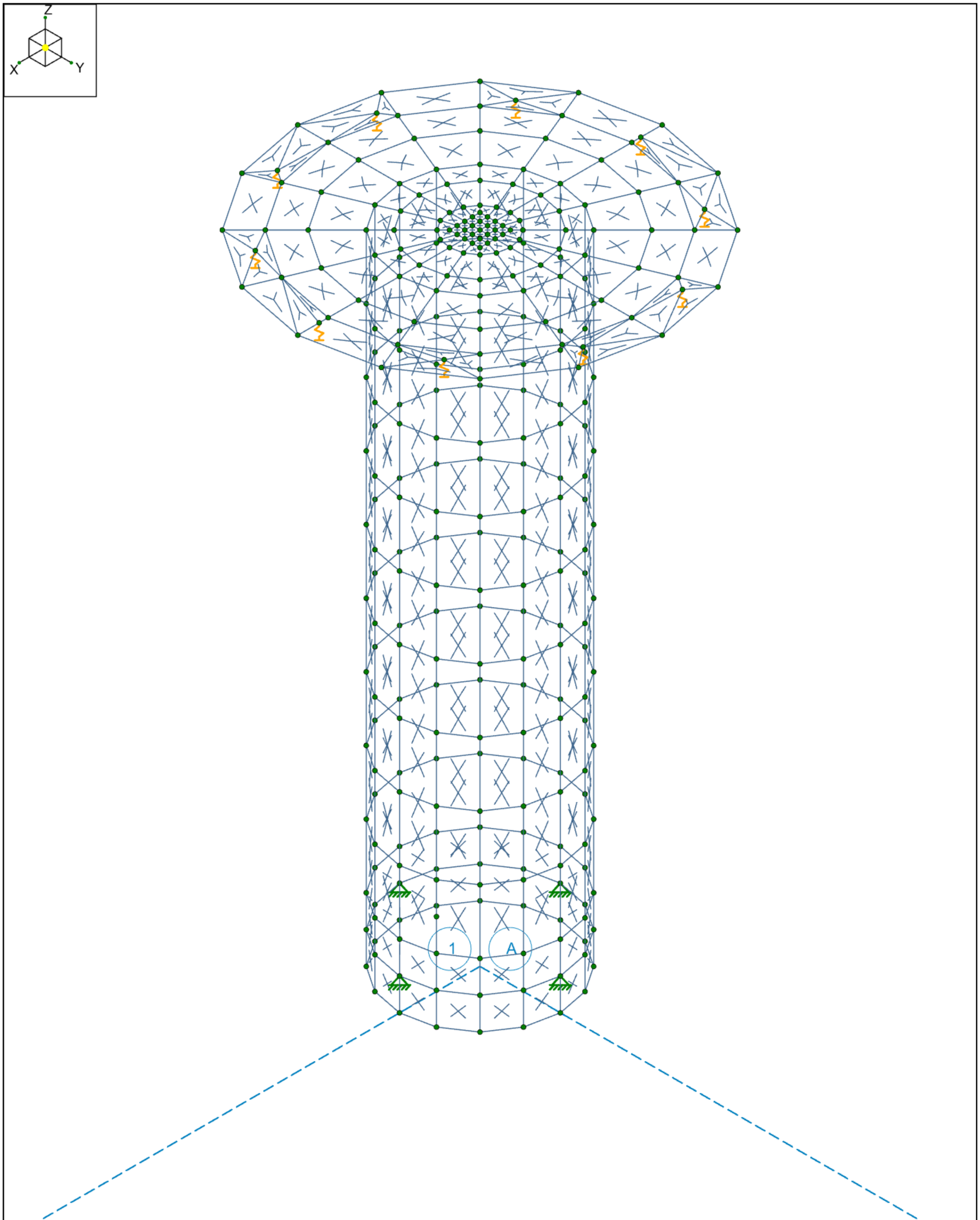




AISC 15th (360-16): ASD Code Check

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	8	-	-	-	-
Applied Loading - Shear + Torsion	8	-	-	-	-
Axial Tension Analysis	8	0 k	327.784 k	-	-
Axial Compression Analysis	8	0 k	325.471 k	-	-
Flexural Analysis	8	0.0003111 k-ft	71.158 k-ft	-	-
Shear Analysis	8	82.365 k	98.335 k	**0.838**	**PASS**
Bending & Axial Interaction Check (UC Bending Max)	8	-	-	**0.632**	**PASS**
Torsional Analysis	8	53.319 k-ft	67.065 k-ft	**0.795**	**PASS**

RISA TORSION
ANALYSIS & DESIGN OF
SHEAR STUDS



RISA A NEMETSCHEK COMPANY	Wildman & Morris	SH-60 Helicopter Pedestal	SK-1
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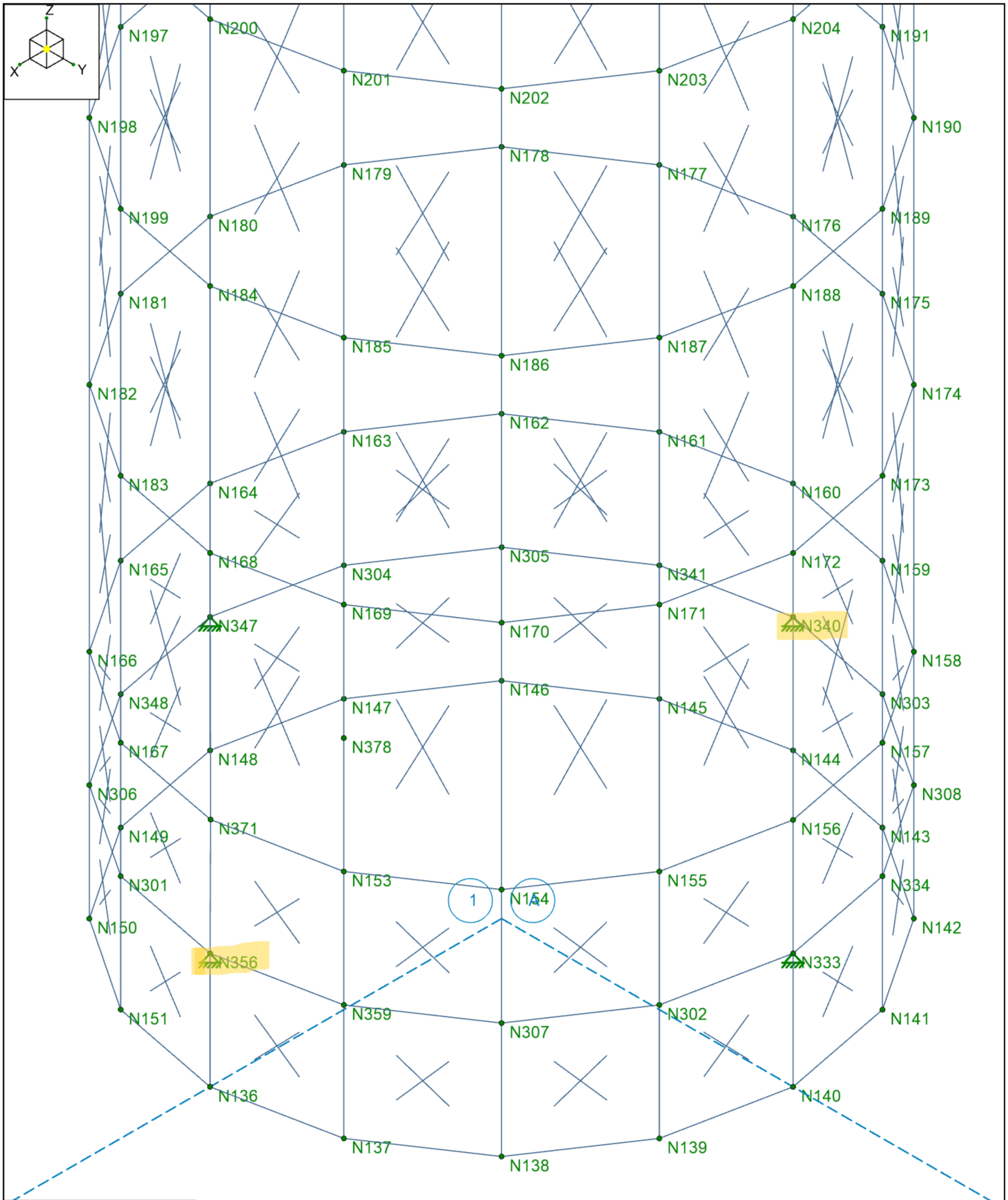
	23026		SH-60 Helicopter Pipe_1016...



Loads: BLC 2, Seismic
Envelope Only Solution

RISA	Wildman & Morris	SH-60 Helicopter Pedestal	SK-1
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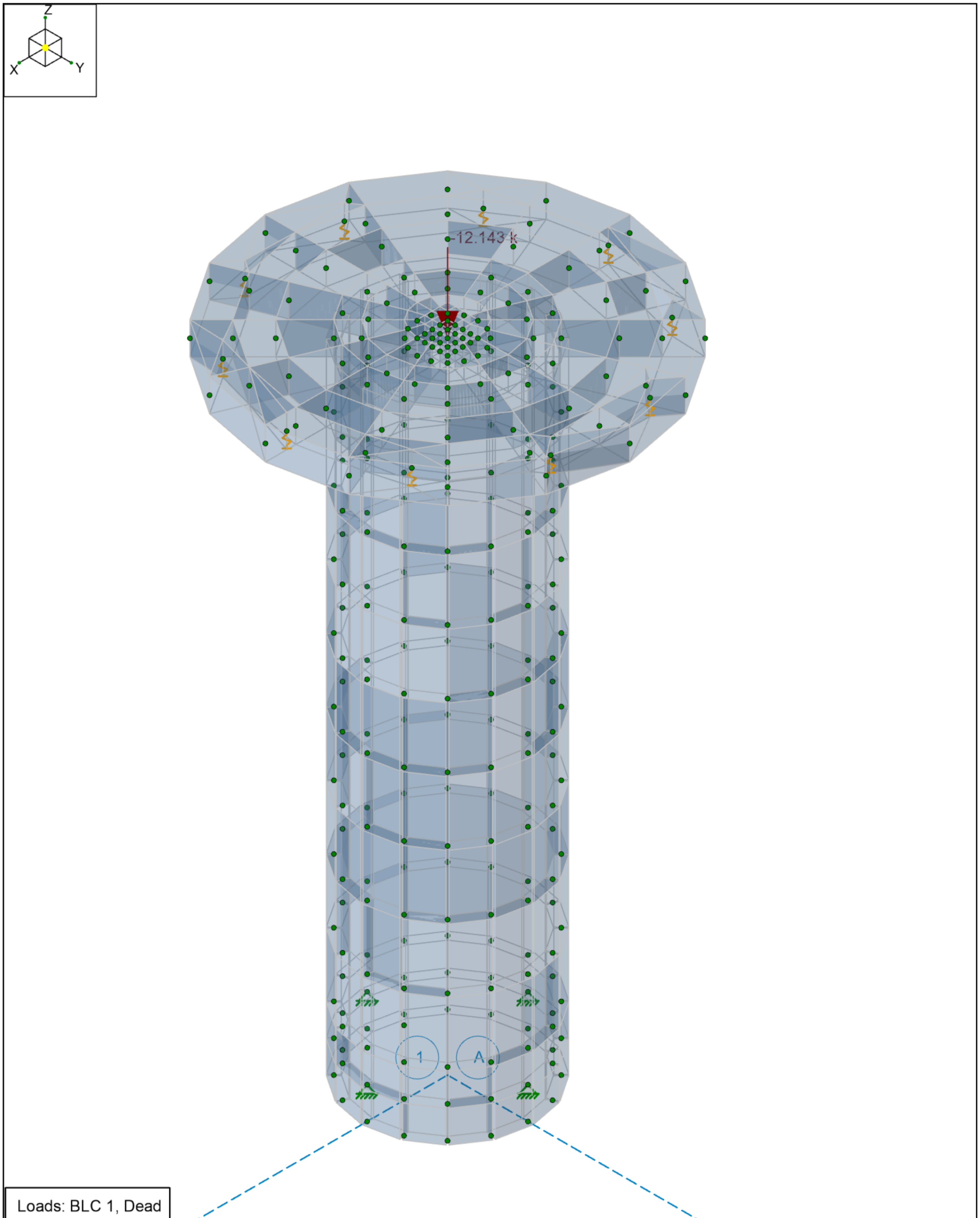
	23026		SH-60 Helicopter Pipe_1016...



Loads: BLC 1, Dead

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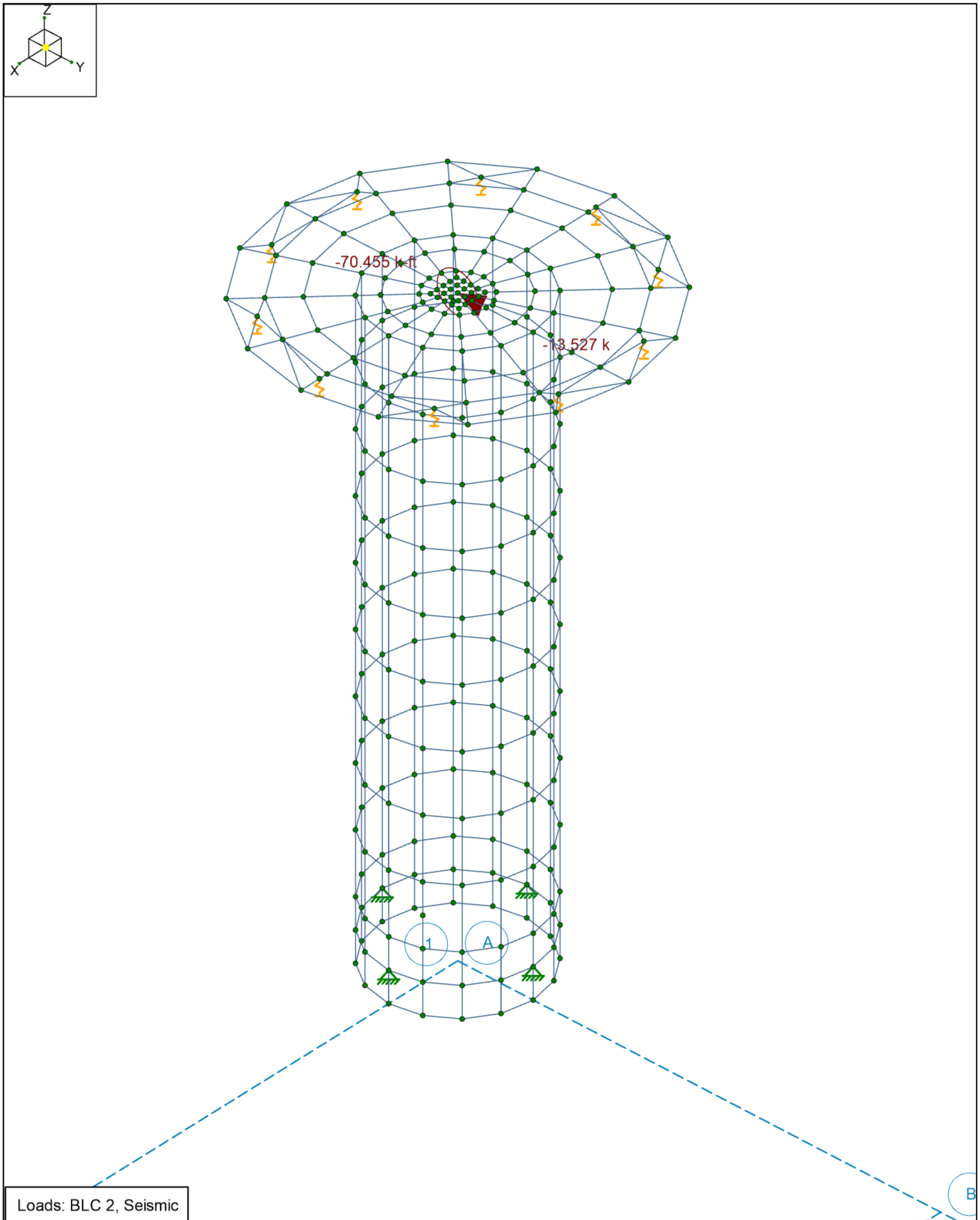

Loads: BLC 2, Seismic

RISA A NEMETSCHEK COMPANY	Wildman & Morris	SH-60 Helicopter Pedestal	SK-4
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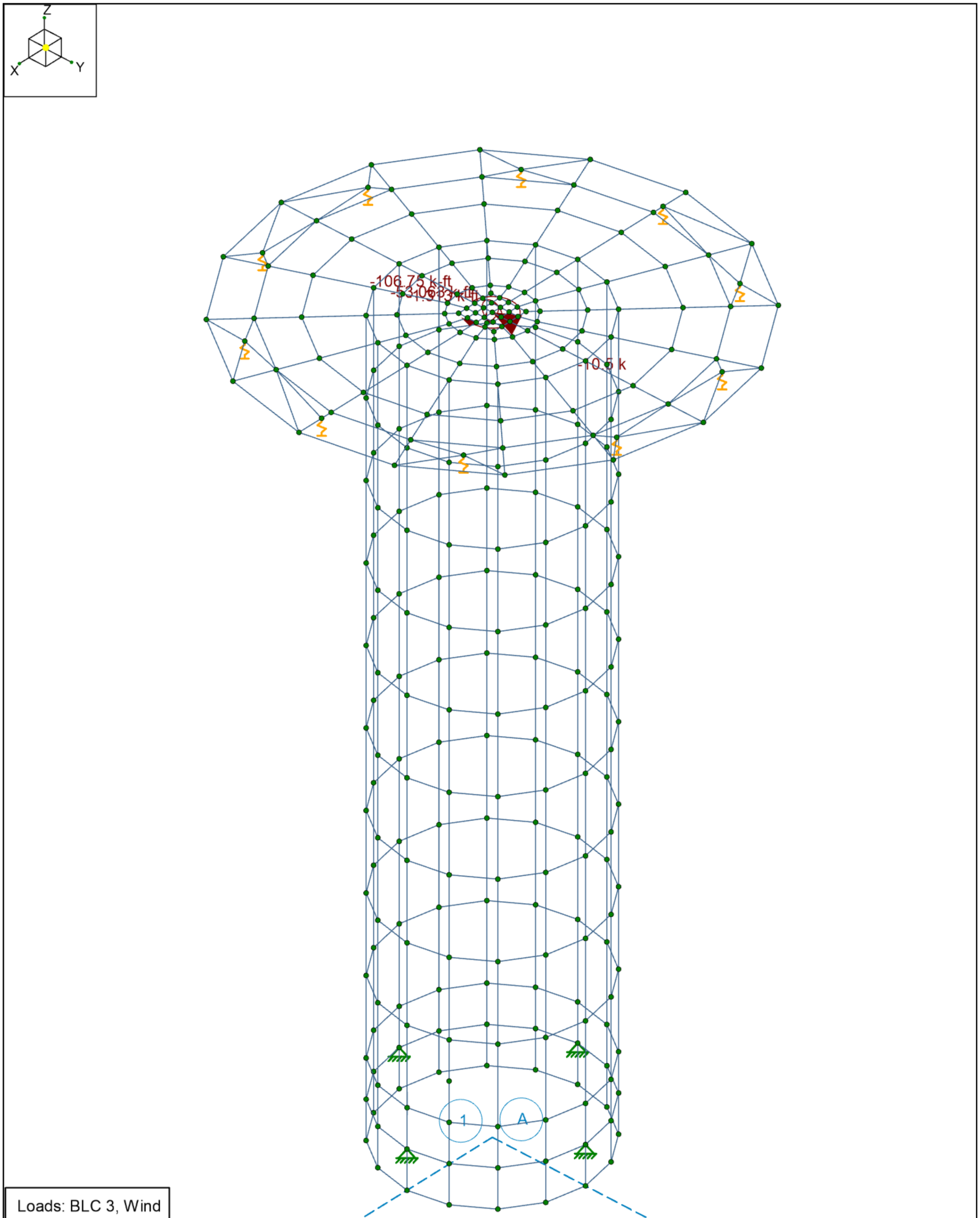


Loads: BLC 3, Wind

RISA	Wildman & Morris	SH-60 Helicopter Pedestal	SK-5
	nicole.caudana		Oct 23, 2023 at 02:00 PM
	23026		SH-60 Helicopter Pipe_1016...

Basic Load Cases

	BLC Description	Category	Nodal
1	Dead	DL	1
2	Seismic	EL	2
3	Wind	WL	25

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	Deflection 1	Yes	Y	DL	1								
2	Deflection 2	Yes	Y	LL	1								
3	Deflection 3	Yes	Y	DL	1	LL	1						
4	IBC 21/ASCE Strength 1	Yes	Y	DL	1.4								
5	IBC 21/ASCE Strength 2 (a)	Yes	Y	DL	1.2	LL	1.6	LLS	1.6				
6	IBC 21/ASCE Strength 3 (b)	Yes	Y	DL	1.2	WL	0.5						
7	IBC 21/ASCE Strength 4 (a)	Yes	Y	DL	1.2	WL	1	LL	0.5	LLS	1		
8	IBC 21/ASCE Strength 5	Yes	Y	DL	0.9	WL	1						
9	IBC 21/ASCE Strength 6 (a)	Yes	Y	DL	1.2	Sds*DL	0.2	EL	1	LL	0.5	LLS	1
10	IBC 21/ASCE Strength 6 (b)	Yes	Y	DL	1.2	Sds*DL	0.2	EL	-1	LL	0.5	LLS	1
11	IBC 21/ASCE Strength 7 (a)	Yes	Y	DL	0.9	Sds*DL	-0.2	EL	1				
12	IBC 21/ASCE Strength 7 (b)	Yes	Y	DL	0.9	Sds*DL	-0.2	EL	-1				

Node Loads and Enforced Displacements (BLC 1 : Dead)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]
1	N2	L	Z	-12.143

Node Loads and Enforced Displacements (BLC 2 : Seismic)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]
1	N2	L	Y	-13.527
2	N2	L	MX	-70.455

Node Loads and Enforced Displacements (BLC 3 : Wind)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]
1	N7	L	Y	3.336
2	N9	L	Y	3.336
3	N11	L	Y	3.336
4	N12	L	Y	3.336
5	N4	L	Y	3.336
6	N6	L	Y	3.336
7	N13	L	Y	3.336
8	N14	L	Y	3.336
9	N7	L	Y	-3.336
10	N9	L	Y	-3.336
11	N11	L	Y	-3.336
12	N12	L	Y	-3.336
13	N4	L	Y	-3.336
14	N6	L	Y	-3.336
15	N13	L	Y	-3.336
16	N14	L	Y	-3.336
17	N11	L	Z	0
18	N12	L	Z	0
19	N13	L	Z	0

Node Loads and Enforced Displacements (BLC 3 : Wind) (Continued)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]
20	N14	L	Z	0
21	N2	L	MZ	-53.375
22	N2	L	MZ	-53.375
23	N2	L	MX	-53.063
24	N2	L	MY	-1.313
25	N2	L	Y	-10.5

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
0	N5	max	0	12	0	12	0.367	4	0	12	0	12	0	12
1		min	0	1	0	1	0	2	0	1	0	1	0	1
2	N14	max	0	12	0	12	8.863	10	0	12	0	12	0	12
3		min	0	1	0	1	0	2	0	1	0	1	0	1
4	N11	max	0	12	0	12	8.917	10	0	12	0	12	0	12
5		min	0	1	0	1	0	2	0	1	0	1	0	1
6	N12	max	0	12	0	12	17.273	10	0	12	0	12	0	12
7		min	0	1	0	1	0	2	0	1	0	1	0	1
8	N13	max	0	12	0	12	17.359	10	0	12	0	12	0	12
9		min	0	1	0	1	0	2	0	1	0	1	0	1
10	N9	max	0	12	0	12	8.725	9	0	12	0	12	0	12
11		min	0	1	0	1	0	2	0	1	0	1	0	1
12	N10	max	0	12	0	12	0.35	4	0	12	0	12	0	12
13		min	0	1	0	1	0	2	0	1	0	1	0	1
14	N7	max	0	12	0	12	17.191	9	0	12	0	12	0	12
15		min	0	1	0	1	0	2	0	1	0	1	0	1
16	N6	max	0	12	0	12	17.469	11	0	12	0	12	0	12
17		min	0	1	0	1	0	2	0	1	0	1	0	1
18	N4	max	0	12	0	12	8.772	9	0	12	0	12	0	12
19		min	0	1	0	1	0	2	0	1	0	1	0	1
20	N356	max	0.473	12	6.416	11	3.369	4	0	12	0	12	0	12
21		min	-0.151	4	-6.41	12	-11.106	11	0	1	0	1	0	1
22	N333	max	0.001	4	0.848	11	9.853	9	0	12	0	12	0	12
23		min	-0.276	8	-0.151	4	-29.248	12	0	1	0	1	0	1
24	N340	max	0.15	4	6.401	11	3.375	4	0	12	0	12	0	12
25		min	-0.468	12	-6.395	12	-11.064	12	0	1	0	1	0	1
26	N347	max	0.215	8	0.15	4	9.649	10	0	12	0	12	0	12
27		min	-0.005	9	-0.859	12	-29.32	11	0	1	0	1	0	1
28	Totals:	max	0	11	13.527	11	17	9						
29		min	0	12	-13.527	12	0	2						

Frequencies and Participation

No Data to Print...

Check embedments in foundation with shear carried by studs & tension by bolts:

$D_{pipe} := 8.75\ in$

Governing shear: $V_{bt} := \dfrac{M_{wind\_torsion}}{D_{pipe}}$

Concrete Strength: $f'_c := 5\ ksi$

$\lambda_a := 1$ NWC

Check shear carried by studs:

Stud properties: $w_c := 145\ pcf$ $E_c := \left(\dfrac{w_c}{pcf}\right)^{1.5} \cdot 33 \cdot \sqrt{\dfrac{f'_c}{ksi}}\ ksi = 128840.1\ ksi$

3/4" dia. studs: $F_{us} := 65.0 \cdot ksi$ $d_a := 0.75 \cdot in$

$A_{ss} := \pi \cdot \left(\dfrac{d_a}{2}\right)^2 = 0.442\ in^2$

Per Table 17.5.3(b): $\phi_s := 0.75$

$f_{ys} := 36.0 \cdot ksi$ $f_{uts} := min\left(1.9 \cdot f_{ys}, 860 \cdot MPa, F_{us}\right)$ $f_{uts} = 65.0\ ksi$

Embedment depth: $h_{efs} := 3.0 \cdot in$

$V_s := V_{bt} = 73.2\ kip$ $V_s = 73.2\ kip$

$n_{smin} := \dfrac{V_s}{A_{ss} \cdot F_{us}}$ $n_{smin} = 2.5$

Try (4 total) $n_s := 4$ $c_{s1} := 6 \cdot in$

Steel strength of anchor:

Effective area: $A_{se} := A_{ss}$ $c_{s2} := 6 \cdot in$

$V_{sa} := n_s \cdot A_{se} \cdot f_{uts}$ $V_{sa} = 114.9\ kip$ (Eq 17.7.1.2a, ACI318-19)

Shear Strength of Steel Headed Stud Anchors:

$Q := 0.5 \cdot A_{ss} \cdot \sqrt{\left(f'_c \cdot E_c\right)} = 177.29\ kip$ (AISC 360-16 I8.2a, EQ. I8-1)

$R_g := 1.0$ $R_p := 0.75$ $\square \le \square$

$R_g \cdot R_p \cdot A_{ss} \cdot F_{us} = 21.537\ kip$

$Q := R_g \cdot R_p \cdot A_{ss} \cdot F_{us} = 21.537\ kip$

Check breakout strength:

$\psi_{ecV} := 1$ (no eccentricity)

(Eq 17.7.2.3.1, ACI318-19)

$\psi_{edV} := \text{if}\left(c_{s2} < 1.5 \cdot c_{s1}, 0.7 + 0.3 \cdot \dfrac{c_{s2}}{1.5 \cdot c_{s1}}, 1.0\right)$ $\psi_{edV} = 0.9$ (Eq 17.7.2.4.1a, ACI318-19)

$\psi_{cV} := 1.4$ (Table 17.7.2.5.1,ACI318-19)

$A_{Vo} := 4.5 \cdot {c_{s1}}^2$ $A_{Vo} = 162.0\ in^2$ (Fig.R17.7.2.1a, ACI318-19)

$l_e := h_{efs} = 3\ in$

$$V_b := min\left(\left(\left(8 \cdot \left(\dfrac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{\dfrac{d_a}{in}}\right) \cdot \sqrt{\dfrac{f'_c}{ksi}} \cdot \left(\dfrac{c_{s1}}{in}\right)^{1.5} \cdot kip\right), \left(9 \cdot \lambda_a \cdot \sqrt{\dfrac{f'_c}{ksi}} \cdot \left(\dfrac{c_{s1}}{in}\right)^{1.5} \cdot kip\right)\right)$$

(Eq 17.7.2.2.2, ACI318-19)

$V_b = 295.8\ kip$

Stud spacing: $s_1 := 8 \cdot in$

Concrete thickness: $h := 36 \cdot in$

$A_V := \left(2 \cdot \left(1.5 \cdot c_{s1}\right) + s_1\right) \cdot h$ $A_V = 936.0\ in^2$ (Fig. R17.7.2.1b, ACI318-19)

$V_{cbg} := \frac{A_V}{A_{Vo}} \cdot \psi_{ecV} \cdot \psi_{edV} \cdot \psi_{cV} \cdot V_b$ $V_{cbg} = 2153.2\ kip$ (Eq 17.7.2.1b, ACI318-19)

Pryout strength:

$A_N := \left(c_{s1} + 1.5 \cdot h_{efs}\right) \cdot \left(2 \cdot 1.5 \cdot h_{efs}\right) = 0.656\ ft^2$ (Fig. R17.6.2.1, ACI318-19)

$k_{cp} := 2.0$

$e'_N := 6\ in$

$\psi_{ecN} := \frac{1}{\left(1 + \frac{e'_N}{1.5 \cdot h_{efs}}\right)} = 0.429$ (Eq 17.6.2.3.1, ACI318-19)

$\psi_{edN} := 1.0$ (Eq 17.6.2.4.1a, ACI318-19)

$\psi_{cN} := 1.25$ (17.6.2.5.1, ACI318-19)

$\psi_{cpN} := 1.0$ (Eq 17.6.2.4.1a, ACI318-19)

$k_c := 24$

$N_b := k_c \cdot \lambda_a \cdot \sqrt{\frac{f'_c}{ksi}} \cdot \left(\frac{h_{efs}}{in}\right)^{1.5} \cdot kip = 278.855\ kip$

$N_{cbg} := \frac{A_N}{A_{Vo}} \cdot \psi_{ecN} \cdot \psi_{edN} \cdot \psi_{cN} \cdot \psi_{cpN} \cdot N_b$ (Eq 17.6.2.1b, ACI318-19)

$V_{cpg} := k_{cp} \cdot N_{cbg}$ $V_{cpg} = 174.3\ kip$ (Eq 17.7.3.1b, ACI318-19)

$\phi Vn := min\left(\phi_s \cdot V_{sa}, \phi_s \cdot V_{cbg}, \phi_s \cdot V_{cpg}\right)$ $\phi Vn = 86.1\ kip$

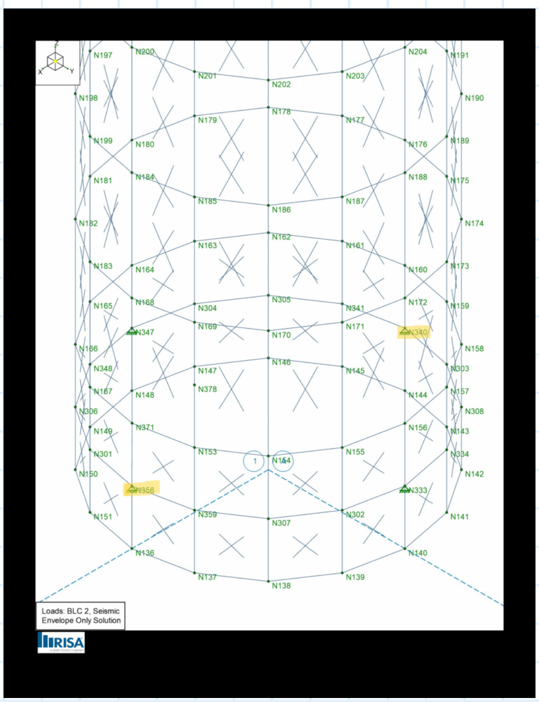
$V_u := V_{bt}$

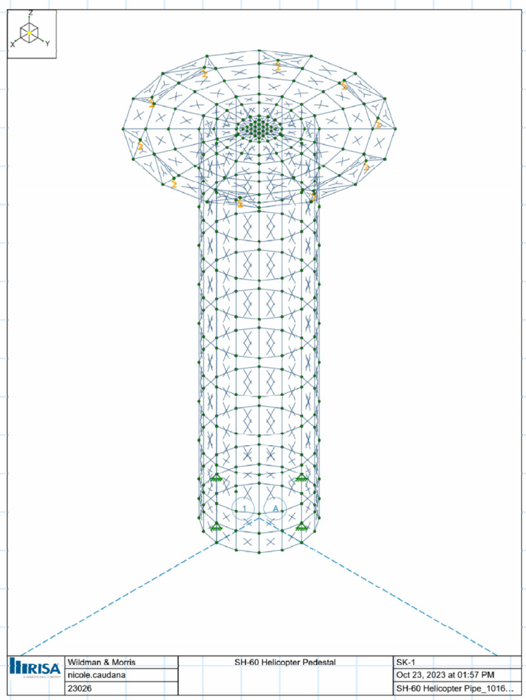
$\frac{V_u}{\phi Vn} = 0.85$ < 1.0 OK

RISA Output

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
0	N5	max	0	12	0	12	0.367	4	0	12	0	12	0	12
1		min	0	1	0	1	0	2	0	1	0	1	0	1
2	N14	max	0	12	0	12	8.863	10	0	12	0	12	0	12
3		min	0	1	0	1	0	2	0	1	0	1	0	1
4	N11	max	0	12	0	12	8.917	10	0	12	0	12	0	12
5		min	0	1	0	1	0	2	0	1	0	1	0	1
6	N12	max	0	12	0	12	17.273	10	0	12	0	12	0	12
7		min	0	1	0	1	0	2	0	1	0	1	0	1
8	N13	max	0	12	0	12	17.359	10	0	12	0	12	0	12
9		min	0	1	0	1	0	2	0	1	0	1	0	1
10	N9	max	0	12	0	12	8.725	9	0	12	0	12	0	12
11		min	0	1	0	1	0	2	0	1	0	1	0	1
12	N10	max	0	12	0	12	0.35	4	0	12	0	12	0	12
13		min	0	1	0	1	0	2	0	1	0	1	0	1
14	N7	max	0	12	0	12	17.191	9	0	12	0	12	0	12
15		min	0	1	0	1	0	2	0	1	0	1	0	1
16	N6	max	0	12	0	12	17.469	11	0	12	0	12	0	12
17		min	0	1	0	1	0	2	0	1	0	1	0	1
18	N4	max	0	12	0	12	8.772	9	0	12	0	12	0	12
19		min	0	1	0	1	0	2	0	1	0	1	0	1
20	N356	max	0.473	12	6.416	11	3.369	4	0	12	0	12	0	12
21		min	-0.151	4	-6.41	12	-11.106	11	0	1	0	1	0	1
22	N333	max	0.001	4	0.848	11	9.853	9	0	12	0	12	0	12
23		min	-0.276	8	-0.151	4	-29.248	12	0	1	0	1	0	1
24	N340	max	0.15	4	6.401	11	3.375	4	0	12	0	12	0	12
25		min	-0.468	12	-6.395	12	-11.064	12	0	1	0	1	0	1
26	N347	max	0.215	8	0.15	4	9.649	10	0	12	0	12	0	12
27		min	-0.005	9	-0.859	12	-29.32	11	0	1	0	1	0	1
28	Totals:	max	0	11	13.527	11	17	9						
29		min	0	12	-13.527	12	0	2						





$V_{max} := 6.42$ ***kip*** ▯<▯ $Q = 21.537$ ***kip*** *OK*

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TORSION ANALYSIS OF PEDESTAL

SH-60 Helicopter Wind Load

Include << V:\draw\23026 RAB Helicopter on a Stick NASNI\400-Technical\Calculations\Loading Criteria and Anchorage Calcs_101123.mcdx

Wind Loads: Wind Horiz: $V_u := P_{wind} = 10.5\ kip$ $\phi := 0.75$

Torsion: $T_u := M_{wind\_yaw} = 53.375\ kip \cdot ft$ $\lambda := 1.0$

Dead Load: $N_u := P_D = 12.143\ kip$ $\theta := 45$ deg.

Concrete: $f'_c := 5000 \cdot psi$ Concrete cover: $c_c := 3 \cdot in$

Rebar: $f_y := 60.0 \cdot ksi$ $f_{yt} := 60.0 \cdot ksi$

At top of pedestal, 18" x 24"

Pedestal Area: Width: $b_w := 18 \cdot in$ Depth: $d := 24 \cdot in - c_c = 21\ in$

$A_g := b_w \cdot (d + c_c) = 432\ in^2$ $A_{cp} := b_w \cdot (d + c_c) = 432\ in^2$

$p_{cp} := 2 \cdot b_w + 2 \cdot (d + c_c) = 84\ in$

Threshold Torsion:

$$T_{th} := \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot \left(\frac{A_{cp}^2}{p_{cp}}\right) \cdot \sqrt{1 + \frac{N_u}{4 \cdot A_g \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi}}$$

$\phi \cdot T_{th} = 10.3\ ft \cdot kip$ Table 22.7.4.1(a) Eq. (c)

Cracking Torsion:

$$T_{cr} := 4 \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot \left(\frac{A_{cp}^2}{p_{cp}}\right) \cdot \sqrt{1 + \frac{N_u}{4 \cdot A_g \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi}}$$

$T_{cr} = 54.9\ ft \cdot kip$ Table 22.7.5.1 Eq. (c)

$\phi \cdot T_{cr} = 41.2\ ft \cdot kip$

T_u exceeds $\phi \cdot T_{cr}$, therefore design to resist T_u (para. 22.7.3.1)

Torsional Strength:

Top of pedestal, first 5ft

Stirrup spacing: $s := 4 \cdot in$

Stirrup depth: $d_s := d - c_c = 18\ in$

Stirrup width: $b_s := b_w - 2 \cdot c_c = 12\ in$

Stirrup perimeter: $p_h := 2 \cdot (b_s - 0.625 \cdot in) + 2 \cdot (d_s - 0.625 \cdot in) = 57.5\ in$

#5 area: $A_t := 0.31 \cdot in^2$

Longitudinal bars: Number: $n := 4$ #11 area: $A_{11} := 1.56 \cdot in^2$

Consider only corner bars

$$A_l := n \cdot A_{11} = 6.24\ in^2$$

$$A_{oh} := (d_s - 0.625 \cdot in) \cdot (b_s - 0.625 \cdot in) = 197.6\ in^2$$

$$A_o := 0.85 \cdot A_{oh} = 168.0\ in^2$$

$$T_n := min\left(\frac{2 \cdot A_o \cdot A_t \cdot f_{yt}}{s} \cdot \cot(\theta), \frac{2 \cdot A_o \cdot A_l \cdot f_y}{p_h} \cdot \tan(\theta)\right)$$

$T_n = 80.4\ ft \cdot kip$ (Eq. 22.7.6.1a & b)

$\phi \cdot T_n = 60.3\ ft \cdot kip$

$\phi \cdot T_n > T_u$ ∴ OKAY

Cross-sectional limits

$A_s := n \cdot A_{11}$ $\rho_w := \frac{A_s}{b_w \cdot d} = 0.017$

$\frac{N_u}{6 \cdot A_g} = 4.68\ psi$ < $0.05 \cdot f'_c = 250\ psi$ OKAY (para. 22.5.5.1.2)

$$V_c := min\left(\left(2 \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi + \frac{N_u}{6 \cdot A_g}\right) \cdot b_w \cdot d, \left(8 \cdot \lambda \cdot (\rho_w)^{\frac{1}{3}} \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi + \frac{N_u}{6 \cdot A_g}\right) \cdot b_w \cdot d\right)$$

$V_c = 55.2\ kip$ Table 22.5.5.1 Eq. (a) & (b)

$5 \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot b_w \cdot d = 134\ kip$ $> V_c$

$$\sqrt{\left(\frac{V_u}{b_w \cdot d}\right)^2 + \left(\frac{T_u \cdot p_h}{1.7 \cdot A_{oh}^2}\right)^2} = 555.3\ psi \le \phi \cdot \left(\frac{V_c}{b_w \cdot d} + 8 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi\right) = 533.8\ psi$$

(Eq. 22.7.7.1a) *OK*

<u>At 10' above base, 20" x 40"</u> $N_u := N_u + 0.63 \cdot kip = 12.8\ kip$

Pedestal Area: Width: $b_w := 20 \cdot in$ Depth: $d := 40 \cdot in - c_c = 37\ in$

$A_g := b_w \cdot (d + c_c) = 800\ in^2$ $A_{cp} := b_w \cdot (d + c_c) = 800\ in^2$

$p_{cp} := 2 \cdot b_w + 2 \cdot (d + c_c) = 120\ in$

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Threshold Torsion:

$$T_{th} := \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot \left(\frac{{A_{cp}}^2}{p_{cp}}\right) \cdot \sqrt{1 + \frac{N_u}{4 \cdot A_g \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi}}$$

$\phi \cdot T_{th} = 24.2\ ft \cdot kip$ Table 22.7.4.1(a) Eq. (c)

Cracking Torsion:

$$T_{cr} := 4 \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot \left(\frac{{A_{cp}}^2}{p_{cp}}\right) \cdot \sqrt{1 + \frac{N_u}{4 \cdot A_g \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi}}$$

$T_{cr} = 129.2\ ft \cdot kip$ Table 22.7.5.1 Eq. (c)

$\phi \cdot T_{cr} = 96.9\ ft \cdot kip$

T_u less than $\phi \cdot T_{cr}$, therefore design to resist T_u (para. 22.7.3.1)

Torsional Strength:

Stirrup spacing: $s := 10 \cdot in$
Stirrup depth: $d_s := d - c_c = 34\ in$
Stirrup width: $b_s := b_w - 2 \cdot c_c = 14\ in$
Stirrup perimeter: $p_h := 2 \cdot (b_s - 0.625 \cdot in) + 2 \cdot (d_s - 0.625 \cdot in) = 93.5\ in$
#5 area: $A_t := 0.31 \cdot in^2$

Longitudinal bars: Number: $n := 4$ #11 area: $A_{11} := 1.56 \cdot in^2$

$A_l := n \cdot A_{11} = 6.24\ in^2$

$A_{oh} := (d_s - 0.625 \cdot in) \cdot (b_s - 0.625 \cdot in) = 446.4\ in^2$

$A_o := 0.85 \cdot A_{oh} = 379.4\ in^2$

$$T_n := min\left(\frac{2 \cdot A_o \cdot A_t \cdot f_{yt}}{s} \cdot \cot(\theta), \frac{2 \cdot A_o \cdot A_l \cdot f_y}{p_h} \cdot \tan(\theta)\right)$$

$T_n = 72.6\ ft \cdot kip$ (Eq. 22.7.6.1a & b)
$\phi \cdot T_n = 54.5\ ft \cdot kip$

$\phi \cdot T_n > T_u \therefore$ OKAY

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Cross-sectional limits

$$A_s := n \cdot A_{11} \qquad \rho_w := \frac{A_s}{b_w \cdot d} = 0.008$$

$$\frac{N_u}{6 \cdot A_g} = 2.66\ psi \quad < \quad 0.05 \cdot f'_c = 250\ psi$$

OKAY
(para. 22.5.5.1.2)

$$V_c := min\left(\left(2 \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi + \frac{N_u}{6 \cdot A_g}\right) \cdot b_w \cdot d, \left(8 \cdot \lambda \cdot (\rho_w)^{\frac{1}{3}} \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi + \frac{N_u}{6 \cdot A_g}\right) \cdot b_w \cdot d\right)$$

$$V_c = 87.2\ kip$$

Table 22.5.5.1 Eq. (a) & (b)

$$5 \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot b_w \cdot d = 262\ kip \quad > V_c$$

$$\sqrt{\left(\frac{V_u}{b_w \cdot d}\right)^2 + \left(\frac{T_u \cdot p_h}{1.7 \cdot A_{oh}^2}\right)^2} = 177.4\ psi \quad \le \quad \phi \cdot \left(\frac{V_c}{b_w \cdot d} + 8 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi\right) = 512.6\ psi$$

(Eq. 22.7.7.1a) *OK*

At 5' above base, 22" x 56"

$$N_u := N_u + 1.05 \cdot kip = 13.8\ kip$$

Pedestal Area: Width: $b_w := 22 \cdot in$ Depth: $d := 56 \cdot in - c_c = 53\ in$

$$A_g := b_w \cdot (d + c_c) = 1232\ in^2 \qquad A_{cp} := b_w \cdot (d + c_c) = 1232\ in^2$$

$$p_{cp} := 2 \cdot b_w + 2 \cdot (d + c_c) = 156\ in$$

Threshold Torsion:

$$T_{th} := \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot \left(\frac{A_{cp}^2}{p_{cp}}\right) \cdot \sqrt{1 + \frac{N_u}{4 \cdot A_g \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi}}$$

$$\phi \cdot T_{th} = 43.8\ ft \cdot kip$$

Table 22.7.4.1(a) Eq. (c)

Cracking Torsion:

$$T_{cr} := 4 \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot \left(\frac{A_{cp}^2}{p_{cp}}\right) \cdot \sqrt{1 + \frac{N_u}{4 \cdot A_g \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi}}$$

$$T_{cr} = 233.8\ ft \cdot kip$$

Table 22.7.5.1 Eq. (c)

$$\phi \cdot T_{cr} = 175.4\ ft \cdot kip$$

T_u less than $\phi \cdot T_{cr}$, therefore design to resist T_u (para. 22.7.3.1)

Torsional Strength:

Stirrup spacing: $s := 12 \cdot in$
Stirrup depth: $d_s := d - c_c = 50\ in$
Stirrup width: $b_s := b_w - 2 \cdot c_c = 16\ in$
Stirrup perimeter: $p_h := 2 \cdot (b_s - 0.625 \cdot in) + 2 \cdot (d_s - 0.625 \cdot in) = 129.5\ in$
#5 area: $A_t := 0.31 \cdot in^2$

Longitudinal bars: Number: $n := 4$ #11 area: $A_{11} := 1.56 \cdot in^2$

$$A_l := n \cdot A_{11} = 6.24\ in^2$$

$$A_{oh} := (d_s - 0.625 \cdot in) \cdot (b_s - 0.625 \cdot in) = 759.1\ in^2$$

$$A_o := 0.85 \cdot A_{oh} = 645.3\ in^2$$

$$T_n := min\left(\frac{2 \cdot A_o \cdot A_t \cdot f_{yt}}{s} \cdot \cot(\theta), \frac{2 \cdot A_o \cdot A_l \cdot f_y}{p_h} \cdot \tan(\theta)\right)$$

$T_n = 102.9\ ft \cdot kip$ (Eq. 22.7.6.1a & b)
$\phi \cdot T_n = 77.2\ ft \cdot kip$

$\phi \cdot T_n > T_u$ ∴ OKAY

Cross-sectional limits

$A_s := n \cdot A_{11}$ $\rho_w := \frac{A_s}{b_w \cdot d} = 0.005$

$\frac{N_u}{6 \cdot A_g} = 1.87\ psi$ < $0.05 \cdot f'_c = 250\ psi$ OKAY (para. 22.5.5.1.2)

$$V_c := min\left(\left(2 \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi + \frac{N_u}{6 \cdot A_g}\right) \cdot b_w \cdot d, \left(8 \cdot \lambda \cdot (\rho_w)^{\frac{1}{3}} \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi + \frac{N_u}{6 \cdot A_g}\right) \cdot b_w \cdot d\right)$$

$V_c = 117.6\ kip$ Table 22.5.5.1 Eq. (a) & (b)

$5 \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot b_w \cdot d = 412\ kip$ $> V_c$

$$\sqrt{\left(\frac{V_u}{b_w \cdot d}\right)^2 + \left(\frac{T_u \cdot p_h}{1.7 \cdot A_{oh}^2}\right)^2} = 85.1\ psi \le \phi \cdot \left(\frac{V_c}{b_w \cdot d} + 8 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi\right) = 499.9\ psi$$

(Eq. 22.7.7.1a) OK

"LOWER PLATE" AND "LOWER TUBE" PIPE ANALYSIS AND DESIGN

Global Parameters - Description:

Project Title	SH-60 Helicopter Pedestal
Company	Wildman & Morris
Designer	B. Miller
Job Number	23026.5
Notes	

Global Parameters - Solution:

Design Method	AISC 15th (360-16): LRFD
Bolt Group Analysis Method	Center of Rotation
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	Yes
Check Rotational Ductility?	Yes
Check Weld Filler Metal Matching?	Yes
Full Shear Eccentricity Considered?	No
Panel-Zone Shear Deformation Considered?	No
Check Weld Base Material Thickness?	Yes
Reduce Available Bolt Strength by Prying Effects Factor Q?	No

SH-60 Base Plate: LRFD Results Report

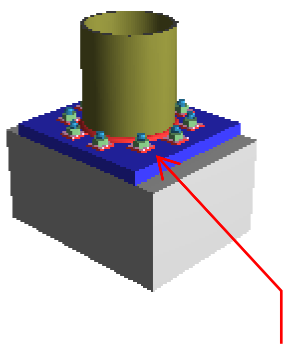

Plate washers will be rotated to clear pipe

Material Properties:

Column	HSS10.750X0.500	A500 Gr.B Round	F_y = 42.00 ksi	F_u = 58.00 ksi
Base Plate	P1.50x18.00x19.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Plate Washers	P0.25x2.00x2.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Axial	16.83 kips	*Axial load on the column*
Strong Axis Shear	0.00 kips	*Shear load on the column that causes strong axis bending*
Weak Axis Shear	13.53 kips	*Shear load on the column that causes weak axis bending*
Strong Axis Moment	0.00 kips-ft	*Column moment about the strong axis*
Weak Axis Moment	70.45 kips-ft	*Column moment about the weak axis*

Note: Unless specified, all code references are from AISC 360-16

Limit State	Required	Available	Unity Check	Result
Geometry Restrictions				**FAIL**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3)d_{bolt} (J3.3)*		
S_{min}	4.40 in	*Min bolt spacing*		
d_{bolt}	1.00 in	*Anchor bolt diameter*		
Check Min Edge Distance	**Pass**	*Condition: min(e_z , e_y) >= ED_{allow} (J3.4)*		
e_y	2.38 in	*Min edge distance y*		
e_z	2.22 in	*Min edge distance z*		
ED_{allow}	1.38 in	*Minimum allowed edge distance*		
Check Max Edge Distance	**Pass**	*Condition: Condition: max(d_z , d_y) <= min(6.00 in, 12*t) (J3.5)*		
d_y	2.38 in	*Max edge distance y*		
d_z	2.22 in	*Max edge distance z*		
ED_{allow}	6.00 in	*Maximum allowed edge distance*		
t	1.50 in	*Thickness of base plate*		
Check Plate Washer Encroachment on Column	**Fail**	Plate washers will be rotated to clear pipe		
Check Plate Washers Encroachment on Each Other	**Pass**			
Check Plate Washer Width	**Fail**	*Condition: L_{pw} > d_h + (2*AB_{tol})*		
L_{pw}	2.00 in	*Plate washer length*		
d_h	1.25 in	*Anchor bolt hole diameter*		
AB_{tol}	0.38 in	*Anchor bolt placement tolerance (per AISC 303-10, Section 7.5.1)*		
Check Plate Washer Weld Edge Distance	**Pass**	*Condition: min(e_z , e_y) - ((L_{pw} / 2) + D_{pw}) >= 1/16 in*		
D_{pw}	0.19 in	*Plate washer weld size*		
Load Distribution (Strong Axis)				**n/a**

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Limit State	Required	Available	Unity Check	Result
Strong and Weak Axis checked separately. No interaction check performed.				
Design with small moment ($e <= e_{crit}$).				
M_{uz}	0.00 kips-ft	*User input strong axis moment*		
P_u	16.83 kips	*User input axial load*		
N	19.00 in	*Plate length*		
B	18.00 in	*Plate width*		
ϕR_n	2.21 ksi	*Allowable bearing stress (see 'Concrete Bearing' check)*		
q_{max}	39.78 kips/in	*Bearing force = φR_n*B*		
e	0.00 in	*Equivalent eccentricity = $\|(M_{uz} / P_u)\|$*		
e_{crit}	9.29 in	*Critical eccentricity = (N/2) - P_u /(2*q_{max})*		
Y_y	19.00 in	*Bearing length = N-2*e (per AISC DG1, Eqn 3.3.8)*		
Load Distribution (Weak Axis)				**n/a**
Strong and Weak Axis checked separately. No interaction check performed.				
Design with large moment ($e > e_{crit}$). A free body diagram is used.				
M_{uy}	70.45 kips-ft	*User input weak axis moment*		
P_u	16.83 kips	*User input axial load*		
N	19.00 in	*Plate length*		
B	18.00 in	*Plate width*		
ϕR_n	2.21 ksi	*Allowable bearing stress (see 'Concrete Bearing' check)*		
q_{max}	41.99 kips/in	*Bearing force = φR_n*N*		
e	50.24 in	*Equivalent eccentricity = $\|(M_{uy} / P_u)\|$*		
e_{crit}	8.80 in	*Critical eccentricity = (B/2) - P_u /(2*q_{max})*		
Y_z	1.85 in	*Bearing length*		
T_{r1}	11.89 kips	*Tension in the bolts of Row 1 (bolts 3, 4)*		
T_{r2}	9.68 kips	*Tension in the bolts of Row 2 (bolts 2, 5)*		
T_{r3}	6.10 kips	*Tension in the bolts of Row 3 (bolts 1, 6)*		
T_{r4}	2.53 kips	*Tension in the bolts of Row 4 (bolts 7, 10)*		
T_{r5}	0.32 kips	*Tension in the bolts of Row 5 (bolts 8, 9)*		
Concrete Bearing			**0.02**	**PASS**
$R_n = 0.85 * f'_c * \alpha$		$\phi = 0.65$	(ACI 318-14 Table 22.8.3.2)	
Calculate Bearing Stress				
f'_c	4.00 ksi	*Concrete compressive strength*		
P_u	16.83 kips	*Axial load on the column*		
N	19.00 in	*Plate length*		
B	18.00 in	*Plate width*		
A_1	342.00 in^2	*Plate area = B*N*		
L	24.00 in	*Concrete support length*		
W	18.00 in	*Concrete support width*		
L'	19.00 in	*Effective concrete support length*		
W'	18.00 in	*Effective concrete support width*		
A_2	342.00 in^2	*Effective concrete support area = L' * W'*		

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Limit State	Required	Available	Unity Check	Result
α	1.00	*Bearing stress increase factor = min(2, $(A_2 / A_1)^{0.5}$)*		
ϕR_n	2.21 ksi	*Allowable bearing stress*		
Check Strong Axis	**Pass**	*Condition: $f_{py} <= \varphi R_n$*		
Design with small moment				
Y_y	19.00 in	*Bearing length (see 'Load Distribution Strong Axis' check)*		
f_{py}	0.05 ksi	*Required bearing stress = $P_u / (B * Y_y)$*		
UC_y	0.02	*Unity check for Strong Axis*		
Check Weak Axis	**Pass**	*Condition: $f_{pz} <= \varphi R_n$*		
Design with large moment				
f_{pz}	2.21 ksi	*Required bearing stress = φR_n*		
Plate Flexural Yielding(Compression)(Strong Axis)	0.06 kips-ft/in	1.52 kips-ft/in	**0.04**	**PASS**
$M_n = F_y * t_p^2 / 4$		**ϕ = 0.9**	(AISC DG1 (3.3.13))	
F_y	36.00 ksi	*Minimum yield strength of base plate*		
t_p	1.50 in	*Thickness of base plate*		
ϕM_n	1.52 kips-ft/in	*Base plate bending capacity per unit width*		
N	19.00 in	*Base plate length*		
B	18.00 in	*Base plate width*		
f_p	0.05 ksi	*Required bearing stress (see 'Concrete Bearing' check)*		
Y	19.00 in	*Bearing length (see 'Load Distribution' check)*		
D	10.80 in	*Column diameter*		
m	5.18 in	*m = (N - 0.8*D)/2 (per AISC DG1, sect. 3.1.3)*		
n	4.68 in	*n = (B - 0.8*D)/2 (per AISC DG1, sect. 3.1.3)*		
M_{pl1}	0.06 kips-ft/in	*Bending moment (straight) per unit width, $M_{pl1} = f_p * m^2 / 2$ (per AISC DG1, sect 3.3.2)*		
β	1.00	*Side bending adjustment factor. β=Y/((Y/2)+n), if (Y<2*n), otherwise β=1 (see Help for reference)*		
M_{pl2}	0.04 kips-ft/in	*Bending moment (side) per unit width, $M_{pl2} = f_p * n^2 / 2 * \beta$*		
M_{pl}	0.06 kips-ft/in	*Required bending moment per unit width , $M_{pl} = max(M_{pl1}, M_{pl2})$ (per AISC DG1, sect 3.1.2)*		
Plate Flexural Yielding(Compression)(Weak Axis)	1.28 kips-ft/in	1.52 kips-ft/in	**0.84**	**PASS**
$M_n = F_y * t_p^2 / 4$		**ϕ = 0.9**	(AISC DG1 (3.3.13))	
F_y	36.00 ksi	*Minimum yield strength of base plate*		
t_p	1.50 in	*Thickness of base plate*		
ϕM_n	1.52 kips-ft/in	*Base plate bending capacity per unit width*		
N	19.00 in	*Base plate length*		
B	18.00 in	*Base plate width*		
f_p	2.21 ksi	*Required bearing stress (see 'Concrete Bearing' check)*		
Y	1.85 in	*Bearing length (see 'Load Distribution' check)*		
D	10.80 in	*Column diameter*		
m	5.18 in	*m = (N - 0.8*D)/2 (per AISC DG1, sect. 3.1.3)*		
n	4.68 in	*n = (B - 0.8*D)/2 (per AISC DG1, sect. 3.1.3)*		
M_{pl1}	1.28 kips-ft/in	*Bending moment (straight) per unit width, $M_{pl1} = f_p * Y*(n - Y/2)$ (per AISC DG1, sect 3.3.2)*		
β	0.30	*Side bending adjustment factor. β=Y/((Y/2)+m), if (Y<2*m), otherwise β=1 (see Help for reference)*		

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SH-60 Base Plate: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
M_{pl2}	0.75 kips-ft/in	*Bending moment (side) per unit width, $M_{pl2} = f_p * m^2 / 2 * \beta$*		
M_{pl}	1.28 kips-ft/in	*Required bending moment per unit width , $M_{pl} = max(M_{pl1}, M_{pl2})$ (per AISC DG1, sect 3.1.2)*		
Plate Flexural Yielding(Tension)(Weak Axis)	4.87 kips-ft	14.15 kips-ft	**0.34**	**PASS**
$M_n = b_e*F_y*t_p^2 / 4$		**ϕ = 0.9**	(AISC DG1 (3.3.13))	
b_e	9.32 in	*Effective width of plate section*		
F_y	36.00 ksi	*Minimum yield strength of base plate*		
t_p	1.50 in	*Thickness of base plate*		
ϕM_n	14.15 kips-ft	*Base plate bending strength*		
T_3	11.89 kips	*Tension force in anchor bolt 3*		
T_4	11.89 kips	*Tension force in anchor bolt 4*		
x_3	2.46 in	*Moment arm for anchor bolt 3*		
x_4	2.46 in	*Moment arm for anchor bolt 4*		
M_{pl}	4.87 kips-ft	*Bending moment caused by tension at bolt, $M_{pl}=T_3*x_3 +T_4*x_4$*		
Anchor Bolt Tension (Weak Axis)	11.89 kips	25.62 kips	**0.46**	**PASS**
$R_n = F_{nt} * A_b$		**ϕ = 0.75**	(J3-2)	
Prying effects are ignored				
Check User Note Limit:	**$f_{rt}/(F_{nt}*\phi) <= 0.3$**			
f_{rt}	15.14 ksi	*Required tensile stress = T_{bolt} / A_b*		
F_{nt}	43.50 ksi	*Nominal tensile stress, per Table J3.2*		
Because $f_{rt}/(F_{nt}*\phi) > 0.3$, the Bolt Tensile Check is required				
Check Interaction Limit:	**$f_{rv} / (F_{nv}*\phi) <= 0.3$**			
f_{rv}	1.72 ksi	*Required shear stress: $f_{rv} = (V_u / N_{bolt}) / A_b$*		
F_{nv}	26.10 ksi	*Nominal shear stress, per Table J3.2*		
Because $f_{rv} / (R_{nv}*\phi) <= 0.3$, this check shall use the regular F_{nt} stress				
T_{bolt}	11.89 kips	*Max tension at bolts, see 'Load Distribution (Weak Axis)' check*		
V_u	13.53 kips	*Resultant shear force, see 'Anchor Bolt Shear' check*		
N_{bolt}	10	*Number of bolts*		
A_b	0.79 in^2	*Bolt cross sectional area*		
ϕR_n	25.62 kips	*Bolt tensile strength*		
Anchor Bolt Shear	13.53 kips	153.74 kips	**0.09**	**PASS**
$R_n = F_{nv}* A_b* N_{bolt}$		**ϕ = 0.75**	(J3-1)	
V_{uy}	0.00 kips	*Strong axis shear*		
V_{uz}	13.53 kips	*Weak axis shear*		
$V_u = (V_{uy}^2 + V_{uz}^2)^{0.5}$	13.53 kips	*Resultant shear force*		
F_{nv}	26.10 ksi	*Shear stress N type*		
A_b	0.79 in^2	*Area of bolt*		
N_{bolt}	10	*Number of bolts*		
ϕR_n	153.74 kips	*Bolt shear rupture strength*		

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SH-60 Base Plate: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
Anchor Bolt Bending	21.74 ksi	32.62 ksi	**0.67**	**PASS**
$F'_{nt} = \min(1.3*F_{nt} - (F_{nt} / (\phi*F_{nv})*f_{rv}), F_{nt})$		**$\phi = 0.75$**	(J3-3)	
F_{nt}	43.50 ksi	*Nominal tensile stress, per Table J3.2*		
F_{nv}	26.10 ksi	*Nominal shear stress, per Table J3.2*		
V_u	13.53 kips	*Resultant shear force, see 'Anchor bolt shear' check*		
N_{bolt}	10	*Number of bolts*		
A_b	0.79 in^2	*Bolt cross sectional area*		
f_{rv}	1.72 ksi	*Required shear stress: $f_{rv} = (V_u / N_{bolt}) / A_b$*		
$\phi F'_{nt}$	32.62 ksi	*Available tensile stress*		
t_p	1.50 in	*Thickness of base plate*		
t_{pw}	0.25 in	*Plate washer thickness*		
M_u	0.09 kips-ft	*Bending moment = $(V_u / N_{bolt})*(t_p + (t_{pw} / 2)) / 2$*		
d	1.00 in	*Bolt diameter*		
S	0.17 in^3	*Bolt section modulus = $d^3 / 6$*		
f_{tb}	6.59 ksi	*Bending stress = M_u / S*		
f_{rt}	15.14 ksi	*Required axial tensile stress, see 'Anchor Bolt Tension (Weak Axis)' check*		
f_t	21.74 ksi	*Total required tensile stress = $f_{tb} + f_{rt}$*		
Plate Washer Bending	0.09 kips-ft	0.17 kips-ft	**0.56**	**PASS**
$M_n = F_y * \pi * d_h * t_{pw}^2 / 4$		**$\phi = 0.90$**	(Eqn F2-1)	
T_u	11.89 kips	*Max anchor bolt tension, see 'Load Distribution' checks*		
F_y	36.00 ksi	*Minimum yield strength of washer plate*		
d_h	1.25 in	*Anchor bolt hole diameter in base plate*		
$d_{p\text{-}hole}$	1.06 in	*Anchor bolt hole diameter in plate washer*		
t_{pw}	0.25 in	*Plate washer thickness*		
M_u	0.09 kips-ft	*Bending on plate washer = $T_u*(d_h / 2 - d_{p\text{-}hole} / 2)$*		
ϕM_n	0.17 kips-ft	*Plate washer bending capacity*		
Anchor Bolt Bearing on Plate Washer	13.53 kips	153.74 kips	**0.09**	**PASS**
$R_n = N_{bolt} * \min(2.4*d_b*t_{pw}*F_u, R_{n\text{-}bolt})$		**$\phi = 0.75$**	(section J3.10)	
V_u	13.53 kips	*Resultant shear force, see 'Anchor Bolt Shear' check*		
N_{bolt}	10	*Number of bolts*		
d_b	1.00 in	*Bolt diameter*		
t_{pw}	0.25 in	*Thickness of washer plate*		
F_u	58.00 ksi	*Minimum tensile stress of material*		
$R_{n\text{-}bolt}$	20.50 kips	*Bolt shear strength, $R_{n\text{-}bolt} = F_{nv}*A_{bolt}$*		
F_{nv}	26.10 ksi	*Nominal shear stress of bolt*		
A_{bolt}	0.79 in^2	*Area of bolt*		
ϕRn	153.74 kips	*Total bolt bearing strength*		
Plate Washer Punching Shear	11.89 kips	18.02 kips	**0.66**	**PASS**
$T_n = 0.60 * F_y * A_{gv}$		**$\phi = 1.00$**	(Eqn J4-3)	
T_u	11.89 kips	*Max anchor bolt tension, see 'Load Distribution' checks*		
F_y	36.00 ksi	*Minimum yield strength of washer plate*		
$d_{P\text{-}hole}$	1.06 in	*Anchor bolt hole diameter on plate washer*		
t_{pw}	0.25 in	*Plate washer thickness*		

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SH-60 Base Plate: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
A_{gv}	0.83 in^2	*Shear area = $d_{P\text{-}hole} * \pi * t_{pw}$*		
ϕT_n	18.02 kips	*Plate washer tensile capacity based on punching shear*		
Plate Washer Weld Limitations				**PASS**
Weld Max/Min Size, Length			(J2.2b)	
Check Weld Max Size	**Pass**			
D	0.19 in	*Weld size*		
D_{max}	0.25 in	*Max Size Allowed*		
t	0.25 in	*Min shelf dimension*		
Check Weld Min Size	**Pass**			
D	0.19 in	*Weld size*		
D_{min}	0.12 in	*Min size allowed per Table J2.4*		
t_{min}	0.25 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: $L_{min} \geq 4*D$ per J2.2b*		
D	0.19 in	*Weld size*		
L_{min}	2.00 in	*Min weld segment length*		
Check Weld Max Length	**Pass**	*Condition: $L_{max} \leq 100*D$*		
D	0.19 in	*Weld size*		
L_{max}	2.00 in	*Max weld segment length*		
Column Weld Limitations				**PASS**
Weld Min Size			(J2.2b)	
Check Weld Min Size	**Pass**			
D	0.50 in	*Weld size*		
D_{min}	0.19 in	*Min size allowed per Table J2.4*		
t_{min}	0.47 in	*Controlling member thickness*		
Plate Washer Weld Strength	13.53 kips	334.08 kips	**0.04**	**PASS**
$\phi Rn = N_{bolt} * C_1 * \alpha * 1.392 * D_{16} * L$				
Single Fillet				
$1.392 = \phi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, ϕ=0.75 (AISC 15th Eqn 8-2a)				
V_{uy}	0.00 kips	*Strong axis shear*		
V_{uz}	13.53 kips	*Weak axis shear*		
$V_u = (V_{uy}^2 + V_{uz}^2)^{0.5}$	13.53 kips	*Resultant shear force*		
N_{bolt}	10	*Number of bolts*		
C_1	1.00	*Electrode strength coefficient (AISC 15th table 8-3)*		
t	1.50 in	*Base material thickness (base plate)*		
α	1.00	*Base material proration factor (re-arrangement of AISC 15th Eqn 9-2)*		
D_{16}	3.00	*Weld fillet size in sixteenths of an inch*		
d	2.00 in	*Plate washer size*		
L	8.00 in	*Weld length L=4*d*		
ϕRn	334.08 kips	*Weld strength*		
Column Weld Strength	9.24 kips/in	11.14 kips/in	**0.83**	**PASS**
$\phi Rn = C_1 * \alpha * 1.392 * D_{16}$				
Single Fillet				
$1.392 = \phi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, ϕ=0.75 (AISC 15th Eqn 8-2a)				
C_1	1.00	*Electrode strength coefficient (AISC 15th table 8-3)*		
t	0.47 in	*Base material thickness (column)*		

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SH-60 Base Plate: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
α	1.00	*Base material proration factor (re-arrangement of AISC 15th Eqn 9-2)*		
D_{16}	8.00	*Weld fillet size in sixteenths of an inch*		
r_u	0.40 kips/in	*Required strength of the weld for in-plane force*		
r_0	9.23 kips/in	*Required strength of the weld for out-of-plane force (tensile)*		
r_{3d}	9.24 kips/in	*Resultant force $r_{3d} = (r_u^2 + r_0^2)^{0.5}$*		
ϕRn	11.14 kips/in	*Weld strength*		

SH-60 Base Plate: Connection Properties Report

Connection	
Connection Title	SH-60 Base Plate
Connection Type	Single Column Base Plate Connection
Connection Category	
Bolt Layout	Custom
Plate Washers	Yes
Loading (LRFD)	
Axial	16.827 kips
Strong Axis Shear	0.000 kips
Weak Axis Shear	13.527 kips
Strong Axis Moment	0.000 kips-ft
Weak Axis Moment	70.455 kips-ft
Components	
Column Section	HSS10.750X0.500
Material	A500 Gr.B Round
Base Plate	P1.50x18.00x19.00
Material	A36
Length	19.000 in
Width	18.000 in
Thickness	1.500 in
Static Friction Coefficient	0.550 Coeff
Hole Type	OVS
Plate Washers	P0.25x2.00x2.00
Material	A36
Width/Length(Square)	2.000 in
Thickness	0.250 in
Concrete Support	C24.00x18.00x12.00
Length	24.000 in
Width	18.000 in
Thickness	
Compressive Strength (f'c)	4.000 ksi
Anchor Bolts	1" F1554 Gr.36-N
Material	F1554 Gr.36-N
Diameter, in.	1"
Anchors	Bolts Coordinates...
Column Weld	E70
Type	Fillet
Fillet Size	8.000 Sixteenths
Plate Washer Weld	E70
Type	Fillet
Fillet Size	3.000 Sixteenths

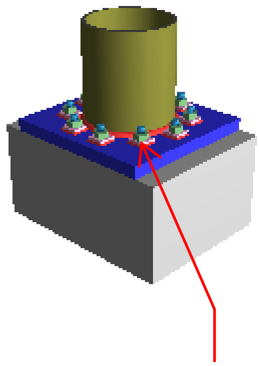
Global Parameters - Description:

Project Title	SH-60 Helicopter Pedestal
Company	Wildman & Morris
Designer	B. Miller
Job Number	23026.5
Notes	

Global Parameters - Solution:

Design Method	AISC 15th (360-16): LRFD
Bolt Group Analysis Method	Center of Rotation
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	Yes
Check Rotational Ductility?	Yes
Check Weld Filler Metal Matching?	Yes
Full Shear Eccentricity Considered?	No
Panel-Zone Shear Deformation Considered?	No
Check Weld Base Material Thickness?	Yes
Reduce Available Bolt Strength by Prying Effects Factor Q?	No

SH-60 Base Plate: LRFD Results Report



Material Properties:				
Column	HSS10.750X0.500	A500 Gr.B Round	F_y = 42.00 ksi	F_u = 58.00 ksi
Base Plate	P1.50x18.00x19.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Plate Washers	P0.50x2.00x2.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:		
Axial	16.83 kips	*Axial load on the column*
Strong Axis Shear	13.53 kips	*Shear load on the column that causes strong axis bending*
Weak Axis Shear	0.00 kips	*Shear load on the column that causes weak axis bending*
Strong Axis Moment	70.45 kips-ft	*Column moment about the strong axis*
Weak Axis Moment	0.00 kips-ft	*Column moment about the weak axis*

Note: Unless specified, all code references are from AISC 360-16

Limit State	Required	Available	Unity Check	Result
Geometry Restrictions				**FAIL**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3)d_{bolt} (J3.3)*		
S_{min}	4.40 in	*Min bolt spacing*		
d_{bolt}	1.00 in	*Anchor bolt diameter*		
Check Min Edge Distance	**Pass**	*Condition: min(e_z , e_y) >= ED_{allow} (J3.4)*		
e_y	2.38 in	*Min edge distance y*		
e_z	2.22 in	*Min edge distance z*		
ED_{allow}	1.38 in	*Minimum allowed edge distance*		
Check Max Edge Distance	**Pass**	*Condition: Condition: max(d_z , d_y) <= min(6.00 in, 12*t) (J3.5)*		
d_y	2.38 in	*Max edge distance y*		
d_z	2.22 in	*Max edge distance z*		
ED_{allow}	6.00 in	*Maximum allowed edge distance*		
t	1.50 in	*Thickness of base plate*		
Check Plate Washer Encroachment on Column	**Fail**	Plate washers will be rotated to clear pipe		
Check Plate Washers Encroachment on Each Other	**Pass**			
Check Plate Washer Width	**Fail**	*Condition: L_{pw} > d_h + (2*AB_{tol})*		
L_{pw}	2.00 in	*Plate washer length*		
d_h	1.25 in	*Anchor bolt hole diameter*		
AB_{tol}	0.38 in	*Anchor bolt placement tolerance (per AISC 303-10, Section 7.5.1)*		
Check Plate Washer Weld Edge Distance	**Pass**	*Condition: min(e_z , e_y) - ((L_{pw} / 2) + D_{pw}) >= 1/16 in*		
D_{pw}	0.19 in	*Plate washer weld size*		
Load Distribution (Strong Axis)				**n/a**

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Limit State	Required	Available	Unity Check	Result
Strong and Weak Axis checked separately. No interaction check performed.				
Design with large moment (e > e_{crit}). A free body diagram is used.				
M_{uz}	70.45 kips-ft	*User input strong axis moment*		
P_u	16.83 kips	*User input axial load*		
N	19.00 in	*Plate length*		
B	18.00 in	*Plate width*		
ϕR_n	2.21 ksi	*Allowable bearing stress (see 'Concrete Bearing' check)*		
q_{max}	39.78 kips/in	*Bearing force = φR_n*B*		
e	50.24 in	*Equivalent eccentricity = $\lvert(M_{uz}/P_u)\rvert$*		
e_{crit}	9.29 in	*Critical eccentricity = (N/2) - $P_u/(2*q_{max})$*		
Y_y	1.91 in	*Bearing length*		
T_{r1}	11.44 kips	*Tension in the bolts of Row 1 (bolt 6)*		
T_{r2}	10.38 kips	*Tension in the bolts of Row 2 (bolts 5, 7)*		
T_{r3}	7.61 kips	*Tension in the bolts of Row 3 (bolts 4, 8)*		
T_{r4}	4.19 kips	*Tension in the bolts of Row 4 (bolts 3, 9)*		
T_{r5}	1.42 kips	*Tension in the bolts of Row 5 (bolts 2, 10)*		
T_{r6}	0.36 kips	*Tension in the bolts of Row 6 (bolt 1)*		
Load Distribution (Weak Axis)				**n/a**
Strong and Weak Axis checked separately. No interaction check performed.				
Design with small moment (e <= e_{crit}).				
M_{uy}	0.00 kips-ft	*User input weak axis moment*		
P_u	16.83 kips	*User input axial load*		
N	19.00 in	*Plate length*		
B	18.00 in	*Plate width*		
ϕR_n	2.21 ksi	*Allowable bearing stress (see 'Concrete Bearing' check)*		
q_{max}	41.99 kips/in	*Bearing force = φR_n*N*		
e	0.00 in	*Equivalent eccentricity = $\lvert(M_{uy}/P_u)\rvert$*		
e_{crit}	8.80 in	*Critical eccentricity = (B/2) - $P_u/(2*q_{max})$*		
Y_z	18.00 in	*Bearing length = B-2*e (per AISC DG1, Eqn 3.3.8)*		
Concrete Bearing			**0.02**	**PASS**
$R_n = 0.85 * f'_c * \alpha$		**$\phi = 0.65$**	(ACI 318-14 Table 22.8.3.2)	
Calculate Bearing Stress				
f'_c	4.00 ksi	*Concrete compressive strength*		
P_u	16.83 kips	*Axial load on the column*		
N	19.00 in	*Plate length*		
B	18.00 in	*Plate width*		
A_1	342.00 in^2	*Plate area = B*N*		
L	24.00 in	*Concrete support length*		
W	18.00 in	*Concrete support width*		
L'	19.00 in	*Effective concrete support length*		
W'	18.00 in	*Effective concrete support width*		
A_2	342.00 in^2	*Effective concrete support area = L' * W'*		

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SH-60 Base Plate: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
α	1.00	*Bearing stress increase factor = min(2, $(A_2 / A_1)^{0.5}$)*		
ϕR_n	2.21 ksi	*Allowable bearing stress*		
Check Strong Axis	**Pass**	*Condition: $f_{py} <= \varphi R_n$*		
Design with large moment				
f_{py}	2.21 ksi	*Required bearing stress = φR_n*		
Check Weak Axis	**Pass**	*Condition: $f_{pz} <= \varphi R_n$*		
Design with small moment				
Y_z	18.00 in	*Bearing length (see 'Load Distribution Weak Axis' check)*		
f_{pz}	0.05 ksi	*Required bearing stress = $P_u / (N * Y_z)$*		
UC_z	0.02	*Unity check for Weak Axis*		
Plate Flexural Yielding(Compression)(Strong Axis)	1.48 kips-ft/in	1.52 kips-ft/in	**0.98**	**PASS**
$M_n = F_y * t_p^2 / 4$		**ϕ = 0.9**	(AISC DG1 (3.3.13))	
F_y	36.00 ksi	*Minimum yield strength of base plate*		
t_p	1.50 in	*Thickness of base plate*		
ϕM_n	1.52 kips-ft/in	*Base plate bending capacity per unit width*		
N	19.00 in	*Base plate length*		
B	18.00 in	*Base plate width*		
f_p	2.21 ksi	*Required bearing stress (see 'Concrete Bearing' check)*		
Y	1.91 in	*Bearing length (see 'Load Distribution' check)*		
D	10.80 in	*Column diameter*		
m	5.18 in	*m = (N - 0.8*D)/2 (per AISC DG1, sect. 3.1.3)*		
n	4.68 in	*n = (B - 0.8*D)/2 (per AISC DG1, sect. 3.1.3)*		
M_{pl1}	1.48 kips-ft/in	*Bending moment (straight) per unit width, $M_{pl1} = f_p * Y*(m - Y/2)$ (per AISC DG1, sect 3.3.2)*		
β	0.34	*Side bending adjustment factor. β=Y/((Y/2)+n), if (Y<2*n), otherwise β=1 (see Help for reference)*		
M_{pl2}	0.68 kips-ft/in	*Bending moment (side) per unit width, $M_{pl2} = f_p * n^2 / 2 * \beta$*		
M_{pl}	1.48 kips-ft/in	*Required bending moment per unit width , $M_{pl} = max(M_{pl1}, M_{pl2})$ (per AISC DG1, sect 3.1.2)*		
Plate Flexural Yielding(Compression)(Weak Axis)	0.06 kips-ft/in	1.52 kips-ft/in	**0.04**	**PASS**
$M_n = F_y * t_p^2 / 4$		**ϕ = 0.9**	(AISC DG1 (3.3.13))	
F_y	36.00 ksi	*Minimum yield strength of base plate*		
t_p	1.50 in	*Thickness of base plate*		
ϕM_n	1.52 kips-ft/in	*Base plate bending capacity per unit width*		
N	19.00 in	*Base plate length*		
B	18.00 in	*Base plate width*		
f_p	0.05 ksi	*Required bearing stress (see 'Concrete Bearing' check)*		
Y	18.00 in	*Bearing length (see 'Load Distribution' check)*		
D	10.80 in	*Column diameter*		
m	5.18 in	*m = (N - 0.8*D)/2 (per AISC DG1, sect. 3.1.3)*		
n	4.68 in	*n = (B - 0.8*D)/2 (per AISC DG1, sect. 3.1.3)*		
M_{pl1}	0.04 kips-ft/in	*Bending moment (straight) per unit width, $M_{pl1} = f_p * n^2 / 2$ (per AISC DG1, sect 3.3.2)*		
β	1.00	*Side bending adjustment factor. β=Y/((Y/2)+m), if (Y<2*m), otherwise β=1 (see Help for reference)*		

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SH-60 Base Plate: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
M_{pl2}	0.06 kips-ft/in	*Bending moment (side) per unit width, $M_{pl2} = f_p * m^2 / 2 * \beta$*		
M_{pl}	0.06 kips-ft/in	*Required bending moment per unit width , $M_{pl} = max(M_{pl1}, M_{pl2})$ (per AISC DG1, sect 3.1.2)*		
Plate Flexural Yielding(Tension)(Strong Axis)	1.87 kips-ft	5.95 kips-ft	**0.31**	**PASS**
$M_n = b_e*F_y*t_p^2 / 4$		**ϕ = 0.9**	(AISC DG1 (3.3.13))	
b_e	3.91 in	*Effective width of plate section*		
F_y	36.00 ksi	*Minimum yield strength of base plate*		
t_p	1.50 in	*Thickness of base plate*		
ϕM_n	5.95 kips-ft	*Base plate bending strength*		
T_6	11.44 kips	*Tension force in anchor bolt 6*		
x_6	1.96 in	*Moment arm for anchor bolt 6*		
M_{pl}	1.87 kips-ft	*Bending moment caused by tension at bolt, $M_{pl}=T_6*x_6$*		
Anchor Bolt Tension (Strong Axis)	11.44 kips	25.62 kips	**0.45**	**PASS**
$R_n = F_{nt} * A_b$		**ϕ = 0.75**	(J3-2)	
Prying effects are ignored				
Check User Note Limit:	**$f_{rt}/(F_{nt}*\phi) <= 0.3$**			
f_{rt}	14.56 ksi	*Required tensile stress = T_{bolt} / A_b*		
F_{nt}	43.50 ksi	*Nominal tensile stress, per Table J3.2*		
Because $f_{rt}/(F_{nt}*\phi) > 0.3$, the Bolt Tensile Check is required				
Check Interaction Limit:	**$f_{rv} / (F_{nv}*\phi) <= 0.3$**			
f_{rv}	1.72 ksi	*Required shear stress: $f_{rv} = (V_u / N_{bolt}) / A_b$*		
F_{nv}	26.10 ksi	*Nominal shear stress, per Table J3.2*		
Because $f_{rv} / (R_{nv}*\phi) <= 0.3$, this check shall use the regular F_{nt} stress				
T_{bolt}	11.44 kips	*Max tension at bolts, see 'Load Distribution (Strong Axis)' check*		
V_u	13.53 kips	*Resultant shear force, see 'Anchor Bolt Shear' check*		
N_{bolt}	10	*Number of bolts*		
A_b	0.79 in^2	*Bolt cross sectional area*		
ϕR_n	25.62 kips	*Bolt tensile strength*		
Anchor Bolt Shear	13.53 kips	153.74 kips	**0.09**	**PASS**
$R_n = F_{nv}* A_b* N_{bolt}$		**ϕ = 0.75**	(J3-1)	
V_{uy}	13.53 kips	*Strong axis shear*		
V_{uz}	0.00 kips	*Weak axis shear*		
$V_u = (V_{uy}^2 + V_{uz}^2)^{0.5}$	13.53 kips	*Resultant shear force*		
F_{nv}	26.10 ksi	*Shear stress N type*		
A_b	0.79 in^2	*Area of bolt*		
N_{bolt}	10	*Number of bolts*		
ϕR_n	153.74 kips	*Bolt shear rupture strength*		
Anchor Bolt Bending	21.67 ksi	32.62 ksi	**0.66**	**PASS**
$F'_{nt} = min(1.3*F_{nt}- (F_{nt} / (\phi*F_{nv})*f_{rv}), F_{nt})$		**ϕ = 0.75**	(J3-3)	

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Limit State	Required	Available	Unity Check	Result
F_{nt}	43.50 ksi	*Nominal tensile stress, per Table J3.2*		
F_{nv}	26.10 ksi	*Nominal shear stress, per Table J3.2*		
V_u	13.53 kips	*Resultant shear force, see 'Anchor bolt shear' check*		
N_{bolt}	10	*Number of bolts*		
A_b	0.79 in^2	*Bolt cross sectional area*		
f_{rv}	1.72 ksi	*Required shear stress: $f_{rv} = (V_u / N_{bolt}) / A_b$*		
$\phi F'_{nt}$	32.62 ksi	*Available tensile stress*		
t_p	1.50 in	*Thickness of base plate*		
t_{pw}	0.50 in	*Plate washer thickness*		
M_u	0.10 kips-ft	*Bending moment = $(V_u / N_{bolt})*(t_p + (t_{pw} / 2)) / 2$*		
d	1.00 in	*Bolt diameter*		
S	0.17 in^3	*Bolt section modulus = $d^3 / 6$*		
f_{tb}	7.10 ksi	*Bending stress = M_u / S*		
f_{rt}	14.56 ksi	*Required axial tensile stress, see 'Anchor Bolt Tension (Strong Axis)' check*		
f_t	21.67 ksi	*Total required tensile stress = $f_{tb} + f_{rt}$*		
Plate Washer Bending	0.09 kips-ft	0.66 kips-ft	**0.13**	**PASS**
$M_n = F_y * \pi * d_h * t_{pw}^2 / 4$		$\phi = 0.90$	(Eqn F2-1)	
T_u	11.44 kips	*Max anchor bolt tension, see 'Load Distribution' checks*		
F_y	36.00 ksi	*Minimum yield strength of washer plate*		
d_h	1.25 in	*Anchor bolt hole diameter in base plate*		
$d_{p\text{-}hole}$	1.06 in	*Anchor bolt hole diameter in plate washer*		
t_{pw}	0.50 in	*Plate washer thickness*		
M_u	0.09 kips-ft	*Bending on plate washer = $T_u*(d_h / 2 - d_{p\text{-}hole} / 2)$*		
ϕM_n	0.66 kips-ft	*Plate washer bending capacity*		
Anchor Bolt Bearing on Plate Washer	13.53 kips	153.74 kips	**0.09**	**PASS**
$R_n = N_{bolt} * \min(2.4*d_b*t_{pw}*F_u, R_{n\text{-}bolt})$		$\phi = 0.75$	(section J3.10)	
V_u	13.53 kips	*Resultant shear force, see 'Anchor Bolt Shear' check*		
N_{bolt}	10	*Number of bolts*		
d_b	1.00 in	*Bolt diameter*		
t_{pw}	0.50 in	*Thickness of washer plate*		
F_u	58.00 ksi	*Minimum tensile stress of material*		
$R_{n\text{-}bolt}$	20.50 kips	*Bolt shear strength, $R_{n\text{-}bolt} = F_{nv} * A_{bolt}$*		
F_{nv}	26.10 ksi	*Nominal shear stress of bolt*		
A_{bolt}	0.79 in^2	*Area of bolt*		
ϕRn	153.74 kips	*Total bolt bearing strength*		
Plate Washer Punching Shear	11.44 kips	36.05 kips	**0.32**	**PASS**
$T_n = 0.60 * F_y * A_{gv}$		$\phi = 1.00$	(Eqn J4-3)	
T_u	11.44 kips	*Max anchor bolt tension, see 'Load Distribution' checks*		
F_y	36.00 ksi	*Minimum yield strength of washer plate*		
$d_{P\text{-}hole}$	1.06 in	*Anchor bolt hole diameter on plate washer*		
t_{pw}	0.50 in	*Plate washer thickness*		
A_{gv}	1.67 in^2	*Shear area = $d_{P\text{-}hole} * \pi * t_{pw}$*		
ϕT_n	36.05 kips	*Plate washer tensile capacity based on punching shear*		

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SH-60 Base Plate: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
Plate Washer Weld Limitations				**PASS**
Weld Max/Min Size, Length			(J2.2b)	
Check Weld Max Size	**Pass**			
D	0.19 in	*Weld size*		
D_{max}	0.44 in	*Max Size Allowed*		
t	0.50 in	*Min shelf dimension*		
Check Weld Min Size	**Pass**			
D	0.19 in	*Weld size*		
D_{min}	0.19 in	*Min size allowed per Table J2.4*		
t_{min}	0.50 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: $L_{min} \geq 4*D$ per J2.2b*		
D	0.19 in	*Weld size*		
L_{min}	2.00 in	*Min weld segment length*		
Check Weld Max Length	**Pass**	*Condition: $L_{max} \leq 100*D$*		
D	0.19 in	*Weld size*		
L_{max}	2.00 in	*Max weld segment length*		
Column Weld Limitations				**PASS**
Weld Min Size			(J2.2b)	
Check Weld Min Size	**Pass**			
D	0.50 in	*Weld size*		
D_{min}	0.19 in	*Min size allowed per Table J2.4*		
t_{min}	0.47 in	*Controlling member thickness*		
Plate Washer Weld Strength	13.53 kips	334.08 kips	**0.04**	**PASS**
$\phi Rn = N_{bolt} * C_1 * \alpha * 1.392 * D_{16} * L$				
Single Fillet				
$1.392 = \phi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\phi=0.75$ (AISC 15^{th} Eqn 8-2a)				
V_{uy}	13.53 kips	*Strong axis shear*		
V_{uz}	0.00 kips	*Weak axis shear*		
$V_u = (V_{uy}^2 + V_{uz}^2)^{0.5}$	13.53 kips	*Resultant shear force*		
N_{bolt}	10	*Number of bolts*		
C_1	1.00	*Electrode strength coefficient (AISC 15^{th} table 8-3)*		
t	1.50 in	*Base material thickness (base plate)*		
α	1.00	*Base material proration factor (re-arrangement of AISC 15^{th} Eqn 9-2)*		
D_{16}	3.00	*Weld fillet size in sixteenths of an inch*		
d	2.00 in	*Plate washer size*		
L	8.00 in	*Weld length L=4*d*		
ϕRn	334.08 kips	*Weld strength*		
Column Weld Strength	9.24 kips/in	11.14 kips/in	**0.83**	**PASS**
$\phi Rn = C_1 * \alpha * 1.392 * D_{16}$				
Single Fillet				
$1.392 = \phi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\phi=0.75$ (AISC 15^{th} Eqn 8-2a)				
C_1	1.00	*Electrode strength coefficient (AISC 15^{th} table 8-3)*		
t	0.47 in	*Base material thickness (column)*		
α	1.00	*Base material proration factor (re-arrangement of AISC 15^{th} Eqn 9-2)*		

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SH-60 Base Plate: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
D_{16}	8.00	*Weld fillet size in sixteenths of an inch*		
r_u	0.40 kips/in	*Required strength of the weld for in-plane force*		
r_0	9.23 kips/in	*Required strength of the weld for out-of-plane force (tensile)*		
r_{3d}	9.24 kips/in	*Resultant force* $r_{3d} = (r_u^2 + r_0^2)^{0.5}$		
ϕRn	11.14 kips/in	*Weld strength*		

Connection	
Connection Title	SH-60 Base Plate
Connection Type	Single Column Base Plate Connection
Connection Category	
Bolt Layout	Custom
Plate Washers	Yes
Loading (LRFD)	
Axial	16.827 kips
Strong Axis Shear	13.527 kips
Weak Axis Shear	0.000 kips
Strong Axis Moment	70.455 kips-ft
Weak Axis Moment	0.000 kips-ft
Components	
Column Section	HSS10.750X0.500
Material	A500 Gr.B Round
Base Plate	P1.50x18.00x19.00
Material	A36
Length	19.000 in
Width	18.000 in
Thickness	1.500 in
Static Friction Coefficient	0.550 Coeff
Hole Type	OVS
Plate Washers	P0.50x2.00x2.00
Material	A36
Width/Length(Square)	2.000 in
Thickness	0.500 in
Concrete Support	C24.00x18.00x12.00
Length	24.000 in
Width	18.000 in
Thickness	
Compressive Strength (f'c)	4.000 ksi
Anchor Bolts	1" F1554 Gr.36-N
Material	F1554 Gr.36-N
Diameter, in.	1"
Anchors	Bolts Coordinates...
Column Weld	E70
Type	Fillet
Fillet Size	8.000 Sixteenths
Plate Washer Weld	E70
Type	Fillet
Fillet Size	3.000 Sixteenths

Weld Connection Check

Based on methodology from AISC Steel Manual 15th Ed & AISC 303-10 Section 3.1.2 Option 3

Input			Current AISC Code:	15th Ed		
Transfer Data						
LC 1.2D+W:			M, wind torsion =	53.375	kip-ft	
Tension Load, T =	**10.2**	kips	Vt = V / C =	**1.97**	kips/in	
N =	10	bolts				
T = T /(C * 1/N) =	**3.76**	k/in	Wind Shear V, =	10.5	kip	
			Vv = V / C =	**0.39**	kips/in	

$$V = \sqrt{(T^2 + V_v{}^2 + V_t{}^2)}$$

V =	**4.27**	k/in

Member & Components Property Summary

Pipe Diameter	8.625	in	Pipe C = Circum	27.10	in	
Plate	ASTM A36					
Plate	Dia=	17	in	Radius =	8.5	in
thickness	t =	1	in			
Steel Grade: A36	F_y =	36	ksi	F_u =	58	ksi
Bolt		ASTM A325				
Bolt	dia =	0.75	in	friction type shear connection (A325-N)		
	grade =	A325-N		F_u =		ksi
	F_{nt} =	90	ksi	F_{nv} =	54	ksi
slip critical	SC =	no				

Weld Strength - Shear

DCR ratio = **0.77**

Shear Force required	V_u =	4.27	kips/in	
thickness of plate	t_p =	1	in	
min size of fillet weld	D_{min} =	3/16	in	AISC 15th Table J2.4
weld size	D =	4	sixteenths	
length of weld	l =	27.1	in	
	θ =	0	°	
assumed filler material strength	F_{EXX} =	70	ksi	
$R_n = 0.6\ F_{EXX}\ (\sqrt{2}/2)(D/16)*l*[1+0.5\sin^{1.5}\theta]$	R_n =	7.425	kips/in	AISC 15th Eq. 8-1
Resistance factor - LRFD	Φ =	0.75		AISC 15th Eq 8-1
	ΦR_n=	**5.6**	kips/in	

Weld Strength - Tension

DCR ratio = **0.68**

Force required	P_u =	3.76	kips/in	
thickness of plate	t_p =	0	in	
min size of fillet weld	D_{min} =	3/16	in	AISC 15th Table J2.4
weld size	D =	4	sixteenths	
length of weld	l =	27.1	in	
	θ =	0	°	
assumed filler material strength	F_{EXX} =	70	ksi	
$R_n = 0.6\ F_{EXX}\ (\sqrt{2}/2)(D/16)*l*[1+0.5\sin^{1.5}\theta]$	R_n =	7.4	kips/in	AISC 15th Eq. 8-1
Resistance factor - LRFD	Φ =	0.75		AISC 15th Eq 8-1
	ΦR_n=	**5.6**	kips	

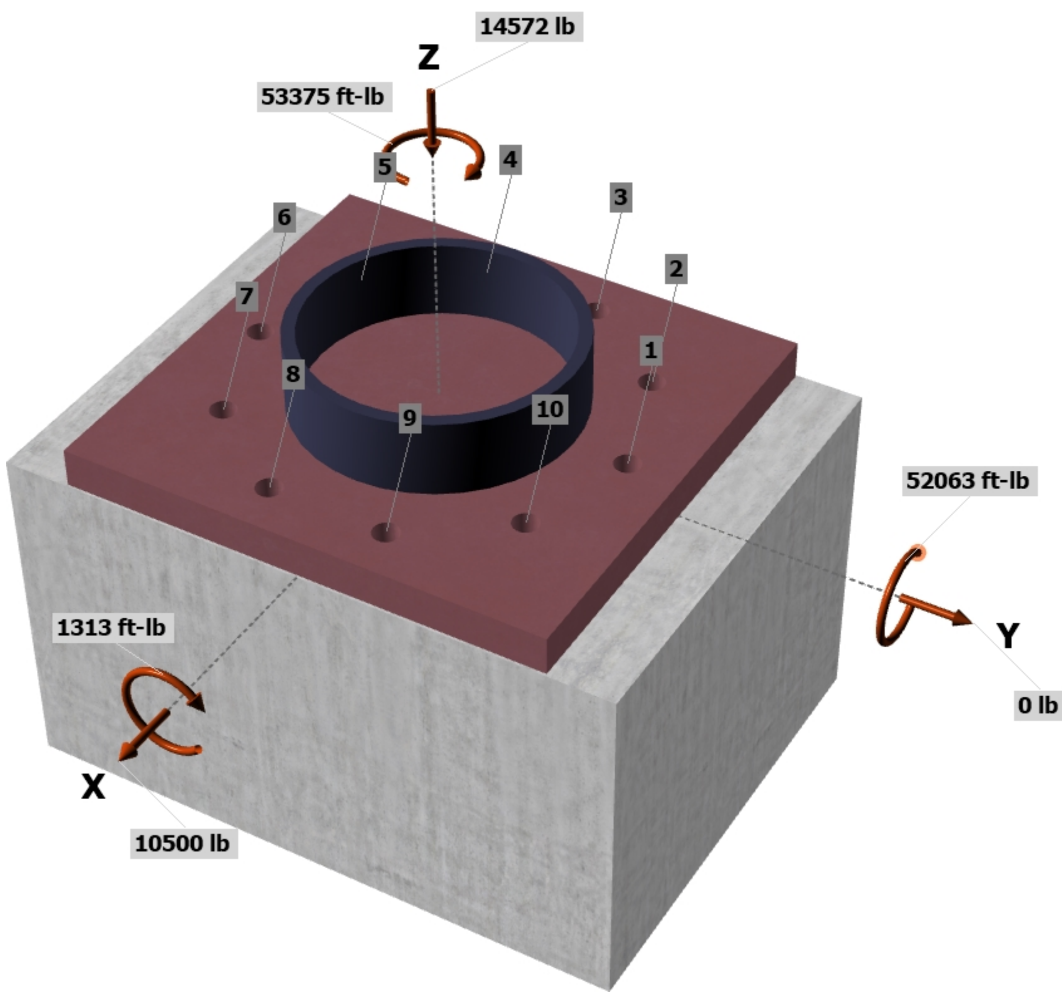
1.2D + W

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.10.5.3 (a) (iii)-(vi) is satisfied
Ductility section for shear: 17.10.6.3 (a) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

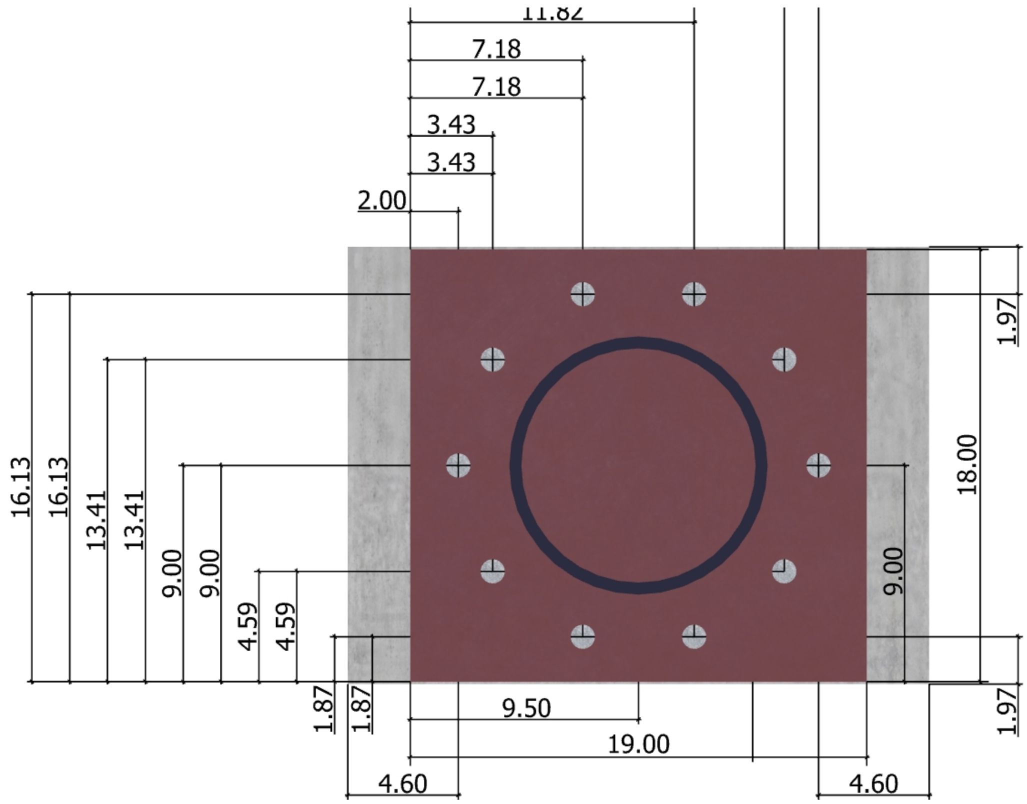
N_{ua} [lb]: -14572
V_{uax} [lb]: 10500
V_{uay} [lb]: 0
M_{ux} [ft-lb]: -1313
M_{uy} [ft-lb]: -52063
M_{uz} [ft-lb]: -53375

<Figure 1>



Company:	Wildman & Morris	Date:	9/8/2022
Engineer:	B. Miller	Page:	3/6
Project:	SH-60 Helicopter Pedestal		
Address:	405 Maple Street, Suite B-10, Ramona, CA 92065		
Phone:	760-789-3305		
E-mail:			

<Figure 2>



SIMPSON Strong-Tie® Anchor Designer™ Software
Version 3.1.2303.1

Company:	Wildman & Morris	Date:	9/8/2022
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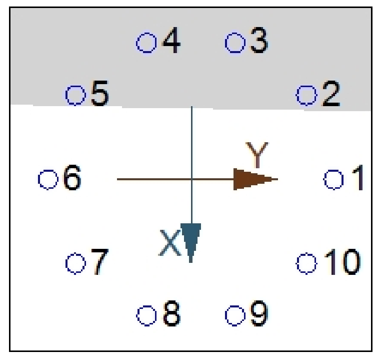
3. Resulting Anchor Forces

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	3344.0	9589.6	0.0	9589.6
2	0.0	7958.8	5019.3	9409.3
3	0.0	3689.0	8121.8	8920.3
4	0.0	-1589.0	8121.8	8275.8
5	0.0	-5858.8	5019.3	7714.8
6	3609.5	-7489.6	0.0	7489.6
7	7711.9	-5858.8	-5019.3	7714.8
8	10196.9	-1589.0	-8121.8	8275.8
9	10114.9	3689.0	-8121.8	8920.3
10	7497.1	7958.8	-5019.3	9409.3
Sum	42474.3	10500.0	0.0	85719.5

Maximum concrete compression strain (‰): 0.27
Maximum concrete compression stress (psi): 1174
Resultant tension force (lb): 0
Resultant compression force (lb): 57046
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. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
45450	0.75	34088

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

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

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	1.50	5000	0.70	31521