P O I N T 2

STRUCTURAL ENGINEERS INC.

APPROVED SET
COUNTY OF GLENN
THESE PRINTS MUST BE ON JOB SITE AT TIME OF INSPECTION

pages 1-60

REVIEWED FOR CODE COMPLIANCE BY:
WILLDAN ENGINEERING

Approval of these plans & specifications shall not be construed to be a permit for, or an approval of any violation of any Federal, State, County or City laws or ordinances. One set of approved plans must be kept on the job until completion.

10:59:27 AM Nov 17, 2022

Structural Calculations

for

Gazebo
City of Hamilton, CA.

AUG 22 2022

REGISTERED PROFESSIONAL ENGINEER
WILLIAM P. LARSON
3148
STRUCTURAL
STATE OF CALIFORNIA

POINT 2 Job No. 2022-047

August 2022

3701 Business Drive, Suite 100, Sacramento, CA 95820
Tel: 916-452-8200 Fax: 916-452-8212

Table of Contents

POINT 2 STRUCTURAL ENGINEERS INC.
3701 Business Drive, Suite 100, Sacramento, CA 95820
Tel: 916-452-8200 Fax: 916-452-8212

STRUCTURAL DESIGN REFERENCES

- *California Building Code*
  2019 Edition
- *ASCE 7-16 Minimum Design Loads for Buildings and Other Structures*

ASCE 7-16 – BASIC LOAD COMBINATIONS

2.3.2 STRENGTH DESIGN/ LOAD RESISTANCE FACTOR DESIGN

1. 1.4D
2. 1.2D + 1.6L + 0.5(L*r* or S or R) or
   1.2D + 1.6L + 0.2D*i* +0.5S (Section 2.3.4)
3. 1.2D + 1.6(L*r* or S or R) + (L or 0.5W)
4. 1.2D + 1.0W + L + 0.5(L*r* or S or R) or
   1.2D + L + D*i* + W*i* +0.5S (Section 2.3.4)
5. 1.2D + 1.0E + L + 0.2S
6. 0.9D + 1.0W or
   0.9D + Di + W*i* (Section 2.3.4)
7. 0.9D + 1.0E

2.4.1 ALLOWABLE STRESS DESIGN/ WORKING STRESS DESIGN

1. D
2. D + L + 0.7Di (Section 2.4.3)
3. D + (L*r* or S or R) or
   D + 0.7D*i* + 0.7W*i* + S (Section 2.4.3)
4. D + 0.75 L + 0.75(L*r* or S or R)
5. D + (.6W or 0.7E)

6a. D + 0.75L +0.75(0.6W) + 0.75(L*r* or S or R)

6b. D + 0.75L + 0.75(.7E) + 0.75S

7. 0.6D + .6W or
   0.6D + 0.7D*i* + 0.7W*i* (Section 2.4.3)
8. 0.6D + 0.7E

D = Dead load	D*i* = Weight of ice
L = Live load	L*r* = Live load (roof)
S = Snow load	R = Rain Load
W= Wind load	W*i*= Wind on ice
E= Earthquake load	

POINT 2 STRUCTURAL ENGINEERS INC.
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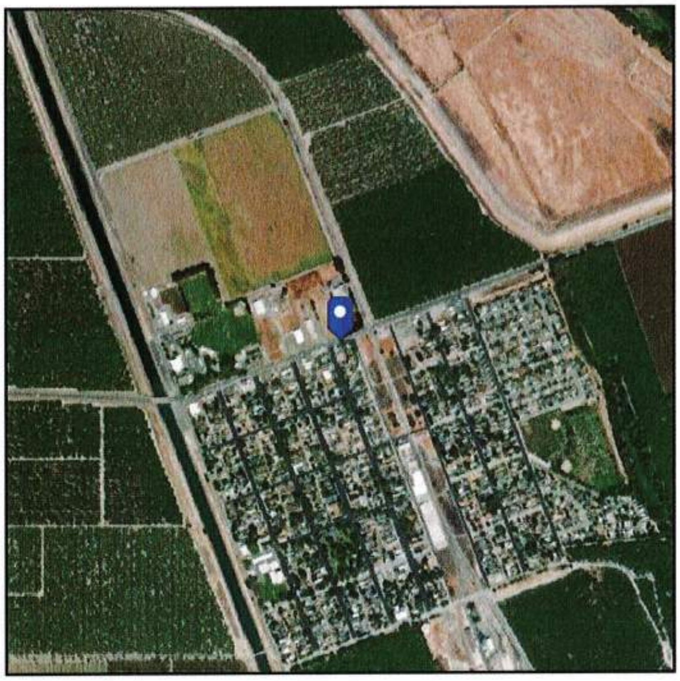
DESIGN PARAMETERS

ASCE 7 Hazards Report

Address:
Hamilton City
California,

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Stiff Soil

Elevation: 151.38 ft (NAVD 88)
Latitude: 39.74609
Longitude: -122.01366



Wind

Results:

Wind Speed	94 Vmph
10-year MRI	65 Vmph
25-year MRI	71 Vmph
50-year MRI	76 Vmph
100-year MRI	80 Vmph

Data Source: ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed: Wed Jun 01 2022

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

Site Soil Class: D - Stiff Soil

Results:

S_S :	0.824	S_{D1} :	N/A
S_1 :	0.337	T_L :	16
F_a :	1.17	PGA :	0.367
F_v :	N/A	PGA_M :	0.452
S_{MS} :	0.964	F_{PGA} :	1.233
S_{M1} :	N/A	I_e :	1
S_{DS} :	0.643	C_v :	1.212

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Wed Jun 01 2022

Date Source: USGS Seismic Design Maps

C AND C WIND LOADING

POINT 2 STRUCTURAL ENGINEERS INC.
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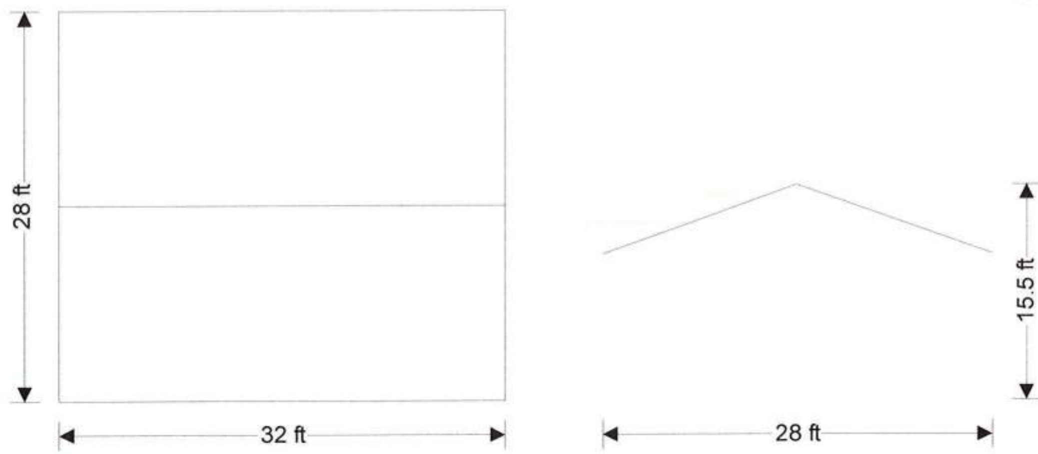
Tekla. Tedds Point 2 Structural Engineers Inc 3701 Business Dr Suite 100 Sacramento, CA 95820	Project			Job Ref.	
	Section			Sheet no./rev. 1	
	Calc. by B	Date 8/8/2022	Chk'd by	Date	App'd by	Date

WIND LOADING

In accordance with ASCE7-16

Using the components and cladding design method

Tedds calculation version 2.1.11



Plan Elevation

Building data

Type of roof	Gable free
Length of building	b = **32.00** ft
Width of building	d = **28.00** ft
Height to eaves	H = **10.50** ft
Pitch of roof	α_0 = **19.5** deg
Mean height	h = **12.98** ft
Wind flow	Clear

General wind load requirements

Basic wind speed	V = **94.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **33** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$ = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Open buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.00**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **0.00**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.85**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **16.3** psf

Tekla Tedds

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Suite 100
Sacramento, CA 95820

Project				Job Ref.	
Section				Sheet no./rev. 2	
Calc. by B	Date 8/8/2022	Chk'd by	Date	App'd by	Date

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **16.32** psf

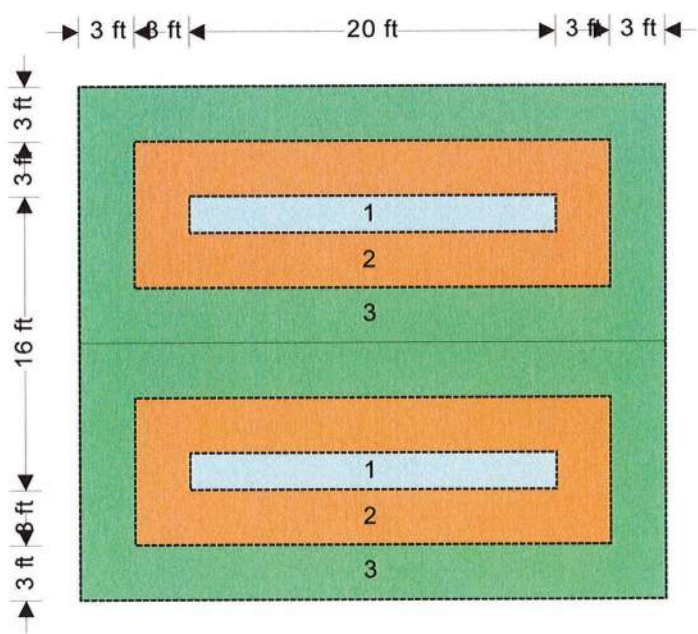
Equations used in tables

Net pressure $p = q_h \times [GC_p - GC_{pi}]$

Components and cladding pressures - Roof (Figure 30.7-2)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+C_N$	$-C_N$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=9 sf	1	-	-	9.0	1.16	-1.04	16.1	-14.4 #
18 sf	1	-	-	18.0	1.16	-1.04	16.1	-14.4 #
>36 sf	1	-	-	36.1	1.16	-1.04	16.1	-14.4 #
<=9 sf	2	-	-	9.0	1.79	-1.61	24.8	-22.3
18 sf	2	-	-	18.0	1.79	-1.61	24.8	-22.3
>36 sf	2	-	-	36.1	1.16	-1.04	16.1	-14.4 #
<=9 sf	3	-	-	9.0	2.32	-2.08	32.2	-28.9
18 sf	3	-	-	18.0	1.79	-1.61	24.8	-22.3
>36 sf	3	-	-	36.1	1.16	-1.04	16.1	-14.4 #

The final net design wind pressure, including all permitted reductions, used in the design shall not be less than 16psf acting in either direction



Plan on roof

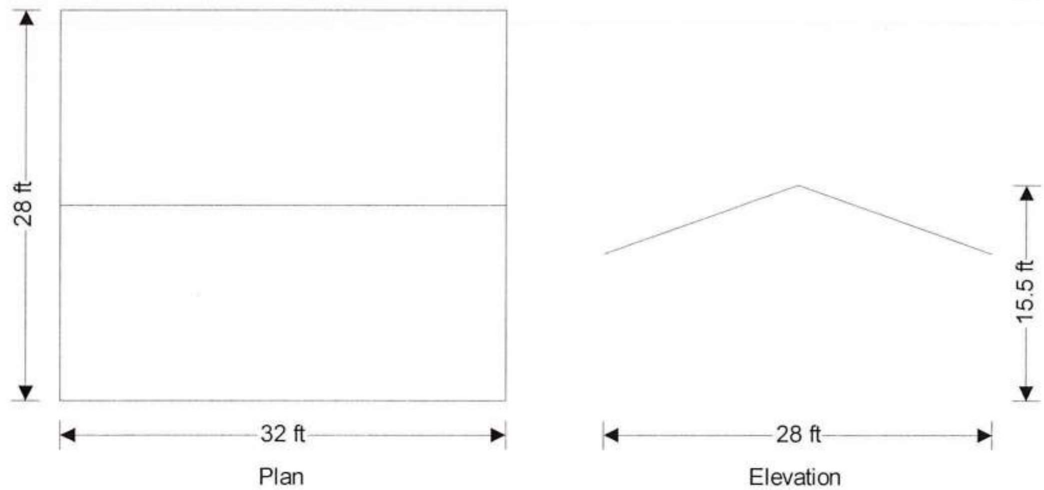
Tekla Tedds	Project				Job Ref.	
Point 2 Structural Engineers Inc 3701 Business Dr Suite 100 Sacramento, CA 95820	Section				Sheet no./rev. 1	
	Calc. by B	Date 8/8/2022	Chk'd by	Date	App'd by	Date

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.11



Building data

Type of roof	Gable free
Length of building	b = **32.00** ft
Width of building	d = **28.00** ft
Height to eaves	H = **10.50** ft
Pitch of roof	α_0 = **19.5** deg
Mean height	h = **12.98** ft
Wind flow	Clear

General wind load requirements

Basic wind speed	V = **94.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **33** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$ = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Open buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.00**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **0.00**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.85**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **16.3** psf

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **16.32** psf

Pressures and forces

Net pressure $p = q_h \times G \times C_N$

Net force $F_w = p \times A_{ref}$

Minimum design wind loading (cl.27.1.5) p_{min_r} = **16** lb/ft^2

Roof load case 1 - Wind 0 - Loadcase A

Zone	Ref. height (ft)	Ext pressure coefficient c_N	Peak velocity pressure q_h (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
1 (+ve)	12.98	1.10	16.32	15.26	475.26	7.25
2 (+ve)	12.98	-0.10	16.32	-1.39	475.26	-0.66

Total vertical net force $F_{w,v}$ = **6.22** kips

Total horizontal net force $F_{w,h}$ = **2.64** kips

Minimum loading

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0)$ = **158.65** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_r} \times A_{vert\_r\_0}$ = **2.54** kips

Roof load case 2 - Wind 0 - Loadcase B

Zone	Ref. height (ft)	Ext pressure coefficient c_N	Peak velocity pressure q_h (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
1 (+ve)	12.98	-0.02	16.32	-0.28	475.26	-0.13
2 (+ve)	12.98	-0.92	16.32	-12.77	475.26	-6.07

Total vertical net force $F_{w,v}$ = **-5.84** kips

Total horizontal net force $F_{w,h}$ = **1.98** kips

Minimum loading

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0)$ = **158.65** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_r} \times A_{vert\_r\_0}$ = **2.54** kips

Roof load case 3 - Wind 90 - Loadcase A

Zone	Ref. height (ft)	Ext pressure coefficient c_N	Peak velocity pressure q_h (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
1 (+ve)	12.98	-0.80	16.32	-11.10	385.52	-4.28
2 (+ve)	12.98	-0.60	16.32	-8.33	385.52	-3.21
3 (+ve)	12.98	-0.30	16.32	-4.16	179.48	-0.75

Total vertical net force $F_{w,v}$ = **-7.76** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Minimum loading

Projected vertical area of roof $A_{vert\_r\_90}$ = **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_r} \times A_{vert\_r\_90}$ = **0.00** kips

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	Section				Sheet no./rev. 3	
	Calc. by B	Date 8/8/2022	Chk'd by	Date	App'd by	Date

Roof load case 4 - Wind 90 - Loadcase B

Zone	Ref. height (ft)	Ext pressure coefficient c_N	Peak velocity pressure q_h (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
1 (+ve)	12.98	0.80	16.32	11.10	385.52	4.28
2 (+ve)	12.98	0.50	16.32	6.94	385.52	2.67
3 (+ve)	12.98	0.30	16.32	4.16	179.48	0.75

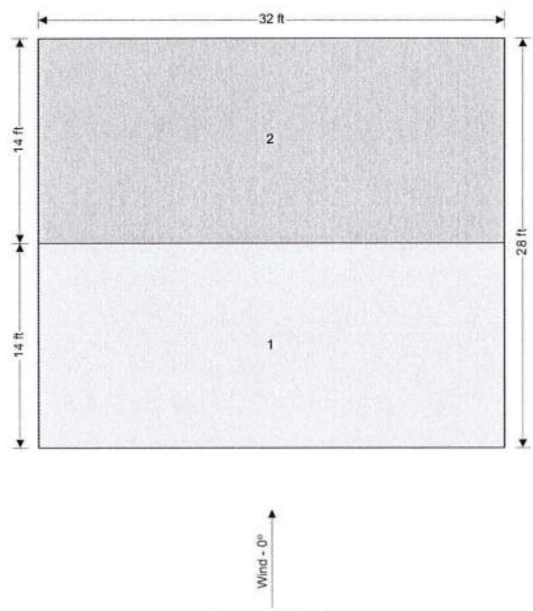
Total vertical net force $F_{w,v}$ = **7.26** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Minimum loading

Projected vertical area of roof $A_{vert\_r\_90}$ = **0.00** ft²

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_r} \times A_{vert\_r\_90}$ = **0.00** kips



Plan view - Gable roof

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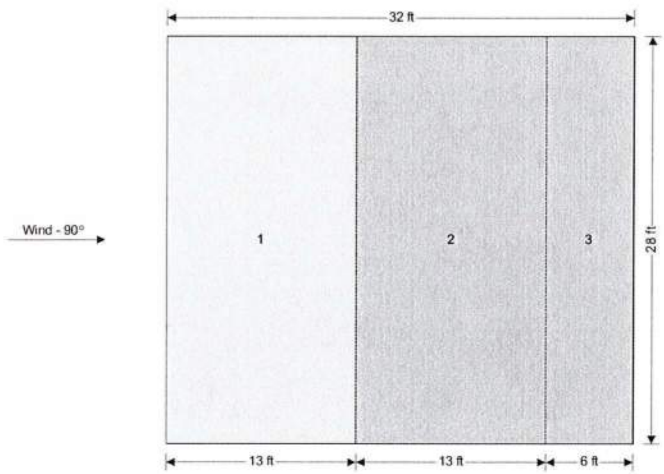


Plan view - Gable roof

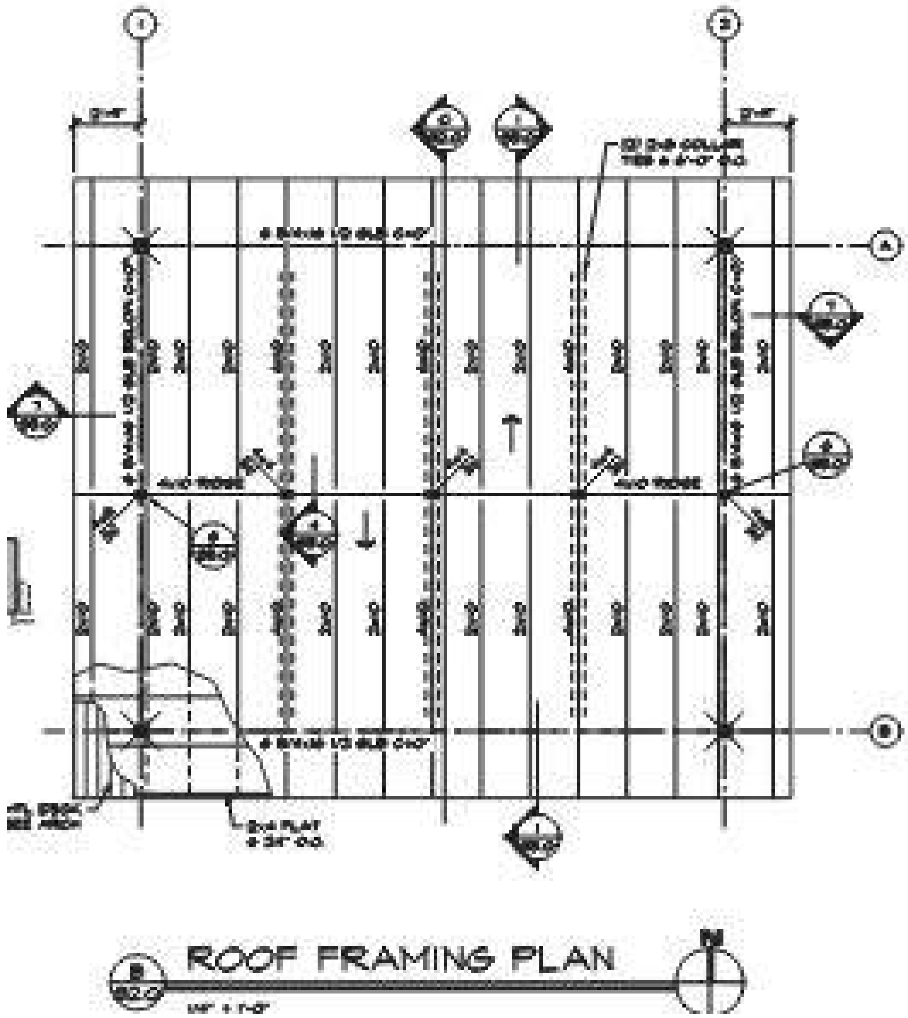
POINT 2 STRUCTURAL ENGINEERS INC
3701 BUSINESS DRIVE, SUITE 100, SACRAMENTO, CA 95820
PHONE: (916) 452-8200 FAX: (916) 452-8212

PROJECT
CLIENT
ENGINEER DATE / / PAGE of
PROJECT NO.

DEAD LOAD

ROOFING MTL	3.0
2x4 @ 24	1.0
2x10 @ 24	1.8
PLY	1.8
MISC	2.4
	10.0
SOLAR	3.5
	13.5

GAZEBO ROOF FRAMING DESIGN



Point 2 Structural Engineers Inc
3701 Business Dr. Suite 100
Sacramento CA 95820
916-452-8200

Wood Beam

Project File: 2022-047.ec6

LIC# : KW-06016300, Build:20.22.7.7 | POINT 2 STRUCTURAL ENGINEERING | (c) ENERCALC INC 1983-2022

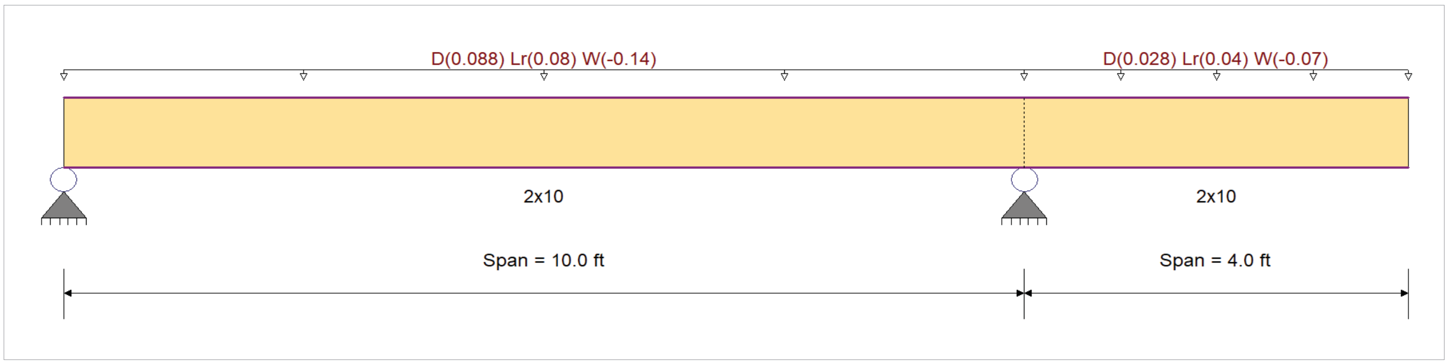
DESCRIPTION: roof rafter @ 24

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1350 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1350 psi	Ebend- xx	1600 ksi
		Fc - Prll	925 psi	Eminbend - xx	580 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625 psi		
Wood Grade :	No.1	Fv	170 psi		
		Ft	675 psi	Density	31.21 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : D = 0.0220, Lr = 0.020, W = -0.0350 ksf, Tributary Width = 4.0 ft, (roof)
Load for Span Number 2
Uniform Load : D = 0.0140, Lr = 0.020, W = -0.0350 ksf, Tributary Width = 2.0 ft, (roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.555** : 1	Maximum Shear Stress Ratio	=	**0.393** : 1
Section used for this span		**2x10**	Section used for this span		**2x10**
fb: Actual	=	1,030.42 psi	fv: Actual	=	83.50 psi
Fb: Allowable	=	1,856.25 psi	Fv: Allowable	=	212.50 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	4.693 ft	Location of maximum on span	=	9.274 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.149 in	Ratio =	644 >=360	Span: 2 : W Only
Max Upward Transient Deflection	-0.162 in	Ratio =	738 >=360	Span: 2 : Lr Only
Max Downward Total Deflection	0.204 in	Ratio =	589 >=180	Span: 2 : +0.60D+0.60W
Max Upward Total Deflection	-0.203 in	Ratio =	472 >=180	Span: 2 : +D+Lr

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.416	0.282	0.90	1.100	1.00	1.00	1.00	1.00	1.00	0.99	555.86	1336.50	0.40	43.08	153.00
Length = 4.0 ft	2	0.094	0.282	0.90	1.100	1.00	1.00	1.00	1.00	1.00	0.22	125.66	1336.50	0.09	43.08	153.00
+D+Lr					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.555	0.393	1.25	1.100	1.00	1.00	1.00	1.00	1.00	1.84	1,030.42	1856.25	0.77	83.50	212.50
Length = 4.0 ft	2	0.164	0.393	1.25	1.100	1.00	1.00	1.00	1.00	1.00	0.54	305.18	1856.25	0.22	83.50	212.50
+D+0.750Lr					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.491	0.345	1.25	1.100	1.00	1.00	1.00	1.00	1.00	1.63	911.77	1856.25	0.68	73.40	212.50
Length = 4.0 ft	2	0.140	0.345	1.25	1.100	1.00	1.00	1.00	1.00	1.00	0.46	260.30	1856.25	0.19	73.40	212.50
+D+0.60W					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Point 2 Structural Engineers Inc
3701 Business Dr. Suite 100
Sacramento CA 95820
916-452-8200

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 2022-047.ec6

LIC# : KW-06016300, Build:20.22.7.7 POINT 2 STRUCTURAL ENGINEERING (c) ENERCALC INC 1983-2022

DESCRIPTION: roof rafter @ 24

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 10.0 ft	1	0.029	0.018	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.12	68.26	2376.00	0.05	4.90	272.00
Length = 4.0 ft	2	0.026	0.018	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.11	62.83	2376.00	0.05	4.90	272.00
+D+0.750Lr+0.450W					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.226	0.153	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.96	538.07	2376.00	0.38	41.56	272.00
Length = 4.0 ft	2	0.050	0.153	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.21	118.93	2376.00	0.09	41.56	272.00
+D+0.450W					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.077	0.044	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.33	183.25	2376.00	0.11	11.85	272.00
Length = 4.0 ft	2	0.007	0.044	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.03	15.71	2376.00	0.01	11.85	272.00
+0.60D+0.60W					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.070	0.061	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.30	165.89	2376.00	0.15	16.59	272.00
Length = 4.0 ft	2	0.048	0.061	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.20	113.10	2376.00	0.08	16.59	272.00
+0.60D					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.140	0.095	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.59	333.52	2376.00	0.24	25.85	272.00
Length = 4.0 ft	2	0.032	0.095	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.13	75.40	2376.00	0.05	25.85	272.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.2037	4.916		0.0000	0.000
	2	0.0000	4.916	+D+Lr	-0.2028	4.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	0.786	1.166	
Overall MINimum	0.368	0.592	
D Only	0.418	0.574	
+D+Lr	0.786	1.166	
+D+0.750Lr	0.694	1.018	
+D+0.60W	0.031	-0.047	
+D+0.750Lr+0.450W	0.404	0.552	
+D+0.450W	0.128	0.108	
+0.60D+0.60W	-0.136	-0.277	
+0.60D	0.251	0.345	
Lr Only	0.368	0.592	
W Only	-0.644	-1.036	

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Sacramento CA 95820
916-452-8200

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 2022-047.ec6

LIC# : KW-06016300, Build:20.22.7.7 POINT 2 STRUCTURAL ENGINEERING (c) ENERCALC INC 1983-2022

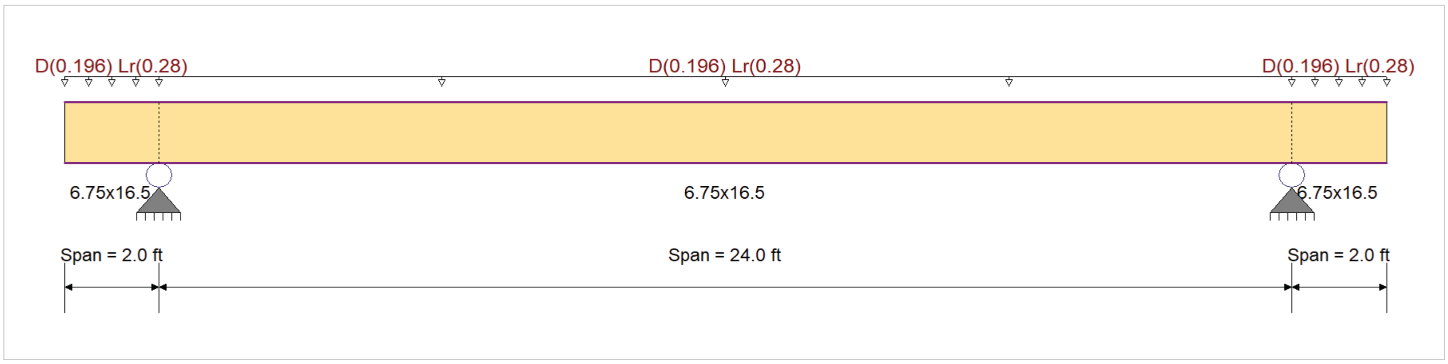
DESCRIPTION: East west GL

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*		
Load Combination :	ASCE 7-16	Fb -	1,850.0 psi		Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi		Eminbend - xx	950.0 ksi
Wood Species :	DF/DF	Fc - Perp	650.0 psi		Ebend- yy	1,600.0 ksi
Wood Grade :	24F-V4	Fv	265.0 psi		Eminbend - yy	850.0 ksi
		Ft	1,100.0 psi		Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling					



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.0140, Lr = 0.020 ksf, Tributary Width = 14.0 ft
Load for Span Number 2
Uniform Load : D = 0.0140, Lr = 0.020 ksf, Tributary Width = 14.0 ft
Load for Span Number 3
Uniform Load : D = 0.0140, Lr = 0.020 ksf, Tributary Width = 14.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.492** 1	Maximum Shear Stress Ratio	=	**0.219** : 1
Section used for this span		**6.75x16.5**	Section used for this span		**6.75x16.5**
fb: Actual	=	1,371.57 psi	fv: Actual	=	72.68 psi
Fb: Allowable	=	2,789.53 psi	Fv: Allowable	=	331.25 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	11.899 ft	Location of maximum on span	=	2.000 ft
Span # where maximum occurs	=	Span # 2	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.450 in	Ratio =	639	>=360	Span: 2 : Lr Only
Max Upward Transient Deflection	-0.117 in	Ratio =	408	>=360	Span: 3 : Lr Only
Max Downward Total Deflection	0.804 in	Ratio =	358	>=180	Span: 2 : +D+Lr
Max Upward Total Deflection	-0.209 in	Ratio =	228	>=180	Span: 3 : +D+Lr

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 2.0 ft	1	0.010	0.134	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.44	17.25	1665.00	2.38	31.99	238.50
Length = 24.0 ft	2	0.301	0.134	0.90	0.930	1.00	1.00	1.00	1.00	1.00	15.41	603.70	2008.46	2.38	31.99	238.50
Length = 2.0 ft	3	0.010	0.134	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.44	17.25	1665.00	0.14	31.99	238.50
+D+Lr					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 2.0 ft	1	0.017	0.219	1.25	1.000	1.00	1.00	1.00	1.00	1.00	1.00	39.19	2312.50	5.40	72.68	331.25
Length = 24.0 ft	2	0.492	0.219	1.25	0.930	1.00	1.00	1.00	1.00	1.00	35.01	1,371.57	2789.53	5.40	72.68	331.25
Length = 2.0 ft	3	0.017	0.219	1.25	1.000	1.00	1.00	1.00	1.00	1.00	1.00	39.19	2312.50	0.32	72.68	331.25

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Sacramento CA 95820
916-452-8200

Project Title:
Engineer:
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Project Descr:

Wood Beam

Project File: 2022-047.ec6

LIC# : KW-06016300, Build:20.22.7.7 POINT 2 STRUCTURAL ENGINEERING (c) ENERCALC INC 1983-2022

DESCRIPTION: East west GL

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+0.750Lr					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 2.0 ft	**1**	0.015	0.189	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.86	33.71	2312.50	4.64	62.51	331.25
Length = 24.0 ft	**2**	0.423	0.189	1.25	0.930	1.00	1.00	1.00	1.00	1.00	30.11	1,179.60	2789.53	4.64	62.51	331.25
Length = 2.0 ft	**3**	0.015	0.189	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.86	33.71	2312.50	0.27	62.51	331.25
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 2.0 ft	**1**	0.003	0.045	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.26	10.35	2960.00	1.43	19.19	424.00
Length = 24.0 ft	**2**	0.101	0.045	1.60	0.930	1.00	1.00	1.00	1.00	1.00	9.25	362.22	3570.60	1.43	19.19	424.00
Length = 2.0 ft	**3**	0.003	0.045	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.26	10.35	2960.00	0.08	19.19	424.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+Lr	-0.2094	0.000
+D+Lr	2	0.8041	12.101		0.0000	0.000
	3	0.0000	12.101	+D+Lr	-0.2094	2.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum		7.002	7.002	
Overall MINimum		3.920	3.920	
D Only		3.082	3.082	
+D+Lr		7.002	7.002	
+D+0.750Lr		6.022	6.022	
+0.60D		1.849	1.849	
Lr Only		3.920	3.920	

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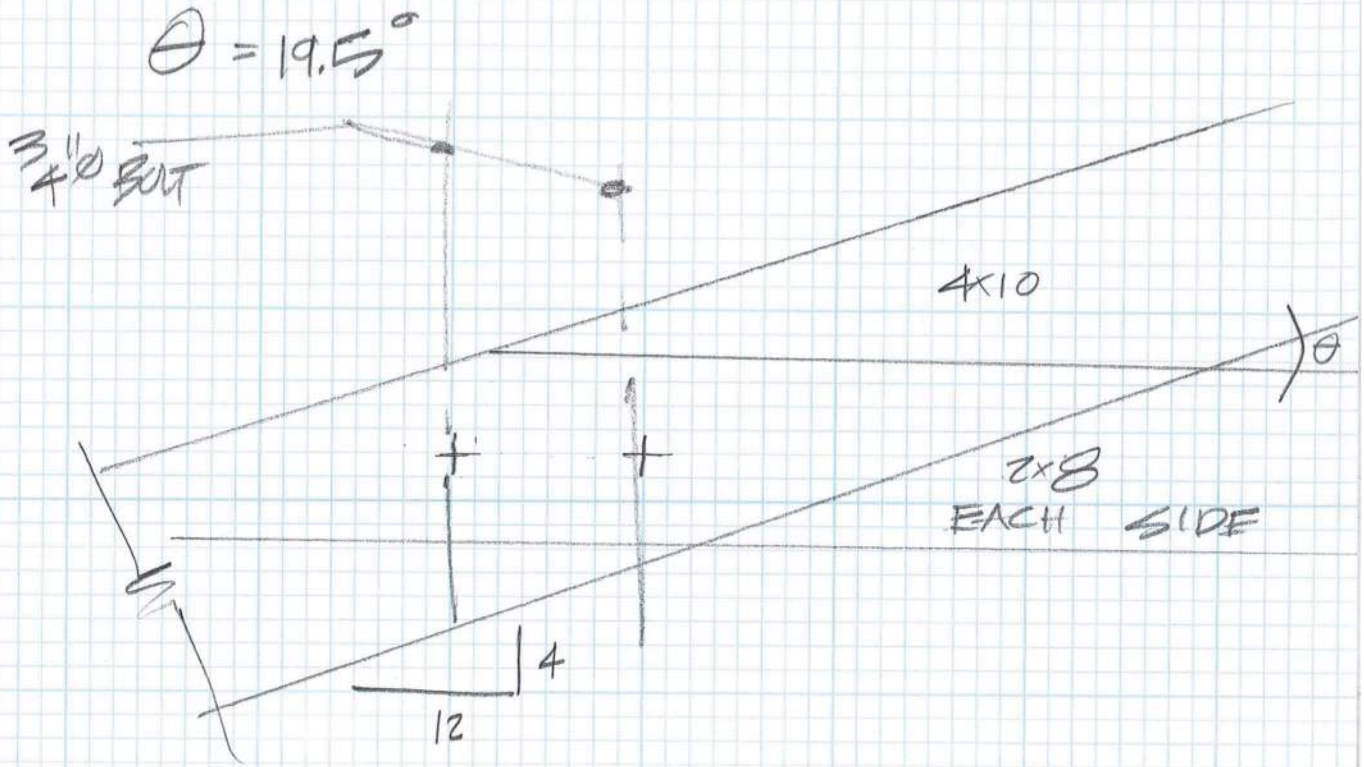
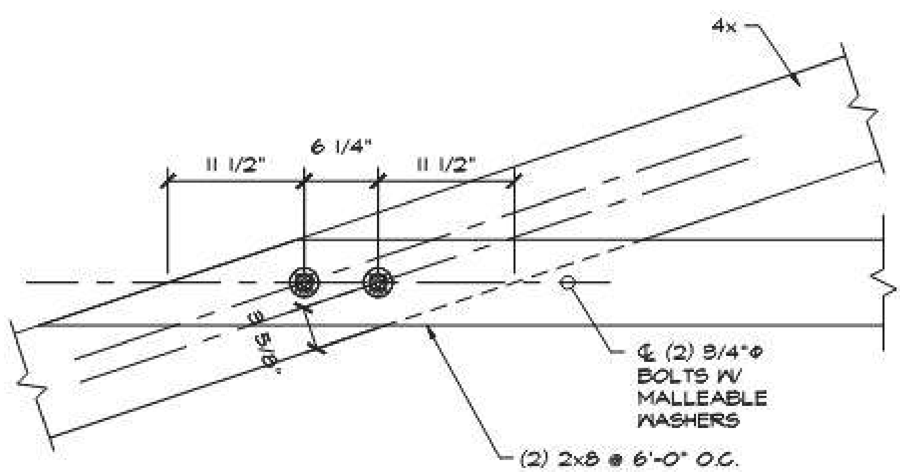




5
S3.0
DETAIL
1" = 1'-0" 044DET006

Tekla Tedds Point 2 Structural Engineers Inc 3701 Business Dr Suite 100 Sacramento, CA 95820	Project				Job Ref.	
	Section				Sheet no./rev. 1	
	Calc. by B	Date 8/3/2022	Chk'd by	Date	App'd by	Date

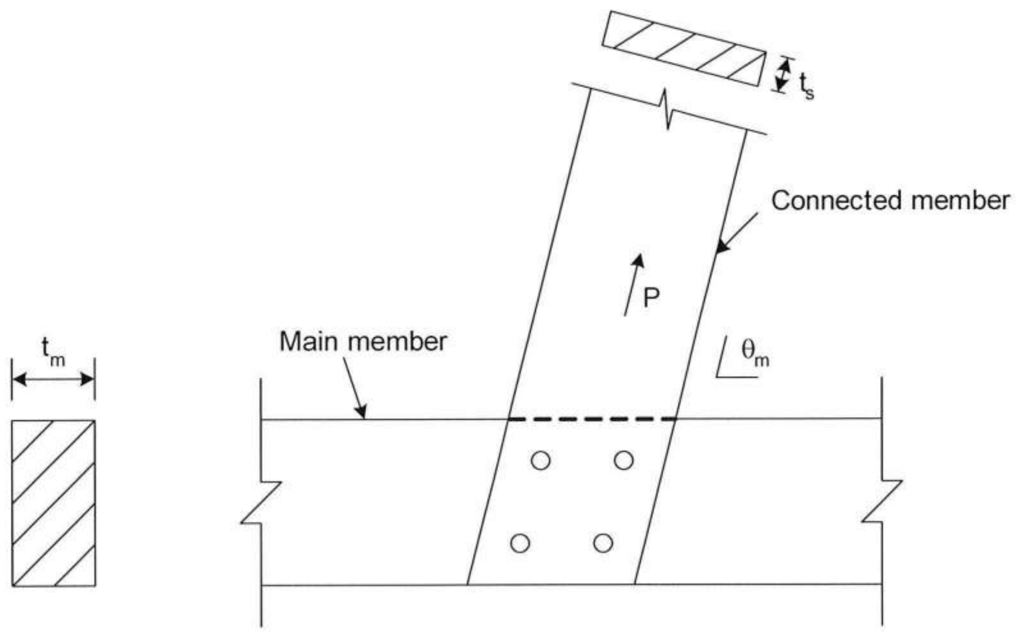
BOLTED TIMBER TO TIMBER CONNECTION DESIGN

In accordance with NDS 2018

Tedds calculation version 1.2.04

Design results summary

	Unit	Required	Provided	Utilization	Result
Connection capacity	lbs	2500	3984	0.628	PASS



Main timber member details

Species of main member **Douglas Fir-Larch**

Size of main member (Table 1B) **4 x 10**

Number of main member N_m = **1**

Thickness of main member t_m = **3.500** in

Angle of load to grain of main member θ_m = **20°**

Connected timber member details

Species of connected member **Douglas Fir-Larch**

Size of connected member (Table 1B) **2 x 8**

Number of connected member N_s = **2**

Thickness of connected member t_s = **1.500** in

Number of interfaces $N_{int} = (N_m + N_s) - 1 =$ **2**

Bolt details

Bolt diameter (Table L1) **3/4"**

Number of rows of bolts R = **1**

Number of columns of bolts C = **2**

Total number of bolts $N_{total} = R \times C =$ **2**

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Applied load

Applied load to the connection — P = **2500** lb

Dowel bearing length (main) (12.3.5)

Dowel bearing length in main member — $l_m = t_m$ = **3.500** in

Dowel bearing length (connected) (12.3.5)

Dowel bearing length in connected member — $l_s = t_s$ = **1.500** in

Bending yield strength (bolt) (Table 12A to 12I footnote no. 2)

Bending yield strength of bolt — F_{yb} = **45000** psi

Dowel bearing strength (main member) (Table 12.3.3 footnote no. 2)

Dowel bearing strength parallel to grain — $F_{e\_par} = 11200 \times G_m \times 1\text{ psi} = \mathbf{5600}$ psi

Dowel bearing strength perpendicular to grain — $F_{e\_perp} = 6100 \times G_m^{1.45} \times 1\text{ psi} / \sqrt{(D / 1\text{ in})} = \mathbf{2578}$ psi

Dowel bearing strength for small dia. fasteners — $F_e = 16600 \times G_m^{1.84} \times 1\text{ psi} = \mathbf{4637}$ psi

Dowel bearing strength at an angle of load to grain — $F_{e\theta m} = (F_{e\_par} \times F_{e\_perp}) / ((F_{e\_par} \times (\sin(\theta_m))^2) + (F_{e\_perp} \times (\cos(\theta_m))^2))$

$F_{e\theta m}$ = **4925** psi

Dowel bearing strength of main member — F_{em} = **4925** psi

Dowel bearing strength (connected timber member) (Table 12.3.3 footnote no. 2)

Dowel bearing strength parallel to grain — $F_{e\_par} = 11200 \times G_s \times 1\text{ psi} = \mathbf{5600}$ psi

Dowel bearing strength perpendicular to grain — $F_{e\_perp} = 6100 \times G_s^{1.45} \times 1\text{ psi} / \sqrt{(D / 1\text{ in})} = \mathbf{2578}$ psi

Dowel bearing strength for small dia. fasteners — $F_e = 16600 \times G_s^{1.84} \times 1\text{ psi} = \mathbf{4637}$ psi

Dowel bearing strength at an angle of load to grain — $F_{e\theta s} = (F_{e\_par} \times F_{e\_perp}) / ((F_{e\_par} \times (\sin(\theta_s))^2) + (F_{e\_perp} \times (\cos(\theta_s))^2))$

$F_{e\theta s}$ = **5600** psi

Dowel bearing strength of connected member — F_{es} = **5600** psi

Preliminary yield limit equation coefficients (Table 12.3.1A notes)

Dowel bearing strength ratio — $R_e = F_{em} / F_{es} = \mathbf{0.879}$

Dowel bearing length ratio — $R_t = l_m / l_s = \mathbf{2.333}$

Preliminary yield limit equation coefficient k_1 — $k_1 = ((\sqrt{(R_e+(2\times R_e^2\times(1+R_t+R_t^2))+(R_t^2\times R_e^3))})-(R_e\times(1+R_t)))/(1+R_e)$

k_1 = **0.708**

Preliminary yield limit equation coefficient k_2 — $k_2 = -1+\sqrt{((2\times(1+R_e))+((2\times F_{yb}\times(1+(2\times R_e))\times D^2))/(3\times F_{em}\times l_m^2))}$

k_2 = **1.129**

Preliminary yield limit equation coefficient k_3 — $k_3 = -1+\sqrt{(((2\times(1+R_e))/R_e)+((2\times F_{yb}\times(2+R_e)\times D^2))/(3\times F_{em}\times l_s^2))}$

k_3 = **1.943**

Angle of load to grain coefficient k_θ — $k_\theta = 1 + (0.25 \times \max(\theta_m, \theta_s) / 90) = \mathbf{1.056}$

Yield limit equations (double shear)

Mode I_m (eq. 12.3-7) — $Z_{Im} = (D \times l_m \times F_{em}) / (4 \times k_\theta) = \mathbf{3062}$ lb

Mode I_s (eq. 12.3-8) — $Z_{Is} = (2 \times D \times l_s \times F_{es}) / (4 \times k_\theta) = \mathbf{2984}$ lb

Mode III_s (eq. 12.3-9) — $Z_{IIIs} = (2 \times k_3 \times D \times l_s \times F_{em}) / ((2 + R_e) \times 3.2 \times k_\theta) = \mathbf{2213}$ lb

Mode IV (eq. 12.3-10) — $Z_{IV} = (2 \times D^2) \times (\sqrt{((2 \times F_{em} \times F_{yb}) / (3 \times (1 + R_e)))}) / (3.2 \times k_\theta) = \mathbf{2953}$ lb

$Z = \min(Z_{Im}, Z_{Is}, Z_{IIIs}, Z_{IV}) = \mathbf{2213}$ lb

Nominal capacity of single fastener — Z = **2213** lb

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Slenderness (Table 12.5.1C footnote no.1)

Slenderness — $l / D = \mathbf{4.000}$

Spacing requirements (perpendicular to grain loading)

End distance (Table 12.5.1A)

End distance (full stregnth) — $a_{q\_full} = 4 \times D = \mathbf{3.000}$ in

End distance (minimum) — $a_{q\_min} = 2 \times D = \mathbf{1.500}$ in

End distance (actual) — $a_q = \mathbf{3.000}$ in

Edge distance (Table 12.5.1C)

Loaded edge — $e_q = 4 \times D = \mathbf{3.000}$ in

Unloaded edge — $e_p = 1.5 \times D = \mathbf{1.125}$ in

Center to center spacing (Table 12.5.1B)

Center to center spacing (full strength) — $s_{\_full} = 4 \times D = \mathbf{3.000}$ in

Center to center spacing (minimum) — $s_{\_min} = 3 \times D = \mathbf{2.250}$ in

Center to center spacing (actual) — $s = \mathbf{3.000}$ in

Row spacing (Table 12.5.1D)

Row spacing — $s_{row} = ((5 \times l) + (10 \times D)) / 8 = 2.813$ in

Geometry factor C_Δ (12.5.1)

End distance (actual) — $a_q = \mathbf{3.000}$ in

End distance (full strength) — $a_{q\_full} = \mathbf{3.000}$ in

Geometry factor for end distance — $C_{\Delta 1} = a_q / a_{q\_full} = \mathbf{1.00}$

Center to center spacing (actual) — $s = \mathbf{3.000}$ in

Center to center spacing (full strength) — $s_{\_full} = \mathbf{3.000}$ in

Geometry factor for spacing — $C_{\Delta 2} = s / s_{\_full} = \mathbf{1.00}$

Geometry factor — $C_\Delta = \min(1, C_{\Delta 1}, C_{\Delta 2}) = \mathbf{1.00}$

Adjustment factor

Load duration factor (Table 2.3.2) — $C_D = \mathbf{0.90}$

Wet service factor (Table 11.3.3) — $C_M = \mathbf{1.0}$

Temperature factor (Table 11.3.4) — $C_t = \mathbf{1.0}$

Group action factor (eq. 11.3-1) — $C_g = \mathbf{1.0}$

Geometry factor (12.5.1) — $C_\Delta = \mathbf{1.00}$

End grain factor (12.5.2) — $C_{eg} = \mathbf{1.0}$

Diaphragm factor (12.5.3) — $C_{di} = \mathbf{1.0}$

Toe nail factor (12.5.4) — $C_{tn} = \mathbf{1.0}$

Total capacity of connection

Capacity of connection — $Z' = Z \times N_{total} \times C_D \times C_M \times C_\Delta = \mathbf{3984}$ lb

$P / Z' = \mathbf{0.628}$

Design result

PASS - Connection capacity exceeds applied load

GAZEBO COL AND FOOTING DESIGN

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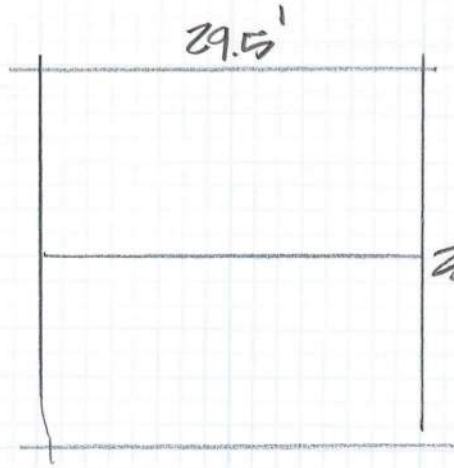
SEISMIC DESIGN



$W = 29.5(28)(13.5\ psf)$
$= 11{,}151$

$C_S = 0.5144$

$V_S = 11{,}151 \times 0.5144 = 5736$

(4) CANTILEVERED COLUMNS

$V_S/COL = 5736/4 = 1435$ COL / FNDN.

$\Omega V_S = 1.25(1435) = 1792$ - CONC ANCHORAGE

CANTILEVERED COLUMN

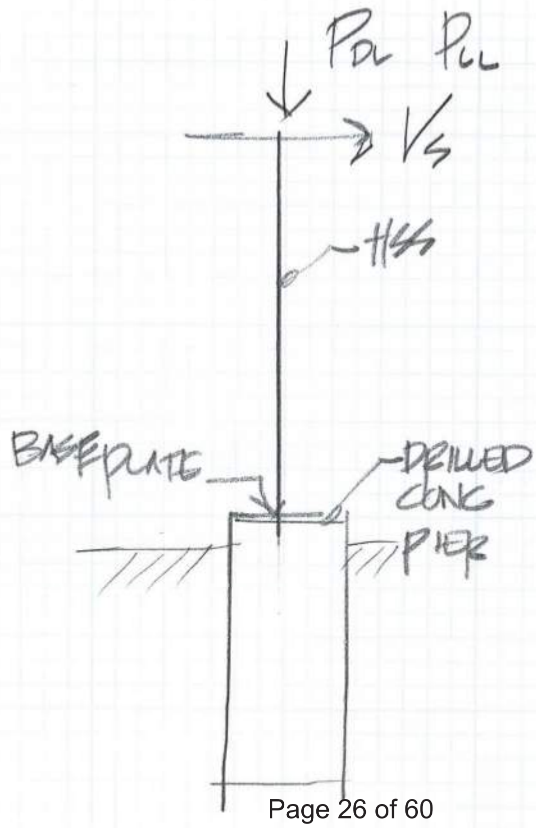
SEE PG

BASEPLATE

SEE PG

DRILLED PIER

SEE PG



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CHECK HSS POST

DL = 826 □' × 13.5 = 11,151 / 4 = 2787

LL 826 □' 20 psf / 4 = 4130

Current Date: 8/5/2022 9:26 AM
Units system: English
File name: J:\P22022\2022-047 Hamilton City Park Pavilion\RamElements\column.retx

Geometry data

GLOSSARY

Cb22, Cb33	: Moment gradient coefficients
Cm22, Cm33	: Coefficients applied to bending term in interaction formula
d0	: Tapered member section depth at J end of member
DJX	: Rigid end offset distance measured from J node in axis X
DJY	: Rigid end offset distance measured from J node in axis Y
DJZ	: Rigid end offset distance measured from J node in axis Z
DKX	: Rigid end offset distance measured from K node in axis X
DKY	: Rigid end offset distance measured from K node in axis Y
DKZ	: Rigid end offset distance measured from K node in axis Z
dL	: Tapered member section depth at K end of member
Ig factor	: Inertia reduction factor (Effective Inertia/Gross Inertia) for reinforced concrete members
K22	: Effective length factor about axis 2
K33	: Effective length factor about axis 3
L22	: Member length for calculation of axial capacity
L33	: Member length for calculation of axial capacity
LB pos	: Lateral unbraced length of the compression flange in the positive side of local axis 2
LB neg	: Lateral unbraced length of the compression flange in the negative side of local axis 2
RX	: Rotation about X
RY	: Rotation about Y
RZ	: Rotation about Z
TO	: 1 = Tension only member 0 = Normal member
TX	: Translation in X
TY	: Translation in Y
TZ	: Translation in Z

Nodes

Node	X [in]	Y [in]	Z [in]	Rigid Floor
2	0.00	138.00	0.00	0

RAM® Elements
CONNECT Edition

Current Date: 8/5/2022 9:27 AM
Units system: English
File name: J:\P22022\2022-047 Hamilton City Park Pavilion\RamElements\column.retx

Load data

GLOSSARY

Comb : Indicates if load condition is a load combination

Load Conditions

Condition	Description	Comb.	Category
DL	Dead Load	No	DL
LL	Live Load	No	LL
EQx	Seismic in X	No	EQ
D1	1.4DL	Yes	
D2	1.2DL+1.6LL	Yes	
D3	1.2DL+EQx	Yes	
D4	1.2DL+EQx+LL	Yes	
D5	0.9DL+EQx	Yes	

Load on nodes

Condition	Node	FX [Lb]	FY [Lb]	FZ [Lb]	MX [Kip*ft]	MY [Kip*ft]	MZ [Kip*ft]
DL	2	0.00	-2787.00	0.00	0.00	0.00	0.00
LL	2	0.00	-4130.00	0.00	0.00	0.00	0.00
EQx	2	1500.00	0.00	0.00	0.00	0.00	0.00

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Units system: English
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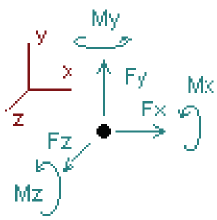
Analysis result

Reactions



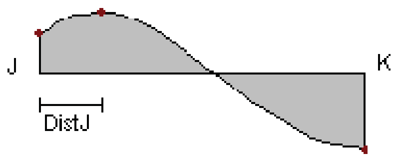
Direction of positive forces and moments

	Forces [Lb]			Moments [Kip*ft]		
Node	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
Condition **DL=Dead Load**						
1	0.00000	2787.00000	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	2787.00000	0.00000	0.00000	0.00000	0.00000
Condition **LL=Live Load**						
1	0.00000	4130.00000	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	4130.00000	0.00000	0.00000	0.00000	0.00000
Condition **EQx=Seismic in X**						
1	-1500.00000	0.00000	0.00000	0.00000	0.00000	17.25000
SUM	-1500.00000	0.00000	0.00000	0.00000	0.00000	17.25000
Condition **D1=1.4DL**						
1	0.00000	3901.80000	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	3901.80000	0.00000	0.00000	0.00000	0.00000
Condition **D2=1.2DL+1.6LL**						
1	0.00000	9952.40000	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	9952.40000	0.00000	0.00000	0.00000	0.00000
Condition **D3=1.2DL+EQx**						
1	-1500.00000	3344.40000	0.00000	0.00000	0.00000	17.25000
SUM	-1500.00000	3344.40000	0.00000	0.00000	0.00000	17.25000
Condition **D4=1.2DL+EQx+LL**						
1	-1500.00000	7474.40000	0.00000	0.00000	0.00000	17.25000
SUM	-1500.00000	7474.40000	0.00000	0.00000	0.00000	17.25000

Condition **D5=0.9DL+EQx**

1	-1500.00000	2508.30000	0.00000	0.00000	0.00000	17.25000
SUM	-1500.00000	2508.30000	0.00000	0.00000	0.00000	17.25000

Points of interest in members



Considered points

CONDITION : **DL=Dead Load**

			Plane 1-2		Plane 1-3		
	Dist to J	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[in]	[Lb]	[Lb]	[Kip*ft]	[Lb]	[Kip*ft]	[Kip*ft]
MEMBER **1**							
0%	0.000	-2787.000	0.000	0.000	0.000	0.000	0.000
100%	138.000	-2787.000	0.000	0.000	0.000	0.000	0.000

CONDITION : **LL=Live Load**

			Plane 1-2		Plane 1-3		
	Dist to J	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[in]	[Lb]	[Lb]	[Kip*ft]	[Lb]	[Kip*ft]	[Kip*ft]
MEMBER **1**							
0%	0.000	-4130.000	0.000	0.000	0.000	0.000	0.000
100%	138.000	-4130.000	0.000	0.000	0.000	0.000	0.000

CONDITION : **EQx=Seismic in X**

			Plane 1-2		Plane 1-3		
	Dist to J	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[in]	[Lb]	[Lb]	[Kip*ft]	[Lb]	[Kip*ft]	[Kip*ft]
MEMBER **1**							
0%	0.000	0.000	1500.000	-17.250	0.000	0.000	0.000
100%	138.000	0.000	1500.000	0.000	0.000	0.000	0.000

CONDITION : **D1=1.4DL**

			Plane 1-2		Plane 1-3		
	Dist to J	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[in]	[Lb]	[Lb]	[Kip*ft]	[Lb]	[Kip*ft]	[Kip*ft]
MEMBER **1**							
0%	0.000	-3901.800	0.000	0.000	0.000	0.000	0.000
100%	138.000	-3901.800	0.000	0.000	0.000	0.000	0.000

CONDITION : **D2=1.2DL+1.6LL**

				Plane 1-2		Plane 1-3	
	Dist to J	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[in]	[Lb]	[Lb]	[Kip*ft]	[Lb]	[Kip*ft]	[Kip*ft]
MEMBER **1**							
0%	0.000	-9952.400	0.000	0.000	0.000	0.000	0.000
100%	138.000	-9952.400	0.000	0.000	0.000	0.000	0.000

CONDITION : **D3=1.2DL+EQx**

				Plane 1-2		Plane 1-3	
	Dist to J	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[in]	[Lb]	[Lb]	[Kip*ft]	[Lb]	[Kip*ft]	[Kip*ft]
MEMBER **1**							
0%	0.000	-3344.400	1500.000	-17.250	0.000	0.000	0.000
100%	138.000	-3344.400	1500.000	0.000	0.000	0.000	0.000

CONDITION : **D4=1.2DL+EQx+LL**

				Plane 1-2		Plane 1-3	
	Dist to J	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[in]	[Lb]	[Lb]	[Kip*ft]	[Lb]	[Kip*ft]	[Kip*ft]
MEMBER **1**							
0%	0.000	-7474.400	1500.000	-17.250	0.000	0.000	0.000
100%	138.000	-7474.400	1500.000	0.000	0.000	0.000	0.000

CONDITION : **D5=0.9DL+EQx**

				Plane 1-2		Plane 1-3	
	Dist to J	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[in]	[Lb]	[Lb]	[Kip*ft]	[Lb]	[Kip*ft]	[Kip*ft]
MEMBER **1**							
0%	0.000	-2508.300	1500.000	-17.250	0.000	0.000	0.000
100%	138.000	-2508.300	1500.000	0.000	0.000	0.000	0.000

RAM® Elements
CONNECT Edition

Current Date: 8/5/2022 9:29 AM
Units system: English
File name: J:\P22022\2022-047 Hamilton City Park Pavilion\RamElements\column.retx

Steel Code Check

Report: Comprehensive

Members: Hot-rolled
Design code: AISC 360-2016 LRFD

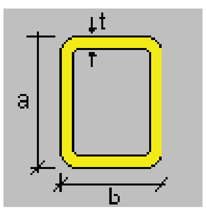
Member	:	**1**
Design status	:	**OK**

DESIGN WARNINGS

Section information

Section name: HSS_SQR 6X6X3_8 **(US)**

Dimensions



a	=	6.000	[in]	Height
b	=	6.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties

Section properties	**Unit**	**Major axis**	**Minor axis**
Gross area of the section. (Ag)	[in2]	7.580	
Moment of Inertia (local axes) (I)	[in4]	39.500	39.500
Moment of Inertia (principal axes) (I')	[in4]	39.500	39.500
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	2.283	2.283
Radius of gyration (principal axes) (r')	[in]	2.283	2.283
Saint-Venant torsion constant. (J)	[in4]	64.600	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	13.200	13.200
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	13.200	13.200
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	13.200	13.200
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	13.200	13.200
Plastic section modulus (local axis) (Z)	[in3]	15.800	15.800
Plastic section modulus (principal axis) (Z')	[in3]	15.800	15.800
Polar radius of gyration. (ro)	[in]	3.225	
Area for shear (Aw)	[in2]	3.457	3.457
Torsional constant. (C)	[in3]	22.122	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[in]	138.00

Distance between member lateral bracing points

Length (Lb) [in]	
Top	Bottom
138.00	138.00

Laterally unbraced length

Length [in]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
138.00	138.00	138.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✔

Axial tension

Ratio : 0.00
Capacity :341100.00 [Lb] Reference : Cl.D2
Demand : 0.00 [Lb] Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n)	[Lb]	341100.00	Cl.D2
Nominal axial tension capacity (P_n)	[Lb]	379000.00	Eq.D2-1

AXIAL COMPRESSION DESIGN ✔

Compression in the major axis 33

Ratio	: 0.04		
Capacity	:261116.60 [Lb]	Reference	: Cl.E3
Demand	: 9952.40 [Lb]	Ctrl Eq.	: D2 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.19	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.19	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.4.1a.Case6
Factored flexural buckling strength(ϕP_{n33})	[Lb]	261116.60	Cl.E3
Unbraced length (L_{33})	[in]	138.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	60.45	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	78.32	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.58	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	38.28	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Lb]	290129.60	Eq.E3-1

Compression in the minor axis 22

Ratio	: 0.04		
Capacity	:261116.60 [Lb]	Reference	: Cl.E3
Demand	: 9952.40 [Lb]	Ctrl Eq.	: D2 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.19	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.19	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.4.1a.Case6
Factored flexural buckling strength(ϕP_{n22})	[Lb]	261116.60	Cl.E3
Unbraced length (L_{22})	[in]	138.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	60.45	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	78.32	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	7.58	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	38.28	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Lb]	290129.60	Eq.E3-1

FLEXURAL DESIGN ✔

Bending about major axis, M33

Ratio	: 0.29		
Capacity	: 59.25 [Kip*ft]	Reference	: Cl.F7.1
Demand	: -17.25 [Kip*ft]	Ctrl Eq.	: D3 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n)	[Kip*ft]	59.25	Cl.F7.1
Yielding (M_n)	[Kip*ft]	65.83	Eq.F7-1

Bending about minor axis, M22

Ratio	: 0.00		
Capacity	: 59.25 [Kip*ft]	Reference	: Cl.F7.1
Demand	: 0.00 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n)	[Kip*ft]	59.25	Cl.F7.1
Yielding (M_n)	[Kip*ft]	65.83	Eq.F7-1

DESIGN FOR SHEAR ✔

Shear in major axis 33

Ratio	: 0.00		
Capacity	: 93344.24 [Lb]	Reference	: Cl.G1
Demand	: 0.00 [Lb]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n)	[Lb]	93344.24	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Lb]	103715.80	Eq.G4-1

Shear in minor axis 22

Ratio	: 0.02		
Capacity	: 93344.24 [Lb]	Reference	: Cl.G1
Demand	: 1500.00 [Lb]	Ctrl Eq.	: D3 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n)	[Lb]	93344.24	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Lb]	103715.80	Eq.G4-1

TORSION DESIGN ✔

Torsion

Ratio	: 0.00		
Capacity	: 49.77 [Kip*ft]	Reference	: Cl.H3.1
Demand	: 0.00 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

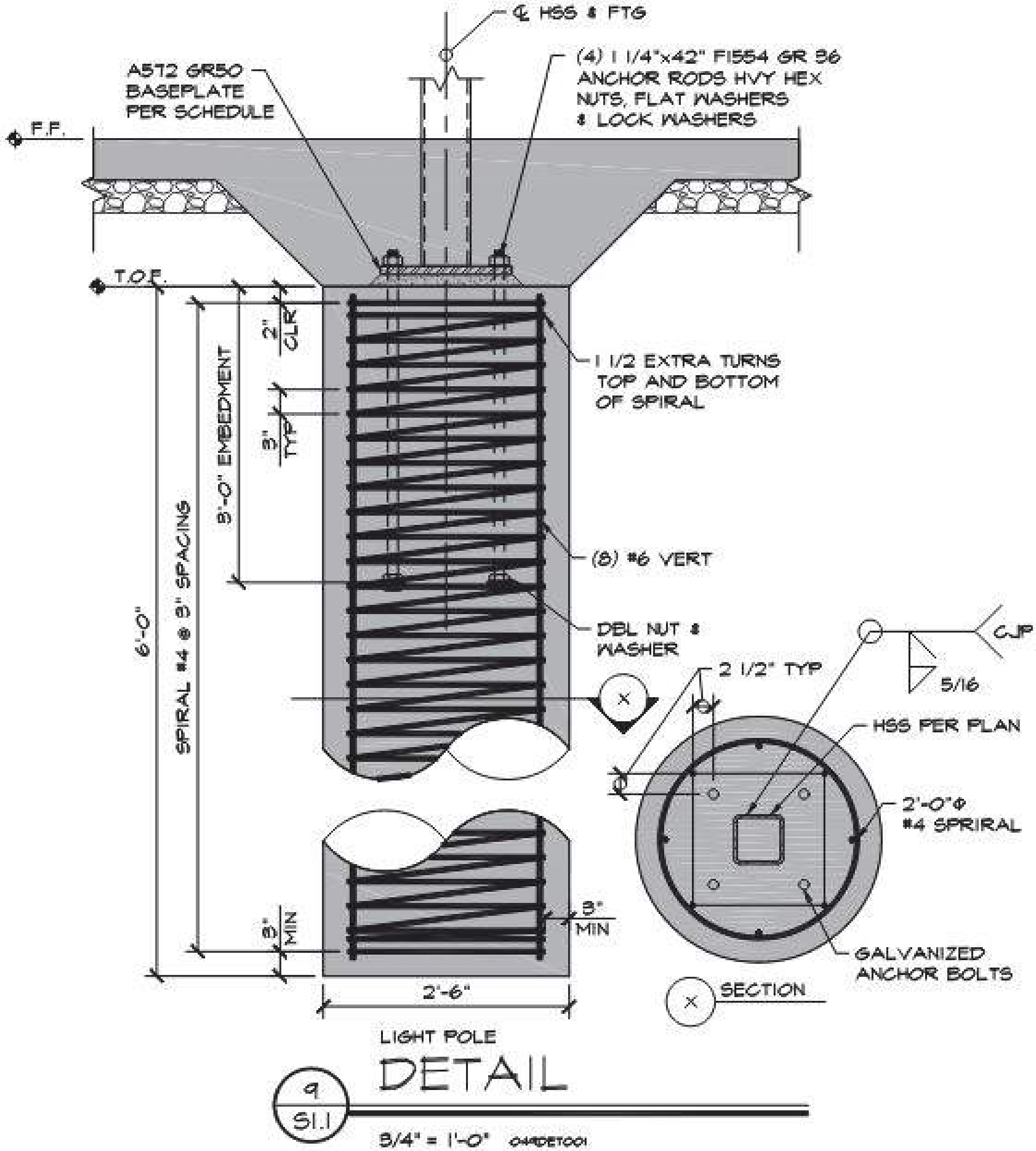
Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n)	[Kip*ft]	49.77	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	55.30	Eq.H3-1

COMBINED ACTIONS DESIGN ✔

Combined flexure and axial

Ratio	: 0.31		
Ctrl Eq.	: D4 at 0.00%	Reference	: Eq.H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force	--	0.31	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	59.25	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	59.25	Cl.H1.1
Available axial strength (P_c)	[Lb]	261116.60	Cl.H1.1

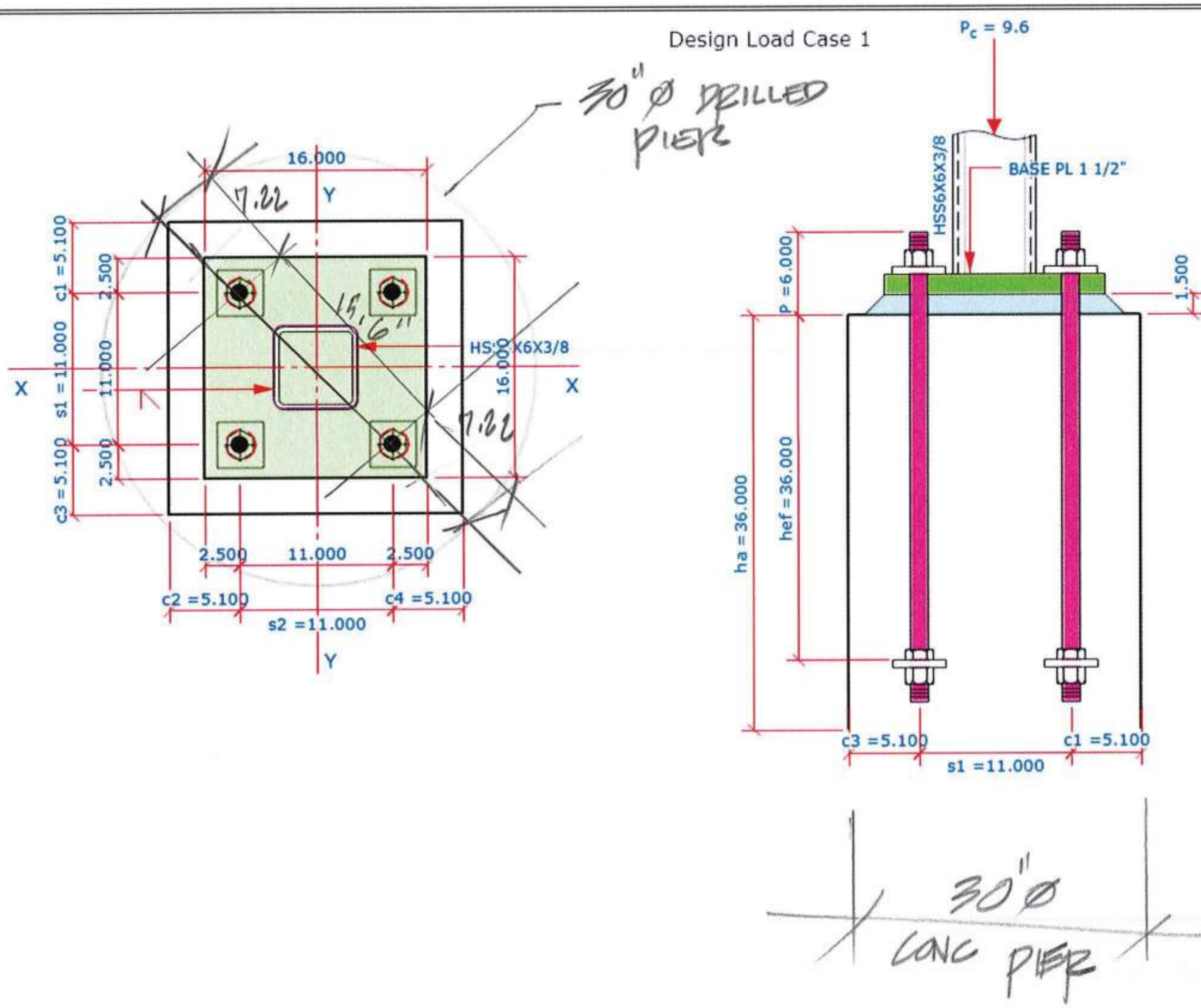


℄ HSS & FTG
A572 GR50 BASEPLATE PER SCHEDULE
(4) 1 1/4"x42" F1554 GR 36 ANCHOR RODS HVY HEX NUTS, FLAT WASHERS & LOCK WASHERS
F.F.
T.O.F.
2" CLR
3" TYP
3'-0" EMBEDMENT
SPIRAL #4 @ 3" SPACING
6'-0"
1 1/2 EXTRA TURNS TOP AND BOTTOM OF SPIRAL
(8) #6 VERT
DBL NUT & WASHER
2 1/2" TYP
CJP
5/16
HSS PER PLAN
2'-0"Φ #4 SPRIRAL
3" MIN
3" MIN
GALVANIZED ANCHOR BOLTS
2'-6"
SECTION
LIGHT POLE
DETAIL
9
S1.1
3/4" = 1'-0"

Column Baseplate

Result Summary - Overall	Anchorage Design		Code=ACI 318-19	
Result Summary - Overall		geometries & weld limitations = **PASS**	limit states max ratio = **0.86**	**PASS**
Anchor Bolt - LC 1	**$P + V_x + M_y$**	geometries & weld limitations = **PASS**	limit states max ratio = **0.37**	**PASS**
Base Plate - LC 1	**$P + M_x$**	geometries & weld limitations = **PASS**	limit states max ratio = **0.86**	**PASS**
Base Plate - LC 1	**$P + M_y$**	geometries & weld limitations = **PASS**	limit states max ratio = **0.86**	**PASS**

Sketch Anchorage Design Code=ACI 318-19



Anchor Forces Calculation

Anchor Tensile Force Calculation

User Input

Anchor edge distance

c_{1u} = 5.100 [in] c_{2u} = 5.100 [in]

c_{3u} = 5.100 [in] c_{4u} = 5.100 [in]

Anchor out-out spacing

s_{1u} = 11.000 [in] s_{2u} = 11.000 [in]

Anchor embedment depth

h_{ef} = 36.000 [in]

Design Load - Load Case 1

Axial force

Axial P = 9.60 [kips] in compression

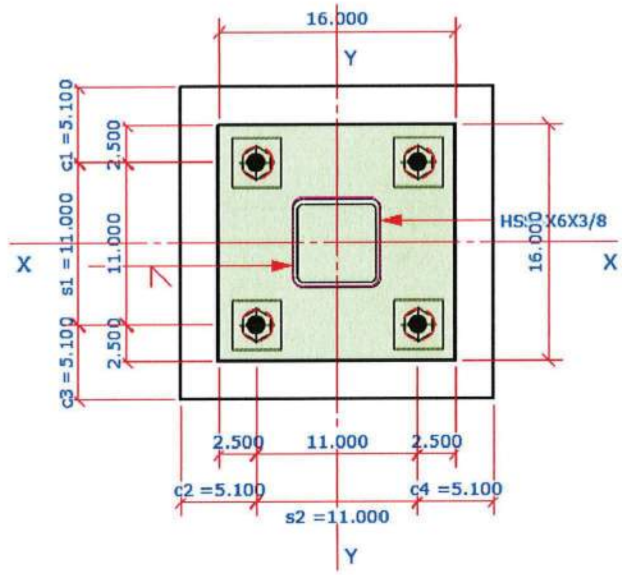
Shear forces

V_y = 0.00 [kips] V_x = 1.90 [kips]

Moment forces

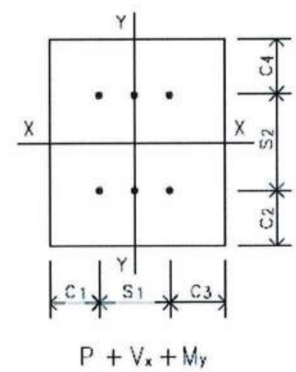
M_x = 0.00 [kip-ft] M_y = 21.60 [kip-ft]

Anchor Layout Plan



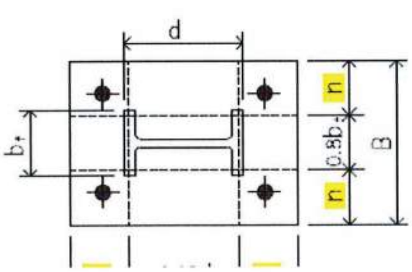
Load Case 1 - Check on P + V_x + M_y

Anchor edge distance	c_1 = 5.100	[in]	c_2 = 5.100	[in]
	c_3 = 5.100	[in]	c_4 = 5.100	[in]
Anchor out-out spacing	s_1 = 11.000	[in]	s_2 = 11.000	[in]
Anchor group load	P_u = 9.60	[kips]	V_u = 1.90	[kips]
	M_u = 21.60	[kip-ft]		



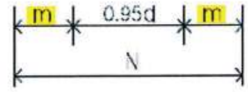
Max Allowed Concrete Pressure

Column sect HSS6X6X3/8	d = 6.000	[in]	b_f = 6.000	[in]
Base plate width & depth	B = 16.000	[in]	N = 16.000	[in]
Pedestal width & depth	b_c = 21.200	[in]	d_c = 21.200	[in]
Base plate area	$A_1 = B \times N$		= 256.00	[in^2]
Pedestal area	$A_2 = b_c \times d_c$		= 449.44	[in^2]



ACI 318-19 Table 14.5.6.1

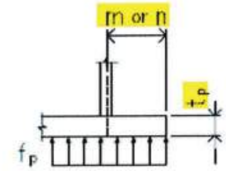
$k = \min(\sqrt{A_2/A_1}, 2)$ = 1.325



AISC Design Guide 1 - 3.1.2 on Page 15

Base plate cantilever dimension $m = (N - 0.95\,d) / 2$ = **5.150** [in]

$n = (B - 0.95\,b_f) / 2$ = 5.150 [in]



				ACI 318-19
Concrete strength & strength reduction factor	$f_c = 3.0$ [ksi]	$\phi_c = 0.65$		Table 21.2.1 (d)
				AISC Design Guide 1
Pedestal max allowed bearing stress	$f_{p(max)} = \phi_c k\, 0.85\, f_c$	= 2.196	[ksi]	3.1.1 on Page 14
	$q_{max} = f_{p(max)} \times B$	= 35.14	[kip/in]	Eq 3.3.4 on Page 23
Factored forces on base plate	$P_u = 9.60$ [kips]	$M_u = 21.60$	[kip-ft]	
Eccentricity	$e = M_u / P_u$	= **27.000**	[in]	
Critical eccentricity	$e_{crit} = N/2 - P_u / (2\, q_{max})$	= 7.863	[in]	Eq 3.3.7 on Page 24
	when $e > e_{crit}$, **large moment case applied**			Step 3 on Page 27

To achieve forces equilibrium in base plate to point A as shown in sketch on the bottom right, the actual concrete bearing stress f_p can be any value $f_p \le f_{p(max)}$ = 2.196 ksi

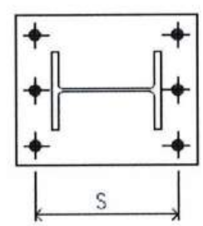
Pedestal actual bearing stress used	$f_p =$	= **2.196**	[ksi]
	$q_{max} = f_p \times B$	= **35.14**	[kip/in]

Anchor Tensile Force Calc - Group Anchor Subject to Moment

Design Basis and Assumptions

1. Assume base plate is rigid and anchor tensile forces are elastic linearly distributed as shown on the right.
2. The concrete bearing stress is assumed to be uniformly distributed as per AISC Design Guide 1 section 3.3.1

User can select the option of base plate thickness $t_p \ge$ (max of base plate overhangs m or n) / 4 in Anchor Bolt - Config & Setting to ensure that base plate has adequate rigidity to match above assumptions.



Anchor Bolt Spacings

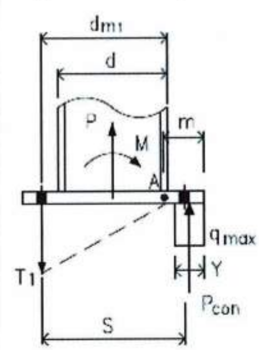
Anchor bolt pattern	= 4B		
Anchor bolt out-out spacing & column depth	s = 11.000 [in]	d = 6.000	[in]

Loads on Anchor Group

Anchor group load	$P_u = 9.60$ [kips] (C)	$M_u = 21.60$	[kip-ft]

Along Anchor Bolt Line - Single Anchor Tensile T_i & No of Anchor Bolt n_i

Anchor bolt line - moment arm	$d_{m1} = 8.500$ [in]		
Bolt line 1 - single anchor T_1	$T_1 =$ **6.91** [kips]	$n_1 =$ **2**	
Sum of anchors tensile force	$T_u = n_1 T_1$	= **13.83**	[kips]
No of anchors in anchor group resisting tension	$n_t = n_1$	= 2	
Resistance moment by tensile anchors	$M_{ra} = n_1 T_1 d_{m1}$	= **9.79**	[kip-ft]



2 BOLT LINE

Moment by Concrete Pressure Reaction

Take the moment of concrtete pressure resultant P_{con} to column flange/base plate intersect point A as shown on above sketch on the right

Max allowed concrete pressure	q_{max} = from above calculation	= 35.14	[kip/in]
Base plate cantilever length	m = from above calculation	= 5.150	[in]
Concrete pressure block length	Y =	= **0.667**	[in]
Concrete pressure stress resultant	$P_{con} = q_{max} Y$	= 23.44	[kips]
Resistance moment by concrete reaction	$M_{rc} = P_{con} \times (m - 0.5Y)$	= **9.41**	[kip-ft]

Below two sections are for verification purpose only. We want to verify that the anchor tensile forces and concrete pressure block length Y shown above make the base plate achieving force equilibrium

Verify Vertical Force Equilibrium

Tensile anchors reaction on base plate - downward	$P_{ar} = n_1 T_1$	= 13.83	[kips]
Base plate compressive load-downward	P_u = from user load input	= 9.60	[kips]
Sum of downward forces on base plate	$P_{dn} = P_{ar} + P_u$	= **23.43**	[kips]
Concrete pressure reaction on base plate - upward	$P_{con} = q_{max} Y$	= 23.44	[kips]
Sum of upward forces on base plate	$P_{up} = P_{con}$	= **23.44**	[kips]

Conclusion : the vertical forces equilibrium is achieved

Verify Overturn/Resistance Moment Equilibrium

Take moment to column edge point A as rotating point as shown on sketch above

Resistance moment by tensile anchors downward reaction forces	$M_{ra} = n_1 T_1 d_{m1}$	= 9.79	[kip-ft]
Resistance moment by concrete pressure reaction force	$M_{rc} = P_{con} \times (m - 0.5Y)$	= 9.41	[kip-ft]
Sum of resistance moment	$= M_{ra} + M_{rc}$	= **19.20**	[kip-ft]
Load on base plate	P_u = 9.60 [kips]	M_u = 21.60	[kip-ft]
Column sect HSS6X6X3/8	d = 6.000 [in]		
Sum of moments from base plate loads taken to point A	$= M_u - P_u \times 0.5\ d$	= **19.20**	[kip-ft]

Conclusion : the summation of moments taken about point A equals to zero

Anchor Bolt - Load Case 1 **$P + V_x + M_y$** P_c=9.6 kip V_x=1.9 kip M_y=21.6 kip-ft Code=ACI 318-19

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.37**	**PASS**

Min Anchor Dimensions Check Per PIP STE05121 - Optional **PASS**

Min Anchor Dimensions Check

Check min anchor dimensions as per PIP STE05121 Application of ASCE Anchorage Design for Petrochemical Facilities - 2018 Table 1 as shown below.

This check is NOT a code requirement. User can turn this check On/Off by changing setting at Anchor Bolt --> Anchor Bolt - Config & Setting --> Check min anchor spacing and edge distance as per PIP STE05121 Table 1

Anchor Rod Inputs

Anchor rod grade and dia	grade = F1554 Gr36	$d_a = 1^1/_4$	[in]	

Min Anchor Edge Distance

Anchor edge distance	c_1 = 5.100 [in]	c_2 = 5.100	[in]	
	c_3 = 5.100 [in]	c_4 = 5.100	[in]	
Min anchor edge distance required	c_{min} = from PIP STE05121 Table 1 below	= **5.000**	[in]	PIP STE05121 Table 1
Min anchor edge distance	$c = \min(c_1, c_2, c_3, c_4)$	= **5.100**	[in]	
		$\geq c_{min}$	OK	

Min Anchor Spacing

Min anchor spacing required	s_{min} = from PIP STE05121 Table 1 below	= **5.000**	[in]	PIP STE05121 Table 1
Anchor bolt pattern	= from user input	= 4B		
Min anchor spacing	s = from user input	= **11.000**	[in]	
		$\geq s_{min}$	OK	

Min Anchor Embedment Depth

Min anchor embedment required	h_{min} = from PIP STE05121 Table 1 below	= **15.000**	[in]	PIP STE05121 Table 1
Min anchor embedment depth	h_{ef} = from user input	= **36.000**	[in]	
		$\geq h_{min}$	OK	

Table 1 from PIP STE05121 Application of ASCE Anchorage Design for Petrochemical Facilities - 2018

PIP STE05121
Application of ASCE Anchorage Design for Petrochemical Facilities

EDITORIAL REVISION
January 2018

Table 1 - Minimum Anchor Dimensions – U.S. Customary Units

(See Figure 1 for dimension locations)

ANCHOR ROD DIAMETER	EFFECTIVE CROSS-SECTIONAL AREA OF ANCHOR ROD IN TENSION (Note 3)	HEAVY HEX HEAD/ NUT WIDTH	ANCHOR TYPE 2 THREAD LENGTH AT BOTTOM OF ANCHOR		*ASCE ANCHORAGE DESIGN REPORT* MINIMUM DIMENSIONS (Note 1)				SLEEVES (See Note 1 (d))	
					h_{ef}	EDGE DISTANCE c_a (Note 2)		SPAC-ING		
			WITH NO AP	WITH AP (Note 4)	$12d_a$	*A307/ A36 F1554* GRADE 36	HIGH-STRENGTH (> 36 ksi) OR TORQUED ANCHORS	$4d_a$	SHELL SIZE	h'_e

d_a	$A_{se,N}$	W_h	TB1	TB2		$4d_a \geq 4.5"$	$6d_a \geq 4.5"$		Diam d_s	Height h_s	$6d_a \geq 6"$
in.	in²	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.
5/8	0.226	1.25	1.25	--	7.5	4.5	4.5	2.5	2	7	6
3/4	0.334	1.44	1.25	2.25	9.0	4.5	4.5	3.0	2	7	6
7/8	0.462	1.69	1.50	2.50	10.5	4.5	5.3	3.5	2	7	6
1	0.606	1.88	1.75	3.00	12.0	4.5	6.0	4.0	3	10	6
1-1/4	0.969	2.31	2.00	3.50	15.0	5.0	7.5	5.0	---	---	---
1-1/2	1.405	2.75	2.25	4.00	18.0	6.0	9.0	6.0	---	---	---
1-3/4	1.900	3.19	2.50	4.75	21.0	7.0	10.5	7.0	---	---	---
2	2.500	3.63	2.75	5.25	24.0	8.0	12.0	8.0	---	---	---
2-1/4	3.250	4.06	3.00	5.75	27.0	9.0	13.5	9.0	---	---	---
2-1/2	4.000	4.50	3.50	6.50	30.0	10.0	15.0	10.0	---	---	---
2-3/4	4.930	4.94	3.75	7.00	33.0	11.0	16.5	11.0	---	---	---
3	5.970	5.31	4.00	7.75	36.0	12.0	18.0	12.0	---	---	---

NOTES:
1. If sleeves are used, the following dimensional modifications apply:
   (a) Embedment should be the greater of $12d_a$ or (h_s + h'_e)
   (b) Edge distance should be increased by $0.5(d_s - d_a)$
   (c) Spacing should be increased by $(d_s - d_a)$
   (d) Partial length sleeves are not recommended for anchors greater than 1 in. See *ASCE Anchorage Design Report*, Section 3.2.3.1.

Minimum Anchor Dimensions Check **PASS**

Min Anchor Dimensions Check

When anchor reinforcement or supplementary reinforcement is provided, the check on min edge distance is not required. Only min anchor spacing is checked as per ACI 318-19 Table 17.9.2(a) — ACI 318-19 17.9.1

ACI 318-19 17.9.1

17.9—Edge distances, spacings, and thicknesses to preclude splitting failure

17.9.1 Minimum spacings and edge distances for anchors and minimum thicknesses of members shall conform to this section, unless supplementary reinforcement is provided to control splitting. Lesser values from product-specific tests

Anchor Rod Inputs

Anchor rod grade and dia	grade = F1554 Gr36	$d_a = 1^1/_4$	[in]

Min Anchor Spacing

Min anchor spacing required	$s_{min} = 4 \times d_a$ for Not Torqued case	= **5.000**	[in]	ACI 318-19 Table 17.9.2(a)
Anchor bolt pattern	= from user input	= 4B		
Min anchor spacing	s = from user input	= **11.000**	[in]	
		$\geq s_{min}$	OK	

ACI 318-19 Table 17.9.2(a)

Table 17.9.2(a)—Minimum spacing and edge distance requirements

Spacing parameter	Anchor type: Cast-in anchors: Not torqued	Cast-in anchors: Torqued	Post-installed expansion and undercut anchors	Post-installed screw anchors
Minimum anchor spacing	$4d_a$	$6d_a$	$6d_a$	Greater of $0.6h_{ef}$ and $6d_a$
Minimum edge distance	Specified cover requirements for reinforcement according to 20.5.1.3	$6d_a$	Greatest of (a), (b), and (c): (a) Specified cover requirements for reinforcement according to 20.5.1.3 (b) Twice the maximum aggregate size (c) Minimum edge distance requirements according to ACI 355.2 or 355.4, or Table 17.9.2(b) when product information is absent	

Anchor Rod Tensile Resistance ratio = 6.9 / 42.2 = **0.16** **PASS**

Anchor rod effective section area $A_{se} = 0.97$ [in^2] $f_{uta} = 58.0$ [ksi]

Anchor rod steel strength in tension $N_{sa} = A_{se} f_{uta}$ = 56.20 [kips] ACI 318-19 17.6.1.2

Max Single Anchor Tensile Force

Refer to Anchor Forces Calculation section above for the detail calculation on how to ge the max single anchor tensile force as shwon below

Max single anchor tensile force T = from Anchor Forces Calculation above = **6.91** [kips]

Strength reduction factor $\phi_{ts} = 0.75$ ACI 318-19 17.5.3(a)

$\phi_{ts} N_{sa} = 0.75 \times 56.20$ = **42.15** [kips]

ratio = **0.16** > T OK

Anchor Reinforcement Tensile Breakout Resistance ratio = 13.8 / 111.6 = **0.12** **PASS**

Concrete & rebar strength $f_c = 3.0$ [ksi] $f_y = 60.0$ [ksi]

Ver rebar no & area bar no = #5 $A_s = 0.31$ [in^2]

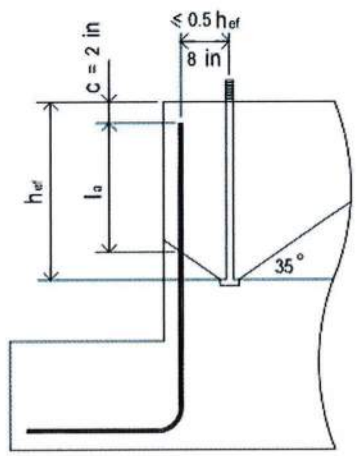
$d_b = 5/8"$

$\Psi_e = 1.0$ $\Psi_r = 1.0$ ACI 318-19 Table 25.4.3.2

$\Psi_o = 1.0$ $\Psi_c = 0.80$

Hook rebar development length $l_{dh} = \max\left(\frac{f_y \Psi_e \Psi_r \Psi_o \Psi_c}{55 \lambda \sqrt{f_c}} d_b^{1.5}, 8d_b, 6"\right)$ = **7.873** [in] ACI 318-19 25.4.3.1

When anchor reinforcement is used, rebar development length on both sides of concrete failure breakout line shall meet the min. development length requirement



Anchor embedment depth h_{ef} = from user input = 36.000 [in]

Avg ver. bar center to anchor rod center distance d_{ar} = from user input = 4.000 [in]

ACI 318-19 25.4.3.1

Min rebar development length required $l_{min} = \max(8d_b, 6")$ = **6.000** [in]

Actual rebar development length $l_a = h_{ef}$ - top cover (2") - d_{ar} tan35 = **31.199** [in]

ratio = **0.19** > l_{min} OK ACI 318-19 25.4.3.1

Anchor group tensile load N_u = from Anchor Forces Calculation above = **13.83** [kips]

No of ver rebar effective to resist anchor tension n_v = from user input = 8.0

Rebar resistance factor $\phi_s = 0.75$ ACI 318-19 17.5.3

Anchor reinft breakout resistance $\phi N_n = \phi_s n_v A_s f_y$ = **111.60** [kips] ACI 318-19 17.5.2.1 (a)

ratio = **0.12** > N_u OK

Anchor Pullout Resistance ratio = 6.9 / 37.6 = **0.25** **PASS**

Anchor head net bearing area & conc strength	A_{brg} = 2.24 [in^2]	f_c = 3.0	[ksi]	
Single bolt pullout resistance	$N_p = 8\,A_{brg}\,f_c$	= 53.69	[kips]	ACI 318-19 17.6.3.2.2a
Pullout cracking factor	Ψ_{cP} = for cracked concrete	= 1.00		ACI 318-19 17.6.3.3.1(b)

Max Single Anchor Tensile Force

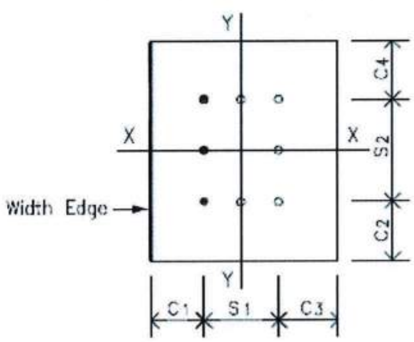
Refer to Anchor Forces Calculation section above for the detail calculation on how to ge the max single anchor tensile force as shwon below

Max single anchor tensile force	T = from Anchor Forces Calculation above	= **6.91**	[kips]	
Strength reduction factor	ϕ_{tc} = 0.70 pullout strength is always Condition B			ACI 318-19 17.5.3(c)
	$\phi_{tc} N_{pn} = \phi_{tc} \Psi_{cP} N_p$	= **37.58**	[kips]	
Seismic design strength reduction	= x 0.75 applicable	= **28.19**	[kips]	ACI 318-19 17.10.5.4(c)
	ratio = **0.25**	> T	OK	

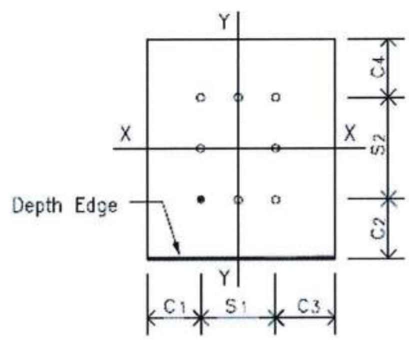
Anchor Side Blowout Resistance ratio = 6.9 / 18.8 = **0.37** **PASS**

Anchor Inputs

Anchor edge distance	c_1 = 5.100 [in]	c_2 = 5.100 [in]
	c_3 = 5.100 [in]	c_4 = 5.100 [in]
Anchor out-out spacing	s_1 = 11.000 [in]	s_2 = 11.000 [in]



Side Blowout Width Edge



Side Blowout Depth Edge

Side Edges Along Y-Y Axis - Width Edges

Anchor edge distance in X direction	$c_{a1} = \min(c_1, c_3)$	= 5.100	[in]	
Anchor embedment depth	h_{ef} = from user input	= 36.000	[in]	
Side blowout check is required on this edge or not	= check if $h_{ef} > 2.5\,c_{a1}$	= True		ACI 318-19 17.6.4.1
	Side blowout check is required			ACI 318-19 17.6.4.1
Anchor out-out distance edges along Y direction	s_2 = from user input	= 11.000	[in]	
Anchor number along Y direction	n_w = from user input	= 2		
Anchor head net bearing area & conc strength	A_{brg} = 2.24 [in^2]	f_c = 3.0	[ksi]	
Lightweight conc modification factor	$\lambda = 1.0$			ACI 318-19 17.2.4.1
Single anchor side blowout capacity	$N_{sb} = 160\,c_{a1}\sqrt{A_{brg}}\,\lambda\sqrt{f_c}$	= 66.85	[kips]	ACI 318-19 17.6.4.1

For multiple anchors along the edge, check if the anchor spacing is close enough so that side blowout capacity shall be calculated as a group ACI 318-19 17.6.4.2

Anchor spacing along Y-Y edges	$s_b = s_2 / (n_w - 1)$	= 11.000	[in]

Multiple tensile anchors space close and work as group or not	= check if $s_b < 6\ c_{a1}$	= True	ACI 318-19 17.6.4.2
Multiple anchors group factor	$= 1 + \frac{s_2}{6c_{a1}}$	= 1.36	ACI 318-19 17.6.4.2
Group anchor side blowout capacity	$N_{sbg} = (1 + \frac{s_2}{6c_{a1}})\ N_{sb}$	= **90.88** [kips]	

Refer to Anchor Forces Calculation section above for the detail calculation on how to ge the max single anchor tensile force as shwon below

Max single anchor tensile force & no of anchors along blowout edge	$T_1 = 6.91$ [kips]	$n_1 = 2$	
Tensile force - anchors along potential blowout edge	$T_w = n_1 T_1$	= **13.83** [kips]	
Strength reduction factor	$\phi_{tc} = 0.75$ supplementary reinft present		ACI 318-19 17.5.3(b)
	$\phi_{tc} N_{sbg} = 0.75 \times 90.88$	= 68.16 [kips]	
Seismic design strength reduction	= x 0.75 applicable	= **51.12** [kips]	ACI 318-19 17.10.5.4(d)
	ratio = **0.27**	$> T_w$ OK	

When there are tensile anchors in the group which are not located on blowout edge, we need to use edge anchors capacity above to work out anchor group tensile capacity

Group anchor no & no of anchor along blowout edge	$n_t = 2$	$n_{bw} = 2$	
Group anchor tensile side blowout capacity	$= 51.12 \frac{n_t}{n_{bw}}$	= **51.12** [kips]	

Side Edges Along X-X Axis - Depth Edges

Anchor edge distance in Y direction	$c_{a2} = \min(c_2, c_4)$	= 5.100 [in]	
Anchor embedment depth	h_{ef} = from user input	= 36.000 [in]	
Side blowout check is required on this edge or not	= check if $h_{ef} > 2.5\ c_{a2}$	= True	ACI 318-19 17.6.4.1
	Side blowout check is required		ACI 318-19 17.6.4.1
Anchor head net bearing area & conc strength	$A_{brg} = 2.24$ [in^2]	$f_c = 3.0$ [ksi]	
Lightweight conc modification factor	$\lambda = 1.0$		ACI 318-19 17.2.4.1
Single anchor side blowout capacity	$N_{sb} = 160\ c_{a2} \sqrt{A_{brg}}\ \lambda \sqrt{f_c}$	= 66.85 [kips]	ACI 318-19 17.6.4.1

When only single anchor in a row of multiple anchors mobilizes tensile for side blowout check , this single anchor has an increased edge distance c_3 by adding s1

Anchor edge distance - after c_3 been adjusted	$c_1 = 5.100$ [in]	$c_3 = 16.100$ [in]	

When amchor edge distance c_1, c_3 are small, when c_1 or $c_3 \leq 3c_{a2}$, anchor N_{sb} sball be multiplied by a reduction factor — ACI 318-19 17.6.4.1.1

Single anchor side blowout capacity	N_{sb} = from above calculation	= **66.85** [kips]	
Anchor edge distance in Y direction	$c_{a2} = \min(c_2, c_4)$	= 5.100 [in]	

Check If $c_1 \leq 3c_{a2}$

Anchor edge distance	c_1 = from user input	= 5.100 [in]	
Edge anchor on c_1 edge	= check if $c_1 \leq 3\ c_{a2}$	= True	ACI 318-19 17.6.4.1.1
Edge anchor side blowout capacity	$N_{sb1} = N_{sb}(1 + c_1 / c_{a2}) / 4$	= 33.42 [kips]	
	where $1.0 \leq c_1 / c_{a2} \leq 3.0$		ACI 318-19 17.6.4.1.1

Check If $c_3 \leq 3c_{a2}$

Anchor edge distance = from user input = 16.100 [in]

Anchor edge distance	c_3 = from user input	= 16.100	[in]	
Edge anchor on c_3 edge	= check if $c_3 \leq 3\ c_{a2}$	= False		ACI 318-19 17.6.4.1.1
Edge anchor side blowout capacity	$N_{sb3} = N_{sb}$	= 66.85	[kips]	

The anchor tensile force is caused by moment, anchors along the outermost bolt line has the max tensile load T_1
Side blowout along depth edge is checked against single corner anchor only which mobilizes max tensile load T_1, so number of anchor along potential side blowout edge below is set as n = 1

Total number of anchors along potential side blowout edge	n = from user input	= 1		
Single anchor side blowout capacity along side blowout edge	$N_{sb} = \min(N_{sb1}, N_{sb3})$	= **33.42**	[kips]	

Refer to Anchor Forces Calculation section above for the detail calculation on how to ge the max single anchor tensile force as shwon below

Tensile force - anchor along potential blowout edge	$T_d = T_1$ from Anchor Forces Calculation	= **6.91**	[kips]	
Strength reduction factor	$\phi_{tc} = 0.75$ supplementary reinft present			ACI 318-19 17.5.3(b)
	$\phi_{tc} N_{sbg} = 0.75 \times 33.42$	= 25.07	[kips]	
Seismic design strength reduction	= x 0.75 applicable	= **18.80**	[kips]	ACI 318-19 17.10.5.4(d)
	ratio = **0.37**	> T_d	OK	

When there are tensile anchors in the group which are not located on blowout edge, we need to use edge anchors capacity above to work out anchor group tensile capacity

Group anchor no & no of anchor along blowout edge	$n_t = 2$	$n_{bd} = 1$		
Group anchor tensile side blowout capacity	$= 18.80 \dfrac{n_t}{n_{bd}}$	= **37.60**	[kips]	

Corner Single Anchor Side Blowout

Check on corner single anchor side blowout capacity considering the corner effect factor as per ACI 318-19 17.6.4.1.1 — ACI 318-19 17.6.4.1.1

Anchor edge distance	$c_{a1} = \min(c_1, c_3)$	= 5.100	[in]	
	$c_{a2} = \min(c_2, c_4)$	= 5.100	[in]	
Consider corner effect or not	= check if $c_{a2} < 3\ c_{a1}$	= True		ACI 318-19 17.6.4.1.1
Single anchor side blowout capacity	$N_{sb1} = (1 + \dfrac{c_{a2}}{c_{a1}})/4 \times N_{sb}$	= 33.42	[kips]	

Refer to Anchor Forces Calculation section above for the detail calculation on how to ge the max single anchor tensile force as shwon below

Max single anchor tensile force	T_1 = from user load input	= **6.91**	[kips]	
Strength reduction factor	$\phi_{tc} = 0.75$ supplementary reinft present			ACI 318-19 17.5.3(b)
	$\phi_{tc} N_{sb} = 0.75 \times 33.42$	= 25.07	[kips]	
Seismic design strength reduction	= x 0.75 applicable	= **18.80**	[kips]	ACI 318-19 17.10.5.4(d)
	ratio = **0.37**	> T_1	OK	

Anchor Group Governing Tensile Resistance

Anchor group governing tensile resistance is the minimum value of the resistance values in the following limit states

No of anchors in anchor group resisting tension	n_t = from Anchor Forces Calculation above	= 2	
Anchor rod tensile resistance	$n_t \phi N_{sa} = 2 \times 42.15$	= 84.30	[kips]
Anchor concrete breakout resistance	ϕN_n = from anchor reinft tensile breakout resistance calc above	= 111.60	[kips]
Anchor pullout resistance	$n_t \phi N_{pm} = 2 \times 28.19$	= 56.37	[kips]
Anchor side blowout resistance	ϕN_{sbg} = from anchor side blowout calc above	= 37.60	[kips]
Anchor group governing tensile resistance	ϕN_n = minimum of above values	**= 37.60**	[kips]

Anchor Rod Shear Resistance ratio = 1.9 / 35.1 = **0.05** **PASS**

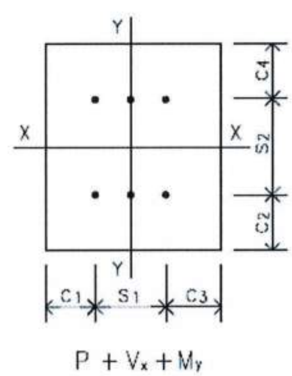
Shear load on anchor group	V_u = from user load input	= **1.90**	[kips]	
Anchor rod effective section area	$A_{se} = 0.97$ [in^2]	$f_{uta} = 58.0$	[ksi]	
No of anchors in the group resisting shear	n_s = from user input	= 2		
Anchor rod steel strength in shear	$V_{sa} = n_s 0.6 A_{se} f_{uta}$	= 67.44	[kips]	ACI 318-19 17.7.1.2b
Strength reduction factor	$\phi_{vs} = 0.65$			ACI 318-19 17.5.3(a)
	$\phi_{vs} V_{sa} =$	= 43.84	[kips]	
Reduction due to built-up grout pad	= x 0.80 applicable	= **35.07**	[kips]	ACI 318-19 17.7.1.2.1
	ratio = **0.05**	> V_u	OK	

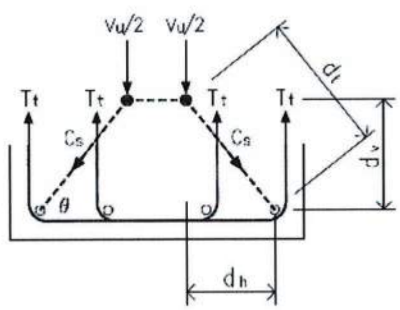
Concrete Shear Breakout Resistance ratio = 1.3 / 16.8 = **0.08** **PASS**

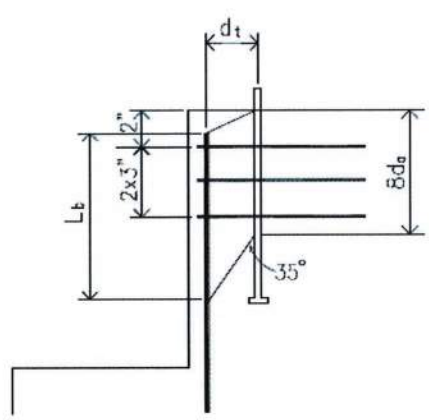
Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinforcement

ACI 318-19 Table 21.2.1 (g)

STM strength reduction factor	$\phi_{st} = 0.75$					
Anchor edge distance	$c_1 = 5.100$	[in]	$c_2 = 5.100$	[in]		
	$c_3 = 5.100$	[in]	$c_4 = 5.100$	[in]		
Anchor dia & embedment depth	$d_a = 1.250$	[in]	$h_{ef} = 36.000$	[in]		
Shear load on anchor group	V_u = from user load input		= **1.90**	[kips]		







Refer to sketch above for the terms and notations used below

Strut-and-Tie model geometry	$d_v = c_1 - 2.75"$ (70mm)	= 2.350	[in]	
	$d_h = \min(c_2 , c_4) - 2.75"$ (70mm)	= 2.350	[in]	
	$\theta = \tan^{-1} (d_v / d_h)$	= 45.0	[°]	
	$d_t = \sqrt{d_v^2 + d_h^2}$	= 3.323	[in]	
Strut compression force	$C_s = \dfrac{0.5\ V_u}{\sin\theta}$	= **1.34**	[kips]	

Strut Bearing Strength

No of anchors in the group resisting shear	n_s = from user input	= 2		
Concrete & rebar strength	$f_c = 3.0$ [ksi]	$f_y = 60.0$	[ksi]	
Strut compressive strength	$f_{ce} = 0.85\ f_c$	= 2.6	[ksi]	ACI 318-19 23.4.3

Bearing of Anchor Bolt

Anchor bearing length	$l_e = \min(8d_a , h_{ef})$	= 10.000	[in]	ACI 318-19 17.7.2.2.1
Anchor bearing area	$A_{brg} = l_e \times d_a$	= 12.50	[in^2]	
Anchor bearing resistance	$C_r = n_s \times \phi_{st} \times f_{ce} \times A_{brg}$	= 47.81	[kips]	
	ratio = **0.04**	$> V_u$	OK	

Bearing of Ver Reinforcement Rebar

Anchor & ver rebar dia	$d_a = 1.250$ [in]	$d_b = 0.626$	[in]	
Anchor bearing area	$A_{brg} = [\ l_e + 1.5d_t - 0.5(d_a + d_b)\ ] \times d_b$	= 8.79	[in^2]	
Anchor bearing resistance	$C_r = \phi_{st} \times f_{ce} \times A_{brg}$	= 16.82	[kips]	
	ratio = **0.08**	$> C_s$	OK	

Tie Reinforcement

* For tie reinforcement, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective

* Assume 100% of hor. tie bars can develop full yield strength as per user's choice in Anchor Reinforcement input

Total number of hor tie bar	$n = n_{leg}$ (leg) x n_{lay} (layer)	= 4		
Rebar resistance factor	$\phi_s = 0.75$			17.5.3 (a)
Hor rebar area & strength	$A_s = 0.20$ [in^2]	$f_{yh} = 60.0$	[ksi]	
Single tie bar tension resistance	$T_r = \phi_s f_{yh} A_s$	= 9.00	[kips]	
Total tie bar tension resistance	$\phi_s V_{nb} = n \times T_r$	= **35.99**	[kips]	17.5.2.1 (b)
	ratio = **0.05**	$> V_u$	OK	

Concrete Pryout Shear Resistance **PASS**

The pryout failure is only critical for short and stiff anchors. It is reasonable to assume that for general cast-in place headed anchors with $h_{ef} \geq 12d_a$, the pryout failure will not govern

Anchor dia & embedment depth	$d_a = 1.250$ [in]	$h_{ef} = 36.000$	[in]	
	$12d_a = 12 \times 1.250$	= **15.000**	[in]	
Anchor embedment depth	h_{ef} = from user input	= **36.000**	[in]	
	ratio = **0.42**	$> 12d_a$	OK	

Anchor Group Governing Shear Resistance

Anchor group governing shear resistance is the minimum value of the resistance values in the following limit states

Anchor rod shear resistance	ϕV_{sa} = from anchor rod shear calc above	= 35.07	[kips]
Anchor reinft shear breakout resistance	ϕV_{nb} = from anchor reinft shear breakout calc	= 35.99	[kips]
Anchor conc shear pryout resistance	ϕV_{cpg} = not govern when $h_{ef} \geq 12d_a$	= N/A	
Anchor group governing shear resistance	ϕV_n = minimum of above values	= **35.07**	[kips]

Anchor Tension and Shear Interaction

ratio = 0.00 / 1.20 = **0.00** **PASS**

Anchor group tensile load	N_u = from Anchor Forces Calculation above	= 13.83	[kips]
Anchor group shear load	V_u = from user load input	= 1.90	[kips]
Anchor group governing tensile resistance	ϕN_n = from calc in above section	= 37.60	[kips]
Anchor group governing shear resistance	ϕV_n = from calc in above section	= 35.07	[kips]

Consider anchor tension-shear interaction check if $\frac{N_u}{\phi N_n} > 0.2$ and $\frac{V_u}{\phi V_n} > 0.2$ = No ACI 318-19 17.8.3

anchor tension-shear interaction can be neglected

$$= \frac{N_u}{\phi N_n} + \frac{V_u}{\phi V_n}$$ = **0.00** ACI 318-19 17.8.3

ratio = **0.00** < 1.2 OK ACI 318-19 17.8.3

Anchor Seismic Design

Tension- Option D Shear- Option C **PASS**

Seismic - Tension Applicable ACI 318-19 17.10.5.1

Option D is selected ACI 318-19 17.10.5.3 (d)

User has to ensure that the tensile load N_u user input above includes the seismic load E, with E increased by multiplying overstrength factor Ω_o ACI 318-19 17.10.5.3 (d)

Seismic SDC >= C and E > 0.2U , Option D is selected to satisfy additional seismic requirements as per ACI 318-19 17.10.5.3 ACI 318-19 17.10.5.3

Seismic - Shear Applicable ACI 318-19 17.10.6.1

Option C is selected 17.10.6.3 (c)

User has to ensure that the shear load V_u user input above includes the seismic load E, with E increased by multiplying overstrength factor Ω_o 17.10.6.3 (c)

Seismic SDC >= C and E > 0.2U , Option C is selected to satisfy additional seismic requirements as per ACI 318-19 17.10.6.3 ACI 318-19 17.10.6.3

Base Plate - Load Case 1 **$P + M_x$** P_c =9.6 kip M_x =0.0 kip-ft Code=ACI 318-19

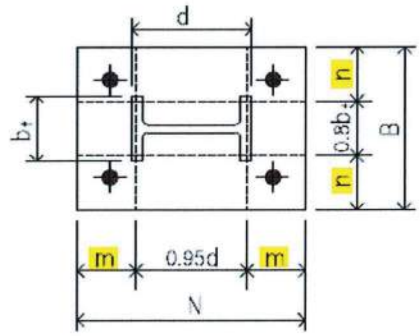
Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.86**	**PASS**

Minimum Base Plate Thickness for Rigidity ratio = 1.288 / 1.500 = **0.86** **PASS**

Please note this check is NOT a code required check. It's a check to meet the design assumption only

To ensure that base plate is rigid and anchor tensile forces are elastic linearly distributed, the base plate thickness ideally to be thicker than the 1/4 of overhangs beyond yield line in both directions as indicated on the right sketch.

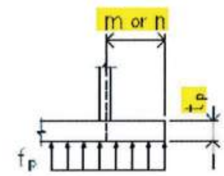
User can turn this check On/Off in Anchor Bolt - Config & Setting by checking or unchecking the option of Min base plate thickness $t_p \geq$ max of base plate overhangs m/4 and n/4



Column sect HSS6X6X3/8	d = 6.000	[in]	b_f = 6.000	[in]
Base plate width & depth	B = 16.000	[in]	N = 16.000	[in]

AISC Design Guide 1 - 3.1.2 on Page 15

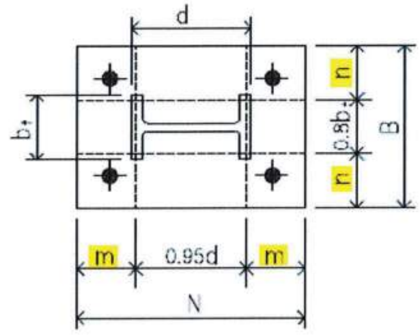
Base plate cantilever dimension	m = (N - 0.95 d) / 2	= 5.150	[in]
	n = (B - 0.95 b_f) / 2	= 5.150	[in]
Base plate thickness	t_p = from user input	= **1.500**	[in]

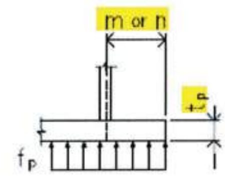


Suggested minimum base plate thickness for rigidity	t_{min} = max (m/4 , n/4)	= **1.288**	[in]
	ratio = **0.86**	< t_p	OK

Base Plate Thickness Check ratio = 0.210 / 1.500 = **0.14** **PASS**

Column sect HSS6X6X3/8	d = 6.000	[in]	b_f = 6.000	[in]
Base plate width & depth	B = 16.000	[in]	N = 16.000	[in]
Pedestal width & depth	b_c = 21.200	[in]	d_c = 21.200	[in]
Base plate area	$A_1 = B \times N$		= 256.00	[in^2]
Pedestal area	$A_2 = b_c \times d_c$		= 449.44	[in^2]
			ACI 318-19 Table 14.5.6.1	
	$k = \min(\sqrt{A_2 / A_1}, 2)$		= 1.325	
			AISC Design Guide 1 - 3.1.2 on Page 15	
Base plate cantilever dimension	$m = (N - 0.95\,d) / 2$		= 5.150	[in]
	$n = (B - 0.95\,b_f) / 2$		= 5.150	[in]
Base plate thickness	t_p = from user input		= **1.500**	[in]





Concrete Base Bearing Strength

Factored compression force	P_u = from user input	= **9.60**	[kips]	
				ACI 318-19
	$k = \min(\sqrt{A_2 / A_1}, 2)$	= 1.325		Table 14.5.6.1
Concrete strength & strength reduction factor	f_c = 3.0 [ksi]	ϕ_c = 0.65		Table 21.2.1 (d)
Pedestal bearing strength	$\phi_c P_n = \phi_c k\, 0.85\, f_c A_1$	= **562.20**	[kips]	Table 14.5.6.1
	ratio = **0.02**	> P_u	OK	

Base Plate Required Thickness

Factored forces on base plate	P_u = 9.60 [kips]	M_u = 0.00	[kip-ft]	
Column sect HSS6X6X3/8	d = 6.000 [in]	b_f = 6.000	[in]	ACI 318-19
Concrete strength & strength reduction factor	f_c = 3.0 [ksi]	ϕ_c = 0.65		Table 21.2.1 (d)
				AISC Design Guide 1
	$X = \frac{4\, d\, b_f}{(d + b_f)^2} \frac{P_u}{\phi_c P_n}$	= 0.017		3.1.2 on Page 16
	$\lambda = \min\left(\frac{2\sqrt{X}}{1 + \sqrt{1 - X}}, 1\right)$	= 0.131		3.1.2 on Page 16
	$\lambda n' = \lambda \frac{\sqrt{d\, b_f}}{4}$	= 0.197	[in]	3.1.3 on Page 17
	$L = \max(m, n)$	= 5.150	[in]	
Base plate strength & strength reduction factor	F_y = 50.0 [ksi]	ϕ_b = 0.90		
Base plate thickness	t_p = from user input	= **1.500**	[in]	
Base plate min. thickness required	$t_{min} = L\left[\frac{2\, P_u}{\phi_b F_y B N}\right]^{0.5}$	= **0.210**	[in]	3.1.2 on Page 16
	ratio = **0.14**	< t_p	OK	

Base Plate - Load Case 1 **P + M_y** P_c=9.6 kip M_y=21.6 kip-ft Code=ACI 318-19

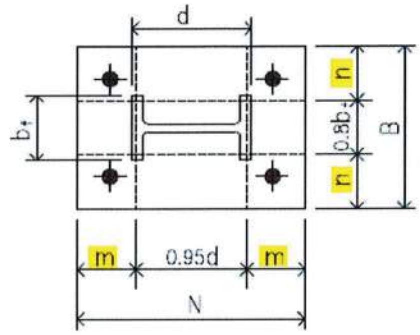
Result Summary geometries & weld limitations = **PASS** limit states max ratio = **0.86** **PASS**

Minimum Base Plate Thickness for Rigidity ratio = 1.288 / 1.500 = **0.86** **PASS**

Please note this check is NOT a code required check. It's a check to meet the design assumption only

To ensure that base plate is rigid and anchor tensile forces are elastic linearly distributed, the base plate thickness ideally to be thicker than the 1/4 of overhangs beyond yield line in both directions as indicated on the right sketch.

User can turn this check On/Off in Anchor Bolt - Config & Setting by checking or unchecking the option of Min base plate thickness $t_p \geq$ max of base plate overhangs m/4 and n/4



Column sect HSS6X6X3/8 d = 6.000 [in] b_f = 6.000 [in]

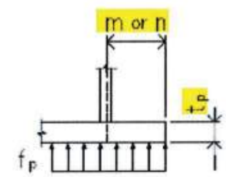
Base plate width & depth B = 16.000 [in] N = 16.000 [in]

AISC Design Guide 1 - 3.1.2 on Page 15

Base plate cantilever dimension $m = (N - 0.95\,d) / 2$ = 5.150 [in]

$n = (B - 0.95\,b_f) / 2$ = 5.150 [in]

Base plate thickness t_p = from user input = **1.500** [in]



Suggested minimum base plate thickness for rigidity $t_{min} = \max(m/4, n/4)$ = **1.288** [in]

ratio = **0.86** < t_p OK

Base Plate Thickness Check ratio = 0.793 / 1.500 = **0.53** **PASS**

Max Allowed Concrete Pressure

Column sect HSS6X6X3/8 d = 6.000 [in] b_f = 6.000 [in]

Base plate width & depth B = 16.000 [in] N = 16.000 [in]

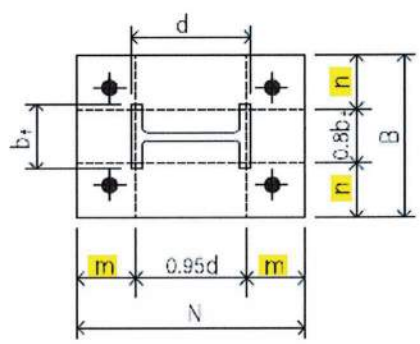
Pedestal width & depth b_c = 21.200 [in] d_c = 21.200 [in]

Base plate area $A_1 = B \times N$ = 256.00 [in^2]

Pedestal area $A_2 = b_c \times d_c$ = 449.44 [in^2]

ACI 318-19 Table 14.5.6.1

$k = \min(\sqrt{A_2 / A_1}, 2)$ = 1.325

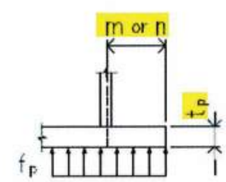


AISC Design Guide 1 - 3.1.2 on Page 15

Base plate cantilever dimension $m = (N - 0.95\,d) / 2$ = **5.150** [in]

$n = (B - 0.95\,b_f) / 2$ = **5.150** [in]

Base plate thickness t_p = from user input = **1.500** [in]



ACI 318-19

Concrete strength & strength reduction factor f_c = 3.0 [ksi] ϕ_c = 0.65 Table 21.2.1 (d)

AISC Design Guide 1

Pedestal max bearing stress $f_{p(max)} = \phi_c\, k\, 0.85\, f_c$ = 2.196 [ksi] 3.1.1 on Page 14

$q_{max} = f_{p(max)} \times B$ = 35.14 [kip/in] Eq 3.3.4 on Page 23

Factored forces on base plate P_u = 9.60 [kips] M_u = 21.60 [kip-ft]

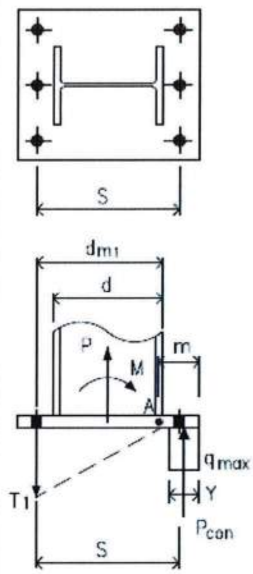
Eccentricity $e = M_u / P_u$ = 27.000 [in]

Critical eccentricity $e_{crit} = N/2 - P_u / (2\, q_{max})$ = 7.863 [in] Eq 3.3.7 on Page 24 when $e > e_{crit}$, **large moment case applied** Step 3 on Page 27

To achieve forces equilibrium in base plate to point A as shown in sketch on the bottom right, the actual concrete bearing stress f_p can be any value $f_p \le f_{p(max)}$ = 2.196 ksi

Pedestal actual bearing stress used	f_p =	= **2.196**	[ksi]
	$q_{max} = f_p \times B$	= **35.14**	[kip/in]

Anchor Tensile Force Calc - Group Anchor Subject to Moment



2 BOLT LINE

Design Basis and Assumptions

1. Assume base plate is rigid and anchor tensile forces are elastic linearly distributed as shown on the right.
2. The concrete bearing stress is assumed to be uniformly distributed as per AISC Design Guide 1 section 3.3.1

User can select the option of base plate thickness $t_p \ge$ (max of base plate overhangs m or n) / 4 in Anchor Bolt - Config & Setting to ensure that base plate has adequate rigidity to match above assumptions.

Anchor Bolt Spacings

Anchor bolt pattern	= 4B		
Anchor bolt out-out spacing & column depth	s = 11.000 [in]	d = 6.000	[in]

Loads on Anchor Group

Anchor group load	P_u = 9.60 [kips] (C)	M_u = 21.60	[kip-ft]

Along Anchor Bolt Line - Single Anchor Tensile T_i & No of Anchor Bolt n_i

Anchor bolt line - moment arm	d_{m1} = 8.500 [in]		
Bolt line 1 - single anchor T_1	T_1 = **6.91** [kips]	n_1 = **2**	
Sum of anchors tensile force	$T_u = n_1 T_1$	= **13.83**	[kips]
No of anchors in anchor group resisting tension	$n_t = n_1$	= 2	
Resistance moment by tensile anchors	$M_{ra} = n_1 T_1 d_{m1}$	= **9.79**	[kip-ft]

Moment by Concrete Pressure Reaction

Take the moment of concrtete pressure resultant P_{con} to column flange/base plate intersect point A as shown on above sketch on the right

Max allowed concrete pressure	q_{max} = from above calculation	= 35.14	[kip/in]
Base plate cantilever length	m = from above calculation	= 5.150	[in]
Concrete pressure block length	Y =	= **0.667**	[in]
Concrete pressure stress resultant	$P_{con} = q_{max} Y$	= 23.44	[kips]
Resistance moment by concrete reaction	$M_{rc} = P_{con} \times (m - 0.5Y)$	= **9.41**	[kip-ft]

Below two sections are for verification purpose only. We want to verify that the anchor tensile forces and concrete pressure block length Y shown above make the base plate achieving force equilibrium

Verify Vertical Force Equilibrium

Tensile anchors reaction on base plate - downward	$P_{ar} = n_1 T_1$	= 13.83	[kips]
Base plate compressive load-downward	P_u = from user load input	= 9.60	[kips]
Sum of downward forces on base plate	$P_{dn} = P_{ar} + P_u$	= **23.43**	[kips]
Concrete pressure reaction on base plate - upward	$P_{con} = q_{max} Y$	= 23.44	[kips]
Sum of upward forces on base plate	$P_{up} = P_{con}$	= **23.44**	[kips]

Conclusion : the vertical forces equilibrium is achieved

Conclusion : the vertical forces equilibrium is achieved

Summation of Moments Taken About Point A

Resistance moment by tensile anchors downward reaction forces	$M_{ra} = n_1 T_1 d_{m1}$		= 9.79	[kip-ft]
Resistance moment by concrete pressure reaction force	$M_{rc} = P_{con} \times (m - 0.5Y)$		= 9.41	[kip-ft]
Sum of resistance moment	$= M_{ra} + M_{rc}$		= **19.20**	[kip-ft]
Load on base plate	$P_u = 9.60$	[kips]	$M_u = 21.60$	[kip-ft]
Column sect HSS6X6X3/8	$d = 6.000$	[in]		
Sum of moments from base plate loads taken to point A	$= M_u - P_u \times 0.5\ d$		= **19.20**	[kip-ft]

Conclusion : the summation of moments taken about point A equals to zero

Anchor Rod Steel Tensile Capacity

Bolt line 1 - single anchor T_1	$T_1 = 6.91$	[kips]	$n_1 = 2$		
Sum of all anchors tensile force along bolt line 1	$T_u = n_1 \times T_1$		= **13.83**	[kips]	
Anchor rod effective section area	$A_{se} = 0.97$	[in^2]	$f_{uta} = 58.0$	[ksi]	
Strength reduction factor	$\phi_{ts} = 0.75$				ACI 318-19 17.5.3(a)
Anchor rod tensile resistance	$T_r = \phi_{ts} n_t A_{se} f_{uta}$		= **84.30**	[kips]	ACI 318-19 17.6.1.2
	ratio = **0.16**		$> T_u$	OK	

Base Plate Flexure Caused by Anchor Rod Tension — **AISC Design Guide 1**

Column sect HSS6X6X3/8	$d = 6.000$	[in]	$t_f = 0.349$	[in]	
Anchor out-out spacing	$s_1 = 11.000$	[in]			
Anchor to column center distance	$f = s_1 / 2$		= 5.500	[in]	
Sum of all anchors tensile force along bolt line 1	$T_u = n_1 \times T_1$		= 13.83	[kips]	
Anchor spacing	$s_1 = 11.000$	[in]	$s_2 = 11.000$	[in]	
Column sect HSS6X6X3/8	$d = 6.000$	[in]	$t = 0.349$	[in]	
T_u to flange moment lever arm	$x = 0.5\ (s_1 - 0.95d)$		= 2.650	[in]	Eq 3.4.6
Base plate width & strength	$B = 16.000$	[in]	$F_y = 50.0$	[ksi]	
	$t_{req\text{-}t} = 2.11 \left(\frac{T_u x}{B F_y} \right)^{0.5}$		= 0.452	[in]	Eq 3.4.7a

Base Plate Flexure Caused by Conc Bearing Pressure

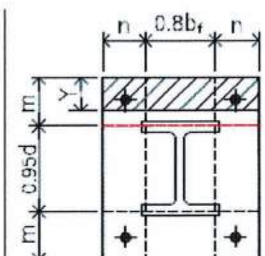
Plate Width Strip Bending

Plate flexure moment per unit width caused by conc bearing reaction - width strip as shown on the right sketch

Conc stress bearing length & base plate overhang	$Y = 0.667$	[in]	$m = 5.150$	[in]
Base plate strength & strength reduction factor	$F_y = 50.0$	[ksi]	$\phi_b = 0.90$	
Pedestal bearing stress used	$f_p =$		= 2.196	[ksi]
			AISC DG1 Eq 3.3.15a-2	

When Y < m plate moment per unit width $0.9\ F_y \frac{t^2}{4} = f_p Y\ (m - 0.5Y)$

$t_{req\text{-}b1} = 2.11 \left(\frac{f_p Y\ (m-0.5Y)}{F_y} \right)^{0.5}$ = 0.793 [in]

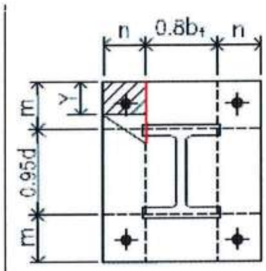


WIDTH STRIP BENDING

Plate Side Strip Bending

Plate flexure moment per unit width caused by conc bearing reaction - side strip as shown on the right sketch

Refer to AISC Engineering Journal 2014 Q4 Volume 51 No 4 Page 234, for base plate side strip bending, use enlarged effective width b_{eff} as shown below



SIDE STRIP BENDING

Conc stress length & plate overhang	Y = 0.667 [in]	n = 5.150	[in]
When Y < 2n	$b_{eff} = Y/2 + n$	= 5.483	[in]
Moment adjustment factor due to enlarged effective plate width	$\beta = \frac{Y}{b_{eff}}$	= 0.122	
Pedestal bearing stress used	$f_p =$	= 2.196	[ksi]

AISC DG1 Eq 3.3.15a-2

Plate moment per unit width $0.9\, F_y \frac{t^2}{4} = \frac{1}{2} f_p n^2 \beta$

β on above right is the factor to reduce plate moment due to enlarged plate effective width

$t_{req\text{-}b2} = 2.11\, \left(\frac{0.5\, f_p n^2 \beta}{F_y} \right)^{0.5}$ = 0.562 [in]

Base plate thickness	t_p = from user input	= **1.500**	[in]
Min required plate thickness	$t_{min} = \max(t_{req\text{-}t}, t_{req\text{-}b1}, t_{req\text{-}b2})$	= **0.793**	[in]
	ratio = **0.53**	$< t_p$	OK

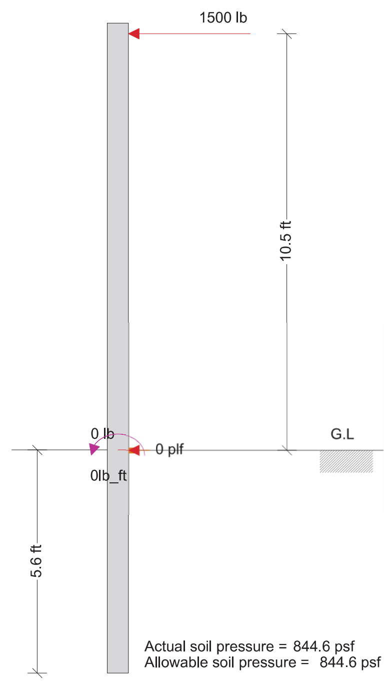
Tekla Tedds	Project				Job Ref.	
Point 2 Structural Engineers Inc 3701 Business Dr Suite 100 Sacramento, CA 95820	Section				Sheet no./rev. 1	
	Calc. by B	Date 8/3/2022	Chk'd by	Date	App'd by	Date

FLAGPOLE EMBEDMENT (IBC)

FOOTING DESIGN

In accordance with IBC 2018

Tedds calculation version 1.2.04



Soil capacity data

Allowable passive pressure L_{sbc} = **150** pcf

Maximum allowable passive pressure P_{max} = **2000** psf

Load factor 1 (1806.1) LDF_1 = **1.00**

Load factor 2 (1806.3.4) LDF_2 = **1.0**

Pole geometry

Shape of the pole **Round**

Diameter of the pole Dia = **30** in

Laterally restrained **Yes**

Load data

First point load P_1 = **1500** lbs

Distance of P_1 from ground surface H_1 = **10.5** ft

Second point load P_2 = **0** lbs

Distance of P_2 from ground surface H_2 = **0** ft

Uniformly distributed load W = **0** plf

Start distance of W from ground surface a = **0** ft

End distance of W from ground surface a_1 = **0** ft

Applied moment M_1 = **0** lb_ft

Distance of M_1 from ground surface H_3 = **0** ft

Shear force and bending moment

Total shear force $F = P_1 + P_2 + W \times (a_1 - a)$ = **1500** lbs

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	Section				Sheet no./rev. 2	
	Calc. by B	Date 8/3/2022	Chk'd by	Date	App'd by	Date

Total bending moment at grade $M_g = P_1 \times H_1 + P_2 \times H_2 + W \times (a_1 - a) \times (a + a_1) / 2 + M_1 =$ **15750** lb_ft

Distance of resultant lateral force $h = abs(M_g / F) =$ **10.5** ft

Embedment depth (1807.3.2.2)

Embedment depth provided $D =$ **5.63** ft

Allowable lateral passive pressure $S_3 = \min(P_{max}, L_{sbc} \times \min(D, 12\ ft)) \times LDF_1 \times LDF_2 =$ **844.6** psf

Embedment depth required $D_1 = ((4.25 \times abs(F) \times h) / (S_3 \times Dia))^{0.5} =$ **5.63** ft

Actual lateral passive pressure $S_4 = (4.25 \times abs(F) \times h) / (D^2 \times Dia) =$ **844.6** psf