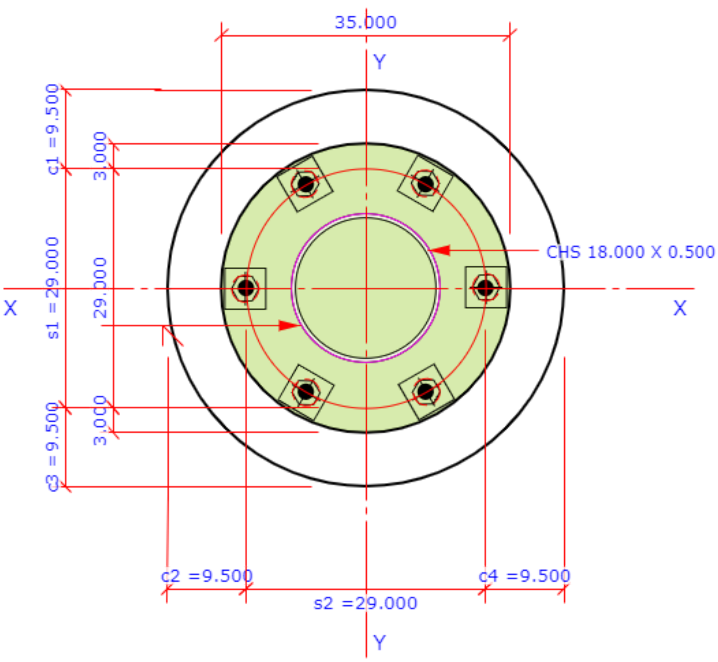
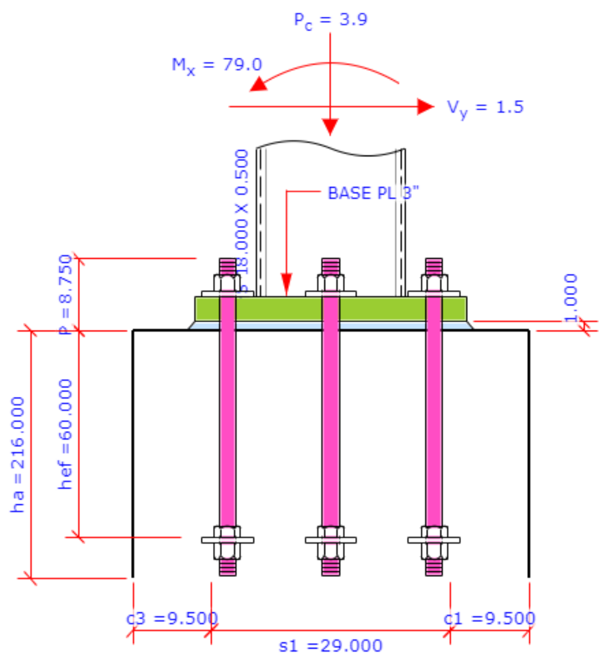
Result Summary - Overall	Anchorage Design	Code=ACI 318-19

Result Summary - Overall		geometries & weld limitations = **PASS**	limit states max ratio = **0.87**	**PASS**
Anchor Bolt - LC 1	**P + V_y + M_x**	geometries & weld limitations = **PASS**	limit states max ratio = **0.87**	**PASS**
Base Plate - LC 1	**P + M_x**	geometries & weld limitations = **PASS**	limit states max ratio = **0.86**	**PASS**

Sketch	Anchorage Design	Code=ACI 318-19

Design Load Case 1





Anchor Forces Calculation

Anchor Tensile Force Calculation

User Input

Anchor edge distance

c_{1u} = 9.500 [in] c_{2u} = 9.500 [in]

c_{3u} = 9.500 [in] c_{4u} = 9.500 [in]

Anchor out-out spacing

s_{1u} = 29.000 [in] s_{2u} = 29.000 [in]

Anchor embedment depth

h_{ef} = 60.000 [in]

Design Load - Load Case 1

Axial force

Axial P = 3.90 [kips] in compression

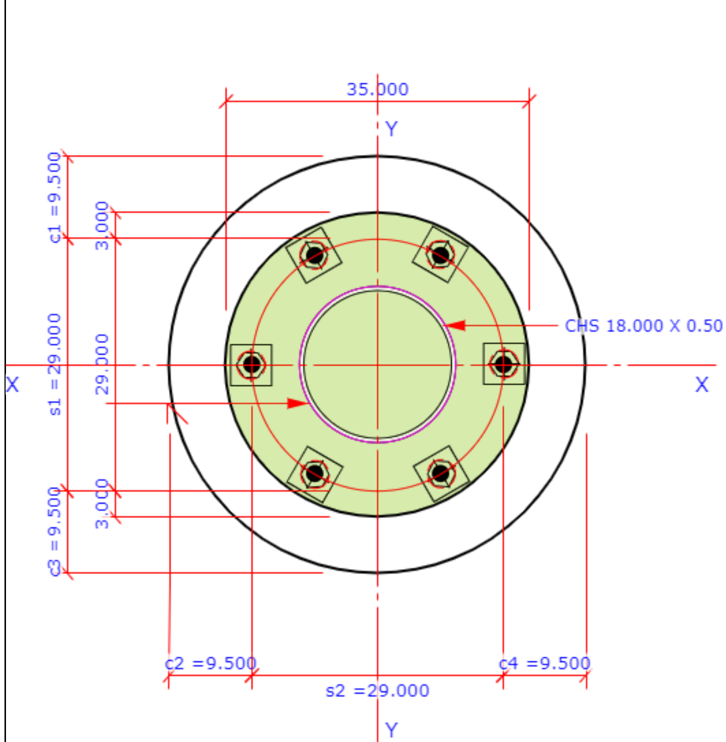
Shear forces

V_y = 1.50 [kips] V_x = 0.00 [kips]

Moment forces

M_x = 79.00 [kip-ft] M_y = 0.00 [kip-ft]

Anchor Layout Plan



Load Case 1 - Check on P + V_y + M_x

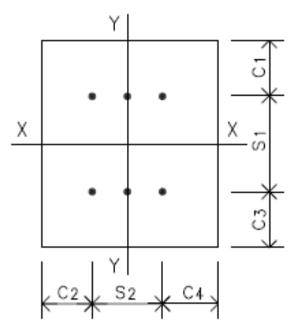
Anchor edge distance c_1 = 9.500 [in] c_2 = 9.500 [in]

c_3 = 9.500 [in] c_4 = 9.500 [in]

Anchor out-out spacing s_1 = 29.000 [in] s_2 = 29.000 [in]

Anchor group load P_u = 3.90 [kips] V_u = 1.50 [kips]

M_u = 79.00 [kip-ft]



$P + V_y + M_x$

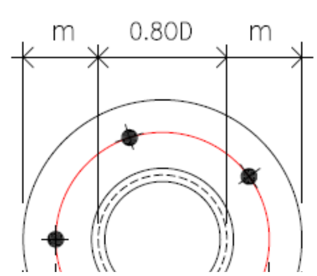
Max Allowed Concrete Pressure

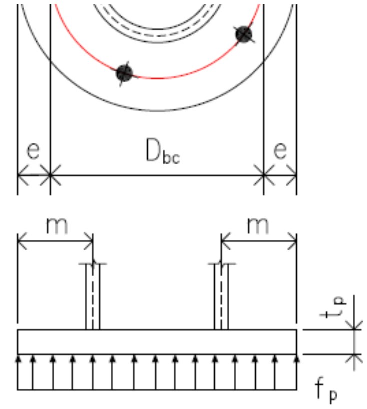
Bolt circle dia & edge distance D_{bc} = 29.000 [in] e = 3.000 [in]

Base plate area $A_1 = \frac{\pi}{4} (D_{bc} + 2\,e)^2$ = 962.11 [in^2]

Bolt circle dia & edge distance D_{bc} = 29.000 [in] c = 9.500 [in]

Base plate area $A_2 = \frac{\pi}{4} (D_{bc} + 2\,c)^2$ = 1809.6 [in^2]





ACI 318-19 Table 14.5.6.1

$k = \min\left(\sqrt{A_2 / A_1}\ ,\ 2\right)$ = 1.371

Column sect Custom Sect — d = 18.000 [in] — b_f = 18.000 [in]

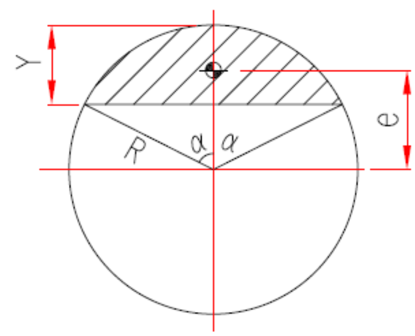
AISC Design Guide 1 - 3.1.2 on Page 15

Base plate cantilever dimension — $m = (N - 0.8\ d) / 2$ = **9.300** [in]

$n = (B - 0.8\ b_f) / 2$ = 10.300 [in]

				ACI 318-19
Concrete strength & strength reduction factor	f_c = 4.0 [ksi]	ϕ_c = 0.65		Table 21.2.1 (d)
				AISC Design Guide 1
Pedestal max bearing stress	$f_{p(max)} = \phi_c\ k\ 0.85\ f_c$		= 3.031 [ksi]	3.1.1 on Page 14
Factored forces on base plate	P_u = 3.90 [kips]	M_u = 79.00 [kip-ft]		
Eccentricity	$e = M_u / P_u$		= **243.077**[in]	

Calculate Circular Bolt Pattern Critical Eccentricity e_{crit}



Refer to sketch on the right, max allowed ecc when no tensile forces mobilized in anchors is the ecc when

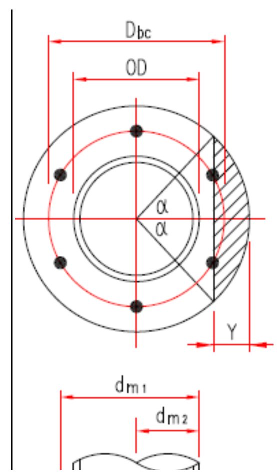
1) Max bearing stress $f_{p(max)}$ is reached so that Y reaches the min and e reaches the max

2) Axial compression P_u equals to bearing stress reaction resultant $P_u = f_{p(max)}\ A$ as there is no anchor tension involved in the vertical forces equilibrium

Axial compression force & max allowed stress under base plate	P_u = 3.90 [kips]	$f_{p(max)}$ = 3.031 [ksi]	
Base plate radius and stress block angle when Y is reached	R = 17.500 [in]	α = 10.170	
Stress block area	$A = \frac{R^2}{2}(2\alpha - \sin(2\alpha))$		= 1.13 [in^2]
Stress block length at angle of α	Y = calc from angle α above		= 0.275 [in]
Max allowed ecc when no anchor is in tensin	$e = \frac{4R\ \sin^3\alpha}{3\ (2\alpha - \sin(2\alpha))}$		= 17.335 [in]
Critical eccentricity	e_{crit} = e value calculated above		= **17.335** [in]

when $e > e_{crit}$, **large moment case applied** — Step 3 on Page 27

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 $t_p \geq$ (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 Dimensions

Circular anchor bolt circle dia & base plate dia — D_{bc} = 29.000 [in] — D_{bp} = 35.000 [in]

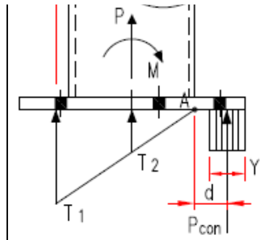
Column sect Custom Sect — OD = 18.000 [in]

Loads on Anchor Group

Anchor group load — P_u = 3.90 [kips] (C) — M_u = 79.00 [kip-ft]

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

Anchor bolt line - moment arm d_{m1} = 21.557 [in] d_{m2} = 9.000 [in]



Bolt line 1 - single anchor T_1 T_1 = **12.20** [kips] n_1 = **2**

Bolt line 2 - single anchor T_2 T_2 = 5.09 [kips] n_2 = 2

Sum of anchors tensile force $T_u = n_1 T_1 + n_2 T_2$ = **34.58** [kips]

No of anchors in anchor group resisting tension $n_t = n_1 + n_2$ = 4

Resistance moment by anchor tensile $M_{ra} = n_1 T_1 d_{m1} + n_2 T_2 d_{m2}$ = **51.46** [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

Pedestal max bearing stress $f_{p(max)} = \phi_c k\ 0.85\ f_c$ = 3.031 [ksi] AISC DG1 3.1.1

Base plate radius and column dia R = 17.500 [in] OD = 18.000 [in]

Stress block length and angle at Y Y = 1.373 [in] α = 22.850

Conc stress block area $A = \frac{R^2}{2}(\ 2\alpha - \sin(2\alpha))$ = 12.54 [in^2]

Conc stress block centroid to circular base plate center distance $e = \frac{4R\ \sin^3\alpha}{3\ (2\alpha - \sin(2\alpha))}$ = 16.678 [in]

Conc stress resultant to point A moment arm $d_c = e - 0.5\ OD$ = 7.678 [in]

Concrete pressure stress resultant $P_{con} = f_{p(max)} A$ = 38.01 [kips]

Resistance moment by concrete stress resultant reaction $M_{rc} = P_{con} \times d_c$ = **24.32** [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 $P_{ar} = n_1 T_1 + n_2 T_2$ = 34.58 [kips]

Base plate compressive load- downward P_u = from user load input = 3.90 [kips]

Sum of downward forces on base plate $P_{dn} = P_{ar} + P_u$ = **38.48** [kips]

Concrete pressure reaction on base plate - upward $P_{con} = q_{max} Y$ = 38.01 [kips]

Sum of upward forces on base plate $P_{up} = P_{con}$ = **38.01** [kips]

Conclusion : the vertical forces equilibrium is achieved

Summation of Moments Taken About Point A

Resistance moment by tensile anchors downward reaction forces $M_{ra} = n_1 T_1 d_{m1} + n_2 T_2 d_{m2}$ = 51.46 [kip-ft]

Resistance moment by concrete pressure reaction force $M_{rc} = P_{con} \times d_c$ = 24.32 [kip-ft]

Sum of resistance moment $= M_{ra} + M_{rc}$ = **75.78** [kip-ft]

Load on base plate P_u = 3.90 [kips] M_u = 79.00 [kip-ft]

Column sect Custom Sect d = 18.000 [in]

Sum of moments from base plate loads taken to point A $= M_u - P_u \times 0.5\ OD$ = **76.07** [kip-ft]

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

---

Load Case 1 - P + V_y + M_x Reduced h_{ef} Calc

Anchor Embedment Depth h_{ef} Adjustment

Anchor embedment depth h_{ef} - If anchors are located less than $1.5h_{ef}$ from three or more edges, h_{ef} needs to be shortened as per ACI 318-19 17.6.2.1.2

ACI 318-19 17.6.2.1.2

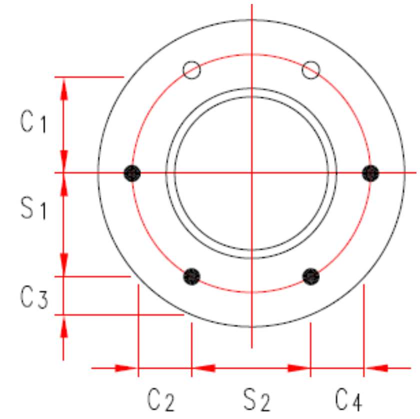
Anchor group edge distances are re-calculated base on tensile anchors in the group as not all anchors mobilized tensile force under the moment

Anchor Group Dimensions

Anchor bolt circle dia & pedestal dia	D_{bc} =	29.000	[in]	D_{pd} =	48.000	[in]
Anchor spacing	s_1 =	14.500	[in]	s_2 =	14.500	[in]
Anchor edge distance	c_1 =	19.125	[in]	c_2 =	9.500	[in]
	c_3 =	10.321	[in]	c_4 =	9.500	[in]

Max anchor spacing within the group used in effective anchor embedment depth calc

Max anchor spacing within the tensile anchors group	s_{1max} =	14.500	[in]	s_{2max} =	29.000	[in]

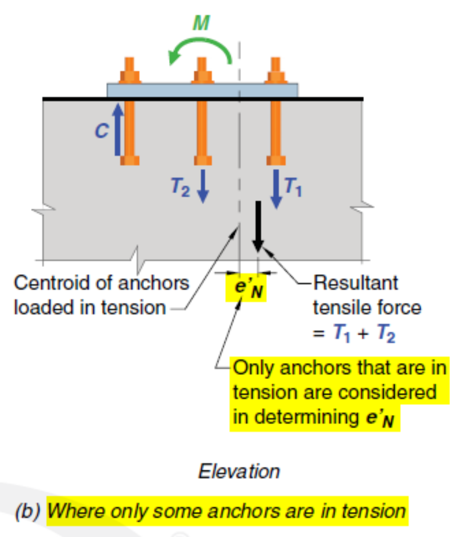


Anchor embedment depth - from user input	h_{ef} = from user input	= 60.000 [in]	
Anchors are located less than $1.5h_{ef}$ from three or more edges	= Yes		
Max of edge distances not exceeding $1.5h_{ef}$	$c_{a,max}$ =	= 19.125 [in]	
Max spacing between anchors within the group	s =	= 29.000 [in]	
Anchor embedment depth - adjusted	$h_{ef} = \max(c_{a,max}/1.5\ ,\ s/3)$	= **12.750** [in]	ACI 318-19 17.6.2.1.2

Concrete Breakout - Tensile Anchors Eccentricity Factor - $\Psi_{ec,N}$ Calc

Modification factor for anchor groups loaded eccentrically in tension as per ACI 318-19 17.6.2.3.1

ACI 318-19 Fig. R17.6.2.3.1
Definition of e_N for an anchor group



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

See calculation above for sketch showing the notations of $T_1 \sim T_2$ and s_{b1} values shown below

Bolt line 1 - single anchor T_1	T_1 = 12.20	[kips]	n_1 = 2
Bolt line 2 - single anchor T_2	T_2 = 5.09	[kips]	n_2 = 2
Anchor distance to bolt line-1	d_{e2} = 12.557	[in]	

Eccentricity e_N of Resultant Anchor Tensile Force

Take bolt line-1 as a rotating point, take moment to bolt line-1

Distance from anchors tensile resultant to bolt line-1 $$d_1 = \frac{n_2 T_2 d_{e2}}{n_1 T_1 + n_2 T_2} = 3.698 \text{ [in]}$$

Distance from anchors group centroid to bolt line-1 $$d_2 = \frac{n_2 d_{e2}}{n_1 + n_2} = 6.279 \text{ [in]}$$

Ecc dist between anchor tensile resultant and anchor group CG $$e_N = d_2 - d_1 = \mathbf{2.580} \text{ [in]}$$

Refer to calc above for details on reduced h_{ef} calc as per ACI 318-19 17.6.2.1.2

Anchor embedment depth h_{ef} = from calc above = 12.750 [in]

ACI 318-19 Eq 17.6.2.3.1

Eccentricity modification factor $$\Psi_{ec,N} = \frac{1}{(1 + e_N / 1.5h_{ef})} \le 1 = \mathbf{0.881}$$

Anchor Bolt - Load Case 1 **P + V_y + M_x** P_c =3.9 kip V_y =1.5 kip M_x =79.0 kip-ft Code=ACI 318-19

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.87**	**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 = 2.000	[in]	

Min Anchor Edge Distance

Anchor edge distance	c_1 = 19.125 [in]	c_2 = 9.500	[in]	
	c_3 = 10.321 [in]	c_4 = 9.500	[in]	
Min anchor edge distance required	c_{min} = from PIP STE05121 Table 1 below	= **8.000**	[in]	PIP STE05121 Table 1
Min anchor edge distance	c = min(c_1 , c_2 , c_3 , c_4)	= **9.500**	[in]	
		≥ c_{min}	OK	

Min Anchor Spacing

Min anchor spacing required	s_{min} = from PIP STE05121 Table 1 below	= **8.000**	[in]	PIP STE05121 Table 1
Anchor bolt pattern	= from user input	= C1		
Min anchor spacing	s = from user input	= **14.500**	[in]	
		≥ s_{min}	OK	

Min Anchor Embedment Depth

Min anchor embedment required	h_{min} = from PIP STE05121 Table 1 below	= **24.000**	[in]	PIP STE05121 Table 1
Min anchor embedment depth	h_{ef} = from user input	= **60.000**	[in]	
		≥ 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.	in^2	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.

Anchor Rod Tensile Resistance ratio = 12.2 / 108.8 = **0.11** **PASS**

Anchor rod effective section area A_{se} = 2.50 [in^2] f_{uta} = 58.0 [ksi]

Anchor rod steel strength in tension $N_{sa} = A_{se} f_{uta}$ = 145.00 [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 = **12.20** [kips]

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

$\phi_{ts} N_{sa}$ = 0.75 x 145.00 = **108.75** [kips]

ratio = **0.11** > T OK

Anchor Concrete Tensile Breakout Resistance ratio = 34.6 / 39.8 = **0.87** **PASS**

Anchor embedment depth-adjusted h_{ef} = from Anchor Forces Calculation above = 12.750 [in]

Conc strength & lightweight conc factor $f_c = 4.0$ [ksi] $\lambda = 1.0$ ACI 318-19 17.2.4.1

Single anchor concrete breakout strength $N_b = 24\lambda \sqrt{f_c}\, h_{ef}^{1.5}$ If $h_{ef} < 11"$ or $h_{ef} > 25"$ = 70.42 [kips] ACI 318-19 17.6.2.2.1

$16\lambda \sqrt{f_c}\, h_{ef}^{5/3}$ If $11" \le h_{ef} \le 25"$ ACI 318-19 17.6.2.2.3

Circular Bolt Pattern Tensile Anchor Breakout A_{NC} Calculation

Refer to Anchor Forces Calculation for details of circular pattern anchor group anchor spacings and edge distances calculation

Anchor bolt circle dia & pedestal dia $D_{bc} = 29.000$ [in] $D_{pd} = 48.000$ [in]

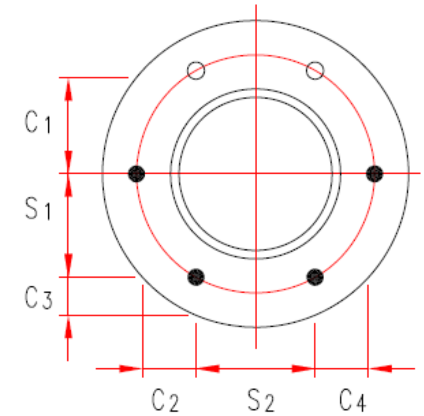
Anchor spacing $s_1 = 14.500$ [in] $s_2 = 14.500$ [in]

Anchor edge distance $c_1 = 19.125$ [in] $c_2 = 9.500$ [in]

$c_3 = 10.321$ [in] $c_4 = 9.500$ [in]

Anchor embedment depth-adjusted h_{ef} = from calc above = 12.750 [in]

Anchor group projected conc failure area A_{NC1} = = 1472.2 [in^2]



$A_{Nco} = 9\, h_{ef}^2$ = 1463.1 [in^2] ACI 318-19 17.6.2.1.4

No of anchors in the group resisting tension n_t = from Anchor Forces Calculation above = 4

$A_{Nc} = \min(A_{Nc1},\ n_t A_{Nco})$ = 1472.2 [in^2] ACI 318-19 17.6.2.1.1

Eccentricity modification factor $\Psi_{ec,N}$ = from Anchor Forces Calculation above = 0.881 ACI 318-19 17.6.2.3.1

Min edge distance c_{min} = = 9.500 [in]

Edge modification factor $\Psi_{ed,N} = \min[0.7 + \dfrac{0.3c_{min}}{1.5h_{ef}},\ 1.0]$ = 0.849 ACI 318-19 17.6.2.4.1

Conc cracking modification factor $\Psi_{c,N}$ = = 1.00 ACI 318-19 17.6.2.5.1

Conc splitting modification factor $\Psi_{cp,N}$ = = 1.00 ACI 318-19 17.6.2.6.1

Concrete breakout resistance $N_{cbg} = \dfrac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ = 53.01 [kips] ACI 318-19 17.6.2.1b

Sum of anchors tensile force in anchor group N_u = from Anchor Forces Calculation above = **34.58** [kips]

Strength reduction factor $\phi_{tc} = 0.75$ supplementary reinft present ACI 318-19 17.5.3(b)

$\phi_{tc} N_{cbg} = 0.75 \times 53.01$ = 39.75 [kips]

Seismic design strength reduction = x 1.0 not applicable = **39.75** [kips] ACI 318-19 17.10.5.4(b)

ratio = **0.87** > N_u OK

Anchor Pullout Resistance ratio = 12.2 / 119.1 = **0.10** **PASS**

Anchor head net bearing area & conc strength A_{brg} = 5.32 [in^2] f_c = 4.0 [ksi]

Single bolt pullout resistance $N_p = 8\ A_{brg}\ f_c$ = 170.11 [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 = **12.20** [kips]

Strength reduction factor ϕ_{tc} = 0.70 pullout strength is always Condition B ACI 318-19 17.5.3(c)

$\phi_{tc} N_{pn} = \phi_{tc} \Psi_{cP} N_p$ = **119.08** [kips]

Seismic design strength reduction = x 1.0 not applicable = **119.08** [kips] ACI 318-19 17.10.5.4(c)

ratio = **0.10** > T OK

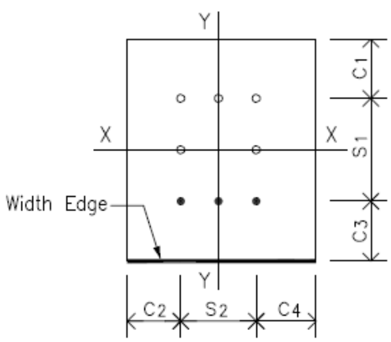
Anchor Side Blowout Resistance ratio = 12.2 / 86.7 = **0.14** **PASS**

Anchor Inputs

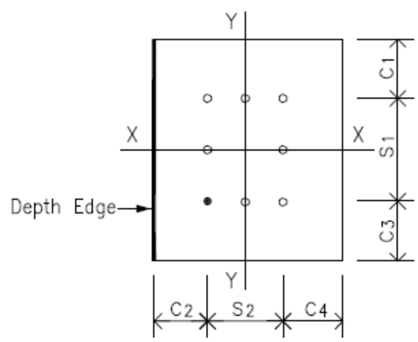
Anchor edge distance c_1 = 19.125 [in] c_2 = 9.500 [in]

c_3 = 10.321 [in] c_4 = 9.500 [in]

Anchor out-out spacing s_1 = 14.500 [in] s_2 = 14.500 [in]



Side Blowout Width Edge



Side Blowout Depth Edge

Side Edges Along X-X Axis - Width Edges

Anchor edge distance in Y direction $c_{a1} = \min(c_1, c_3)$ = 10.321 [in]

Anchor embedment depth h_{ef} = from user input = 60.000 [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 X direction s_2 = from user input = 14.500 [in]

Anchor number along X direction n_w = from user input = 2

Anchor head net bearing area & conc strength A_{brg} = 5.32 [in^2] f_c = 4.0 [ksi]

Lightweight conc modification factor λ = 1.0 ACI 318-19 17.2.4.1

Single anchor side blowout capacity $N_{sb} = 160\ c_{a1} \sqrt{A_{brg}}\ \lambda \sqrt{f_c}$ = 240.80 [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 X-X edges $s_b = s_2 / (n_w - 1)$ = 14.500 [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	$= 1 + \dfrac{s_2}{6c_{a1}}$	= 1.23	ACI 318-19 17.6.4.2
Group anchor side blowout capacity	$N_{sbg} = (1 + \dfrac{s_2}{6c_{a1}})\ N_{sb}$	= **297.19** [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	$T_1 = 12.20$ [kips]	$n_1 = 2$	
Tensile force - anchors along potential blowout edge	$T_w = n_1 T_1$	= **24.39** [kips]	
Strength reduction factor	$\phi_{tc} = 0.75$ supplementary reinft present		ACI 318-19 17.5.3(b)
	$\phi_{tc} N_{sbg} = 0.75 \times 297.19$	= 222.89 [kips]	
Seismic design strength reduction	= x 1.0 not applicable	= **222.89** [kips]	ACI 318-19 17.10.5.4(d)
	ratio = **0.11**	> 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	$n_t = 4$	$n_{bw} = 2$	
Group anchor tensile side blowout capacity	$= 222.89\ \dfrac{n_t}{n_{bw}}$	= **445.78** [kips]	

Side Edges Along Y-Y Axis - Depth Edges

Anchor edge distance in X direction	$c_{a2} = \min(c_2, c_4)$	= 9.500 [in]	
Anchor embedment depth	h_{ef} = from user input	= 60.000 [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	$A_{brg} = 5.32$ [in^2]	$f_c = 4.0$ [ksi]	
Lightweight conc modification factor	$\lambda = 1.0$		ACI 318-19 17.2.4.1
Single anchor side blowout capacity	$N_{sb} = 160\ c_{a2} \sqrt{A_{brg}}\ \lambda \sqrt{f_c}$	= 221.65 [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	$c_1 = 10.321$ [in]	$c_3 = 33.625$ [in]	

When amchor edge distance c_1, c_3 are small, when c_1 or $c_3 \le 3c_{a2}$, 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	= **221.65** [kips]	
Anchor edge distance in X direction	$c_{a2} = \min(c_2, c_4)$	= 9.500 [in]	

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

Anchor edge distance	c_1 = from user input	= 10.321 [in]	
Edge anchor on c_1 edge	= check if $c_1 \le 3\ c_{a2}$	= True	ACI 318-19 17.6.4.1.1
Edge anchor side blowout capacity	$N_{sb1} = N_{sb}(1 + c_1 / c_{a2}) / 4$	= 115.61 [kips]	
	where $1.0 \le c_1 / c_{a2} \le 3.0$		ACI 318-19 17.6.4.1.1

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

Anchor edge distance	c_3 = from user input	= 33.625 [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	$N_{sb3} = N_{sb}$	= 221.65 [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	$N_{sb} = \min(N_{sb1}, N_{sb3})$	= **115.61** [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	= **12.20** [kips]	
Strength reduction factor	$\phi_{tc} = 0.75$ supplementary reinft present		ACI 318-19 17.5.3(b)
	$\phi_{tc} N_{sbg} = 0.75 \times 115.61$	= 86.71 [kips]	
Seismic design strength reduction	= x 1.0 not applicable	= **86.71** [kips]	ACI 318-19 17.10.5.4(d)
	ratio = **0.14**	> 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	$n_t = 4$	$n_{bd} = 1$	
Group anchor tensile side blowout capacity	$= 86.71 \dfrac{n_t}{n_{bd}}$	= **346.84** [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	$c_{a1} = \min(c_1, c_3)$	= 10.321 [in]	
	$c_{a2} = \min(c_2, c_4)$	= 9.500 [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	$N_{sb1} = (1 + \dfrac{c_{a2}}{c_{a1}})/4 \times N_{sb}$	= 120.40 [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	= **12.20** [kips]	
Strength reduction factor	$\phi_{tc} = 0.75$ supplementary reinft present		ACI 318-19 17.5.3(b)
	$\phi_{tc} N_{sb} = 0.75 \times 120.40$	= 90.30 [kips]	
Seismic design strength reduction	= x 1.0 not applicable	= **90.30** [kips]	ACI 318-19 17.10.5.4(d)
	ratio = **0.14**	> 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	= 4
Anchor rod tensile resistance	$n_t \phi N_{sa}$ = 4 x 108.75	= 435.00 [kips]
Anchor concrete breakout resistance	ϕN_{cbg} = from anchor conc breakout calc above	= 39.75 [kips]
Anchor pullout resistance	$n_t \phi N_{pm}$ = 4 x 119.08	= 476.31 [kips]
Anchor side blowout resistance	ϕN_{sbg} = from anchor side blowout calc above	= 346.84 [kips]
Anchor group governing tensile resistance	ϕN_n = minimum of above values	= **39.75** [kips]

Anchor Rod Shear Resistance ratio = 1.5 / 135.7 = **0.01** **PASS**

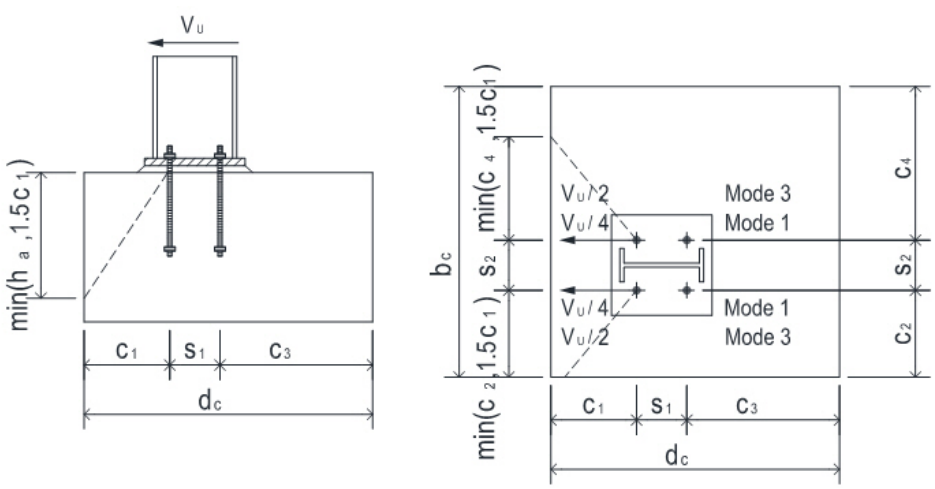
Shear load on anchor group	V_u = from user load input	= **1.50** [kips]	
Anchor rod effective section area	A_{se} = 2.50 [in^2]	f_{uta} = 58.0 [ksi]	
No of anchors in the group resisting shear	n_s = from user input	= 3	
Anchor rod steel strength in tension	$V_{sa} = n_s 0.6 A_{se} f_{uta}$	= 261.00 [kips]	ACI 318-19 17.7.1.2b
Strength reduction factor	ϕ_{vs} = 0.65		ACI 318-19 17.5.3(a)
	$\phi_{vs} V_{sa}$ =	= 169.65 [kips]	
Reduction due to built-up grout pad	= x 0.80 applicable	= **135.72** [kips]	ACI 318-19 17.7.1.2.1
	ratio = **0.01**	> V_u OK	

Concrete Shear Breakout Resistance - Perpendicular To Edge ratio = 1.5 / 22.0 = **0.07** **PASS**

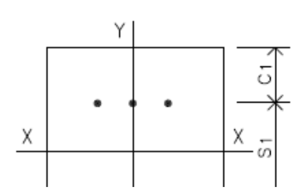
For front anchors shear breakout, the shear force checked against with can be 0.5 x V_u or 1.0 x V_u, depending on whether base plate has oversized hole or not

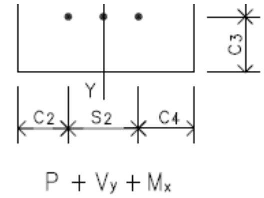
Mode 1 Failure cone at front anchors, strength check against 0.5 x V_u
Mode 3 Failure cone at front anchors, strength check against 1.0 x V_u, applicable when base plate has oversized holes

Mode 3 Oversized hole option is chosen, strength check against 1.0 x V_u
User can go to Anchor Bolt - Config & Setting to change the option



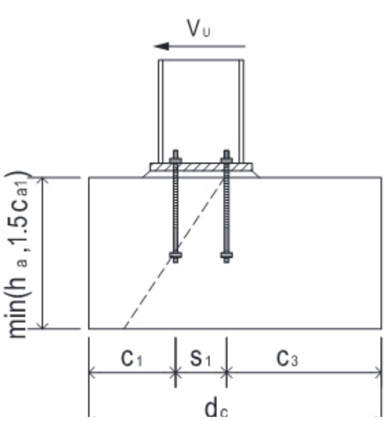
Anchor edge distance	c_1 = 11.443 [in]	c_2 = 13.203 [in]
	c_3 = 10.321 [in]	c_4 = 13.203 [in]
Anchor out-out spacing	s_1 = 25.115 [in]	s_2 = 14.500 [in]

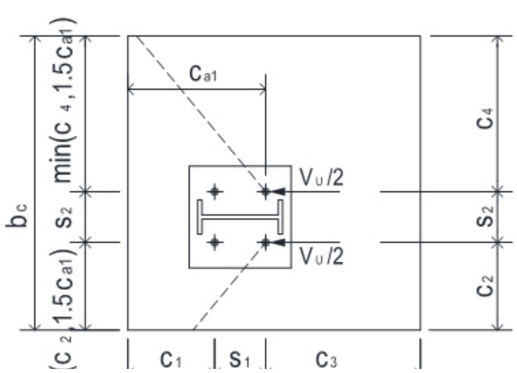




Anchor edge distance	c_1 = from user input	= 11.443	[in]	
Limiting c_{a1} when anchors are influenced by 3 or more edges		= No		ACI 318-19 17.7.2.1.2
Anchor edge distance - adjusted	$c_1 = c_{a1}$ needs NOT to be adjusted	= 11.443	[in]	
	c_2 = 13.203 [in] $\quad 1.5c_1$ = 17.164		[in]	
	$A_{Vc1} = [\min(c_2, 1.5c_1) + s_2 + \min(c_4, 1.5c_1)] \times \min(1.5c_1, h_a)$	= 702.10	[in^2]	ACI 318-19 Fig. R17.7.2.1b
Projected area of single anchor failure surface	$A_{Vco} = 4.5\, c_1^2$	= 589.20	[in^2]	ACI 318-19 17.7.2.1.3
No of <u>front</u> anchors resisting shear	n_s =	= 2		
Projected area of anchor group failure surface	$A_{Vc} = \min(A_{Vc1}, n_s A_{Vco})$	= 702.10	[in^2]	ACI 318-19 17.7.2.1.1
Anchor embedment & diameter	h_{ef} = 60.000 [in] $\quad d_a$ = 2.000		[in]	
Load-bearing length of anchor for shear	$l_e = \min(8d_a, h_{ef})$	= 16.000	[in]	ACI 318-19 17.7.2.2.1
Anchor edge distance & diameter	c_{a1} = 11.443 [in] $\quad d_a$ = 2.000		[in]	
Conc strength & lightweight factor	f_c = 4.0 [ksi] $\quad \lambda$ = 1.0			
	$V_{b1} = 7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\,\lambda\sqrt{f_c}\,c_{a1}^{1.5}$	= 36.73	[kips]	ACI 318-19 17.7.2.2.1a
	$V_{b2} = 9\,\lambda\sqrt{f_c}\,c_{a1}^{1.5}$	= 22.03	[kips]	ACI 318-19 17.7.2.2.1b
Single anchor shear breakout strength	$V_b = \min(V_{b1}, V_{b2})$	= 22.03	[kips]	ACI 318-19 17.7.2.2.1
Eccentricity modification factor	$\Psi_{ec,V}$ = shear acts through center of group	= 1.00		ACI 318-19 17.7.2.3.1
Edge modification factor	$\Psi_{ed,V} = \min[(0.7 + 0.3\, c_2 / 1.5c_1), 1.0]$	= 0.931		ACI 318-19 17.7.2.4.1
Conc cracking modification factor	$\Psi_{c,V}$ =	= 1.20		ACI 318-19 17.7.2.5.1
Anchor edge distance & conc thickness	c_{a1} = 11.443 [in] $\quad h_a$ = 216.000		[in]	
Conc breakout thickness factor	$\Psi_{h,V} = \left(\frac{1.5c_{a1}}{h_a}\right)^{0.5} \ge 1.0$	= 1.00		ACI 318-19 17.7.2.6.1
Strength reduction factor	ϕ_{vc} = 0.75 $\quad$ supplementary reinft present			ACI 318-19 17.5.3(b)
Concrete breakout resistance	$V_{cbg} = \phi_{vc}\frac{A_{Vc}}{A_{Vco}}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_b$	= 21.99	[kips]	ACI 318-19 17.7.2.1b
Mode 3 is used for checking	$V_{cbg1} = 1.0 \times V_{cbg}$	= **21.99**	[kips]	

Mode 2 Failure cone at back anchors





Anchor edge distance $c_{a1} = c_1 + s_1$ = 36.557 [in]

Limiting c_{a1} when anchors are influenced by 3 or more edges = No ACI 318-19 17.7.2.1.2

Anchor edge distance - adjusted $c_{a1} = c_{a1}$ needs NOT to be adjusted = 36.557 [in]

$c_2 = 13.203$ [in] $1.5c_1 = 54.836$ [in]

$A_{Vc1} = [\min(c_2, 1.5c_1) + s_2 + \min(c_4, 1.5c_1)] \times \min(1.5c_1, h_a)$ = 2243.1 [in^2] ACI 318-19 Fig. R17.7.2.1b

Projected area of single anchor failure surface $A_{Vco} = 4.5\ c_1^2$ = 6014.0 [in^2] ACI 318-19 17.7.2.1.3

No of <u>back</u> anchors resisting shear $n_s =$ = 2

Projected area of anchor group failure surface $A_{Vc} = \min(A_{Vc1}, n_s A_{Vco})$ = 2243.1 [in^2] ACI 318-19 17.7.2.1.1

Anchor embedment & diameter $h_{ef} = 60.000$ [in] $d_a = 2.000$ [in]

Load-bearing length of anchor for shear $l_e = \min(8d_a, h_{ef})$ = 16.000 [in] ACI 318-19 17.7.2.2.1

Anchor edge distance & diameter $c_{a1} = 36.557$ [in] $d_a = 2.000$ [in]

Conc strength & lightweight factor $f_c = 4.0$ [ksi] $\lambda = 1.0$

$V_{b1} = 7(\frac{l_e}{d_a})^{0.2}\sqrt{d_a}\ \lambda \sqrt{f_c}\ c_{a1}^{1.5}$ = 209.76 [kips] ACI 318-19 17.7.2.2.1a

$V_{b2} = 9\ \lambda \sqrt{f_c}\ c_{a1}^{1.5}$ = 125.82 [kips] ACI 318-19 17.7.2.2.1b

Single anchor shear breakout strength $V_b = \min(V_{b1}, V_{b2})$ = 125.82 [kips] ACI 318-19 17.7.2.2.1

Eccentricity modification factor $\Psi_{ec,V}$ = shear acts through center of group = 1.00 ACI 318-19 17.7.2.3.1

Edge modification factor $\Psi_{ed,V} = \min[(0.7 + 0.3\ c_2 / 1.5c_1), 1.0]$ = 0.772 ACI 318-19 17.7.2.4.1

Conc cracking modification factor $\Psi_{c,V} =$ = 1.20 ACI 318-19 17.7.2.5.1

Anchor edge distance & conc thickness $c_{a1} = 36.557$ [in] $h_a = 216.000$ [in]

Conc breakout thickness factor $\Psi_{h,V} = (\frac{1.5c_{a1}}{h_a})^{0.5} \geq 1.0$ = 1.00 ACI 318-19 17.7.2.6.1

Strength reduction factor $\phi_{vc} = 0.75$ supplementary reinft present ACI 318-19 17.5.3(b)

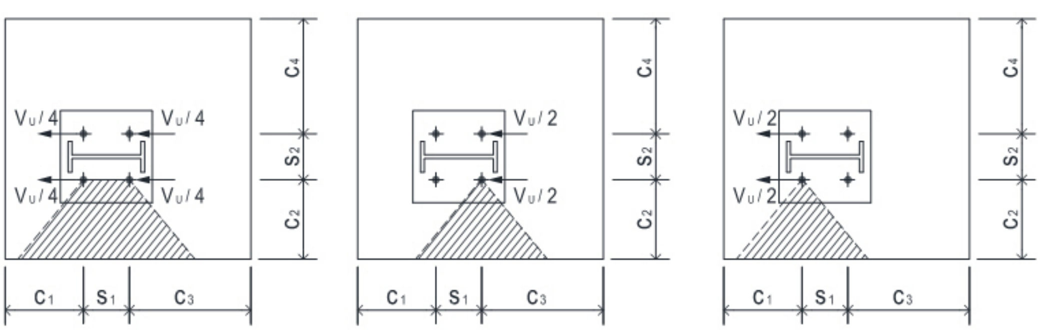
Concrete breakout resistance $V_{cbg2} = \phi_{vc} \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_b$ = **32.61** [kips] ACI 318-19 17.7.2.1b

Shear force in demand V_u = from user input = **1.50** [kips]

Min shear breakout resistance $\phi_{vc} V_{cbg} = \min(V_{cbg1}, V_{cbg2})$ = **21.99** [kips]

ratio = **0.07** $> V_u$ OK

Concrete Shear Breakout Resistance - Parallel To Edge ratio = 1.5 / 51.7 = **0.03** **PASS**



MODE 1 MODE 2 MODE 3

The case of shear parallel to an edge is shown in ACI 318-19 Fig. R17.7.2.1c. The maximum shear that can be applied parallel to the edge, V||, as governed by concrete breakout, is twice the maximum shear that can be applied perpendicular to the edge, V⊥.

ACI 318-19 Fig. R17.7.2.1c
shear force parallel to an edge

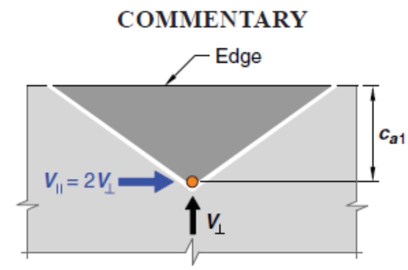
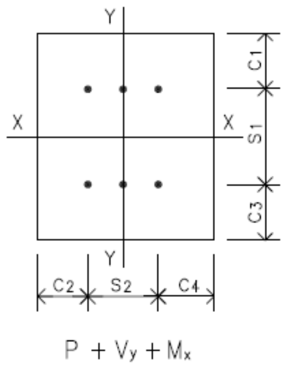


Fig. R17.7.2.1c—Shear force parallel to an edge.

Anchor edge distance	c_1 = 19.125 [in]	c_2 = 9.500 [in]
	c_3 = 10.321 [in]	c_4 = 9.500 [in]
Anchor out-out spacing	s_1 = 14.500 [in]	s_2 = 14.500 [in]



Mode 1 Shear taken evenly by all anchor bolts, strength check against 0.5 x V_u

Anchor edge distance	c_{a1} = min(c_2 , c_4)	= 9.500	[in]	
Limiting c_{a1} when anchors are influenced by 3 or more edges		= No		ACI 318-19 17.7.2.1.2
Anchor edge distance - adjusted	c_{a1} = c_{a1} needs NOT to be adjusted	= 9.500	[in]	
	c_1 = 19.125 [in]	c_3 = 10.321	[in]	
	s_1 = 14.500 [in]	$1.5c_{a1}$ = 14.250	[in]	
	A_{Vc1} = [min(c_1 ,1.5c_{a1})+s_1+min(c_3 ,1.5c_{a1})]x min(1.5c_{a1} , h_a)	= 556.76	[in^2]	ACI 318-19 Fig. R17.7.2.1b
Projected area of single anchor failure surface	$A_{Vco} = 4.5\, c_{a1}^2$	= 406.13	[in^2]	ACI 318-19 17.7.2.1.3
No of <u>front</u> anchors resisting shear	n_{bd} =	= 2		
Projected area of anchor group failure surface	A_{Vc} = min(A_{Vc1} , $n_{bd}\,A_{Vco}$)	= 556.76	[in^2]	ACI 318-19 17.7.2.1.1
Anchor embedment & diameter	h_{ef} = 60.000 [in]	d_a = 2.000	[in]	
Load-bearing length of anchor for shear	l_e = min(8d_a , h_{ef})	= 16.000	[in]	ACI 318-19 17.7.2.2.1
Anchor edge distance & diameter	c_{a1} = 9.500 [in]	d_a = 2.000	[in]	
Conc strength & lightweight factor	f_c = 4.0 [ksi]	λ = 1.0		
	$V_{b1} = 7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\,\lambda\,\sqrt{f_c}\,c_{a1}^{1.5}$	= 27.79	[kips]	ACI 318-19 17.7.2.2.1a
	$V_{b2} = 9\,\lambda\,\sqrt{f_c}\,c_{a1}^{1.5}$	= 16.67	[kips]	ACI 318-19 17.7.2.2.1b
Single anchor shear breakout strength	V_b = min(V_{b1} , V_{b2})	= 16.67	[kips]	ACI 318-19 17.7.2.2.1
Eccentricity modification factor	$\Psi_{ec,V}$ = shear acts through center of group	= 1.00		ACI 318-19 17.7.2.3.1
Edge modification factor	$\Psi_{ed,V}$ = 1.0 for shear parallel to an edge case	= 1.000		ACI 318-19 Fig. R17.7.2.1b Case 2
Conc cracking modification factor	$\Psi_{c,V}$ =	= 1.20		ACI 318-19 17.7.2.5.1
Anchor edge distance & conc thickness	c_{a1} = 9.500 [in]	h_a = 216.000	[in]	
Conc breakout thickness factor	$\Psi_{h,V} = \left(\frac{1.5c_{a1}}{h_a}\right)^{0.5} \geq 1.0$	= 1.00		ACI 318-19 17.7.2.6.1

For **Mode 1** $V_{cbg\text{-}p1}$ is supposed to check against $0.5V_u$, in terms of utilization ratio $\dfrac{0.5V_u}{V_{cbg\text{-}p1}} = \dfrac{V_u}{2V_{cbg\text{-}p1}}$

Strength reduction factor	$\phi_{vc} = 0.75$ supplementary reinft present			ACI 318-19 17.5.3(b)
Concrete breakout resistance	$V_{cbg\text{-}p1} = 2\text{x}\ \phi_{vc}\dfrac{A_{Vc}}{A_{Vco}}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_b$	= **41.13**	[kips]	ACI 318-19 17.7.2.1b

---

Mode 2 Shear taken evenly by back anchor bolts, strength check against 0.5 x V_u

Anchor edge distance	$c_{a1} = \min(c_2, c_4)$	= 9.500	[in]	
Limiting c_{a1} when anchors are influenced by 3 or more edges		= No		ACI 318-19 17.7.2.1.2
Anchor edge distance - adjusted	$c_{a1} = c_{a1}$ needs NOT to be adjusted	= 9.500	[in]	
	$c_1 = 19.125$ [in]	$c_3 = 10.321$	[in]	
	$s_1 = 14.500$ [in]	$1.5c_{a1} = 14.250$	[in]	
	$A_{Vc1} = [\min(s_1+c_1, 1.5c_{a1}) + \min(c_3, 1.5c_{a1})] \text{x} \min(1.5c_{a1}, h_a)$	= 350.14	[in^2]	ACI 318-19 Fig. R17.7.2.1b
Projected area of single anchor failure surface	$A_{Vco} = 4.5\ c_{a1}^2$	= 406.13	[in^2]	ACI 318-19 17.7.2.1.3
No of <u>front</u> anchors resisting shear	$n_{bd} =$	= 2		
Projected area of anchor group failure surface	$A_{Vc} = \min(A_{Vc1}, n_{bd}A_{Vco})$	= 350.14	[in^2]	ACI 318-19 17.7.2.1.1
Anchor embedment & diameter	$h_{ef} = 60.000$ [in]	$d_a = 2.000$	[in]	
Load-bearing length of anchor for shear	$l_e = \min(8d_a, h_{ef})$	= 16.000	[in]	ACI 318-19 17.7.2.2.1
Anchor edge distance & diameter	$c_{a1} = 9.500$ [in]	$d_a = 2.000$	[in]	
Conc strength & lightweight factor	$f_c = 4.0$ [ksi]	$\lambda = 1.0$		
	$V_{b1} = 7(\dfrac{l_e}{d_a})^{0.2}\sqrt{d_a}\ \lambda\sqrt{f_c}\ c_{a1}^{1.5}$	= 27.79	[kips]	ACI 318-19 17.7.2.2.1a
	$V_{b2} = 9\ \lambda\sqrt{f_c}\ c_{a1}^{1.5}$	= 16.67	[kips]	ACI 318-19 17.7.2.2.1b
Single anchor shear breakout strength	$V_b = \min(V_{b1}, V_{b2})$	= 16.67	[kips]	ACI 318-19 17.7.2.2.1
Eccentricity modification factor	$\Psi_{ec,V} =$ shear acts through center of group	= 1.00		ACI 318-19 17.7.2.3.1
Edge modification factor	$\Psi_{ed,V} =$ 1.0 for shear parallel to an edge case	= 1.000		ACI 318-19 Fig. R17.7.2.1b Case 2
Conc cracking modification factor	$\Psi_{c,V} =$	= 1.20		ACI 318-19 17.7.2.5.1
Anchor edge distance & conc thickness	$c_{a1} = 9.500$ [in]	$h_a = 216.000$	[in]	
Conc breakout thickness factor	$\Psi_{h,V} = (\dfrac{1.5c_{a1}}{h_a})^{0.5} \geq 1.0$	= 1.00		ACI 318-19 17.7.2.6.1

For **Mode 2** $V_{cbg\text{-}p1}$ is supposed to check against $0.5V_u$, in terms of utilization ratio $\dfrac{0.5V_u}{V_{cbg\text{-}p1}} = \dfrac{V_u}{2V_{cbg\text{-}p1}}$

Strength reduction factor	$\phi_{vc} = 0.75$ supplementary reinft present			ACI 318-19 17.5.3(b)
Concrete breakout resistance	$V_{cbg\text{-}p2} = 2\text{x}\ \phi_{vc}\dfrac{A_{Vc}}{A_{Vco}}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_b$	= **25.86**	[kips]	ACI 318-19 17.7.2.1b

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Mode 3 Shear taken evenly by front anchor bolts, strength check against 0.5 x V_u

Anchor edge distance	$c_{a1} = \min(c_2, c_4)$	= 9.500	[in]	
Limiting c_{a1} when anchors are influenced by 3 or more edges		= No		ACI 318-19 17.7.2.1.2
Anchor edge distance - adjusted	$c_{a1} = c_{a1}$ needs NOT to be adjusted	= 9.500	[in]	
	$c_1 = 19.125$ [in]	$c_3 = 10.321$	[in]	
	$s_1 = 14.500$ [in]	$1.5c_{a1} = 14.250$	[in]	
				ACI 318-19

	$A_{Vc1} = [\min(c_1, 1.5c_{a1}) + \min(s_1 + c_3, 1.5c_{a1})] \times \min(1.5c_{a1}, h_a)$	= 406.13	[in²]	ACI 318-19 Fig. R17.7.2.1b
Projected area of single anchor failure surface	$A_{Vco} = 4.5\, c_{a1}^2$	= 406.13	[in²]	ACI 318-19 17.7.2.1.3
No of <u>front</u> anchors resisting shear	$n_{bd} =$	= 2		
Projected area of anchor group failure surface	$A_{Vc} = \min(A_{Vc1}, n_{bd} A_{Vco})$	= 406.13	[in²]	ACI 318-19 17.7.2.1.1
Anchor embedment & diameter	$h_{ef} = 60.000$ [in]	$d_a = 2.000$	[in]	
Load-bearing length of anchor for shear	$l_e = \min(8d_a, h_{ef})$	= 16.000	[in]	ACI 318-19 17.7.2.2.1
Anchor edge distance & diameter	$c_{a1} = 9.500$ [in]	$d_a = 2.000$	[in]	
Conc strength & lightweight factor	$f_c = 4.0$ [ksi]	$\lambda = 1.0$		
	$V_{b1} = 7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\, \lambda \sqrt{f_c}\, c_{a1}^{1.5}$	= 27.79	[kips]	ACI 318-19 17.7.2.2.1a
	$V_{b2} = 9\, \lambda \sqrt{f_c}\, c_{a1}^{1.5}$	= 16.67	[kips]	ACI 318-19 17.7.2.2.1b
Single anchor shear breakout strength	$V_b = \min(V_{b1}, V_{b2})$	= 16.67	[kips]	ACI 318-19 17.7.2.2.1
Eccentricity modification factor	$\Psi_{ec,V}$ = shear acts through center of group	= 1.00		ACI 318-19 17.7.2.3.1
Edge modification factor	$\Psi_{ed,V}$ = 1.0 for shear parallel to an edge case	= 1.000		ACI 318-19 Fig. R17.7.2.1b Case 2
Conc cracking modification factor	$\Psi_{c,V} =$	= 1.20		ACI 318-19 17.7.2.5.1
Anchor edge distance & conc thickness	$c_{a1} = 9.500$ [in]	$h_a = 216.000$	[in]	
Conc breakout thickness factor	$\Psi_{h,V} = \left(\frac{1.5c_{a1}}{h_a}\right)^{0.5} \ge 1.0$	= 1.00		ACI 318-19 17.7.2.6.1

For **Mode 3** V_{cbg-p1} is supposed to check against $0.5V_u$, in terms of utilization ratio $\frac{0.5V_u}{V_{cbg-p1}} = \frac{V_u}{2V_{cbg-p1}}$

Strength reduction factor	$\phi_{vc} = 0.75$ supplementary reinft present			ACI 318-19 17.5.3(b)
Concrete breakout resistance	$V_{cbg-p3} = 2x\, \phi_{vc} \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_b$	= **30.00**	[kips]	ACI 318-19 17.7.2.1b
Shear force in demand	V_u = from user input	= **1.50**	[kips]	
Min shear breakout resistance - parallel to edge	$\phi_{vc} V_{cbg-p} = \min(V_{cbg-p1}, V_{cbg-p2}, V_{cbg-p3})$ x2 side	= **51.73**	[kips]	
	ratio = **0.03**	$> V_u$	OK	

Concrete Pryout Shear Resistance ratio = 1.5 / 74.2 = **0.02** **PASS**

Shear load on anchor group	V_u = from user load input	= **1.50**	[kips]	
Anchor embedment depth-adjusted	h_{ef} = from Anchor Forces Calculation above	= 12.750	[in]	
	$k_{cp} = 2.0$			ACI 318-19 17.7.3.1 (b)
Concrete breakout resistance	N_{cbg} = from above calculation	= 53.01	[kips]	
Strength reduction factor	$\phi_{vc} = 0.7$ pryout strength is always Condition B			17.5.3(c)
	$\phi_{vc} V_{cpg} = \phi_{vc} k_{cp} N_{cbg}$	= 74.21	[kips]	ACI 318-19 17.7.3.1b
	ratio = **0.02**	$> V_u$	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	= 135.72	[kips]
Anchor conc shear breakout resistance - perpendicular to edge	ϕV_{cbg} =	from anchor conc breakout calc above	= 21.99	[kips]
Anchor conc shear breakout resistance - parallel to edge	ϕV_{cbg-p} =	from anchor conc breakout calc above	= 51.73	[kips]
Anchor conc shear pryout resistance	ϕV_{cpg} =	from anchor conc pryout calc above	= 74.21	[kips]
Anchor group governing shear resistance	ϕV_n =	minimum of above values	= **21.99**	[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	= 34.58	[kips]
Anchor group shear load	V_u =	from user load input	= 1.50	[kips]
Anchor group governing tensile resistance	ϕN_n =	from calc in above section	= 39.75	[kips]
Anchor group governing shear resistance	ϕV_n =	from calc in above section	= 21.99	[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 **N/A**

Seismic - Tension Not Applicable ACI 318-19 17.10.5.1

Seismic SDC < C or E <= 0.2U , additional seismic requirements in ACI 318-19 17.10.5.3 is NOT required ACI 318-19 17.10.5.3

Seismic - Shear Not Applicable ACI 318-19 17.10.6.1

Seismic SDC < C or E <= 0.2U , additional seismic requirements in ACI 318-19 17.10.6.3 is NOT required ACI 318-19 17.10.6.3

Base Plate - Load Case 1 **P + M_x** P_c =3.9 kip M_x =79.0 kip-ft Code=ACI 318-19

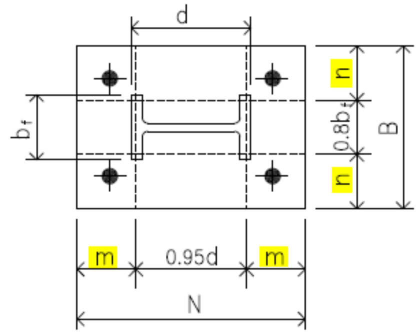
Result Summary geometries & weld limitations = **PASS** limit states max ratio = **0.86** **PASS**

Minimum Base Plate Thickness for Rigidity ratio = 2.575 / 3.000 = **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.

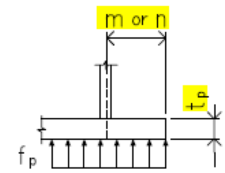
User can turn this check On/Off in Anchor Bolt - Config & Setting by checking or unchecking the option of Min base plate thickness $t_p \geq$ max of base plate overhangs m/4 and n/4



Column sect Custom Sect	d = 18.000 [in]	b_f = 18.000 [in]	
Base plate width & depth	B = 35.000 [in]	N = 33.000 [in]	

AISC Design Guide 1 - 3.1.2 on Page 15

Base plate cantilever dimension	m = (N - 0.8 d) / 2	= 9.300	[in]
	n = (B - 0.8 b_f) / 2	= 10.300	[in]
Base plate thickness	t_p = from user input	= **3.000**	[in]



Suggested minimum base plate thickness for rigidity	t_{min} = max (m/4 , n/4)	= **2.575**	[in]	
	ratio = **0.86**	< t_p		OK

Base Plate Thickness Check ratio = 0.930 / 3.000 = **0.31** **PASS**

User can refer to Anchor Forces Calculation in Anchor Bolt calculation section for details of max anchor tensile load T_1 and conc stress $f_{p(max)}$ calculation. T_1 and $f_{p(max)}$ are used below to check base plate thickness.

Anchor Rod Steel Tensile Capacity

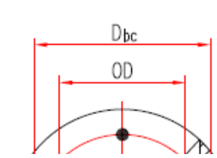
Bolt line 1 - single anchor T_1	T_1 = 12.20 [kips]	n_1 = 1			
Sum of all anchors tensile force along bolt line 1	$T_u = n_1 \times T_1$	= **12.20**	[kips]		
Anchor rod effective section area	A_{se} = 2.50 [in^2]	f_{uta} = 58.0	[ksi]		
Strength reduction factor	ϕ_{ts} = 0.75			ACI 318-19 17.5.3(a)	
Anchor rod tensile resistance	$T_r = \phi_{ts} n_t A_{se} f_{uta}$	= **108.75**	[kips]	ACI 318-19 17.6.1.2	
	ratio = **0.11**	> T_u		OK	

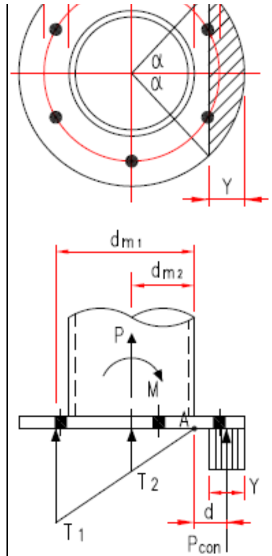
Base Plate Flexure Caused by Anchor Rod Tension **AISC Design Guide 1**

Circular anchor bolt circle dia & column OD	D_{bc} = 29.000 [in]	OD = 18.000	[in]	
Max anchors tensile force	$T_u = T_1$	= 12.20	[kips]	
T_u to CHS wall moment lever arm	x = 0.5 (D_{bc} - OD)	= 5.500	[in]	
Base plate width & strength	B = 11.000 [in]	F_y = 50.0	[ksi]	
	$t_{req\text{-}t} = 2.11 \left(\frac{T_u x}{B F_y} \right)^{0.5}$	= 0.737	[in]	Eq 3.4.7a

Base Plate Flexure Caused by Conc Bearing Pressure

Circular anchor bolt circle dia & base plate dia	D_{bc} = 29.000 [in]	D_{bp} = 35.000	[in]
Column sect Custom Sect	OD = 18.000 [in]		





Full Bearing Stress Geometrics When Ignoring CHS Column

Factored forces on base plate	P_u = 3.90	[kips]	M_u = 79.00	[kip-ft]
Eccentricity	$e = M_u / P_u$		= 243.077	[in]
Critical eccentricity	e_{crit} = from Anchor Forces Calculation		= 17.335	[in]

when $e > e_{crit}$, large moment case applied. There are tensile forces mobolized in anchors and max allowed concrete bearing stress $f_{p(max)}$ will be used to calculate base plate bending moment.

Full stress block length Y and angle under base plate

Stress block length and angle at Y	Y = 1.373	[in]	α = 22.850	
Stress block area & centroid to CHS column center distance	A = 12.54	[in^2]	c_y = 16.678	[in]
Pedestal max bearing stress	$f_{p(max)}$ = from Anchor Forces Calculation		= 3.031	[ksi]

Bearing Stress Geometrics When Consider Bending to CHS Column Wall

Base plate overhang	$m = 0.5\ (D_{bp} - OD)$		= 8.500	[in]

When Y<= m , all stress under base plate contributes to the base plate bending

Base plate width for bending to CHS column wall edge	B =		= 30.017	[in]
Stress block area & stress resultant to CHS wall edge distance	A = 12.54	[in^2]	d_c = 7.678	[in]
Resistance moment by concrete stress resultant reaction	$M_{rc} = f_{p(max)}\, A\, d_c$		= 24.32	[kip-ft]
Base plate strength & strength reduction factor	F_y = 50.0	[ksi]	ϕ_b = 0.90	
	$t_{req\text{-}b1} = 2.11\ \left(\frac{M_{rc}}{B\ F_y}\right)^{0.5}$		= 0.930	[in]
Base plate thickness	t_p = from user input		= **3.000**	[in]
Min required plate thickness	$t_{min} = \max\ (t_{req\text{-}t}, t_{req\text{-}b1})$		= **0.930**	[in]
	ratio = **0.31**		$< t_p$	OK