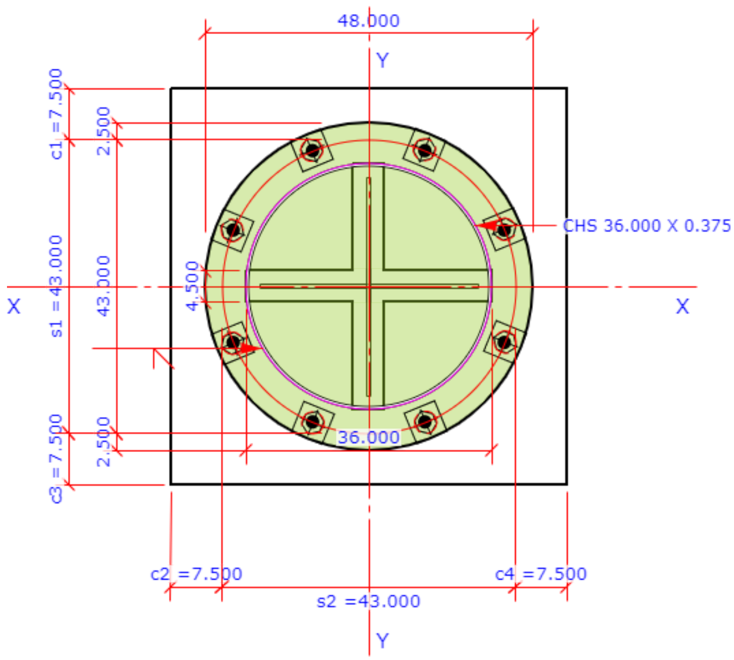
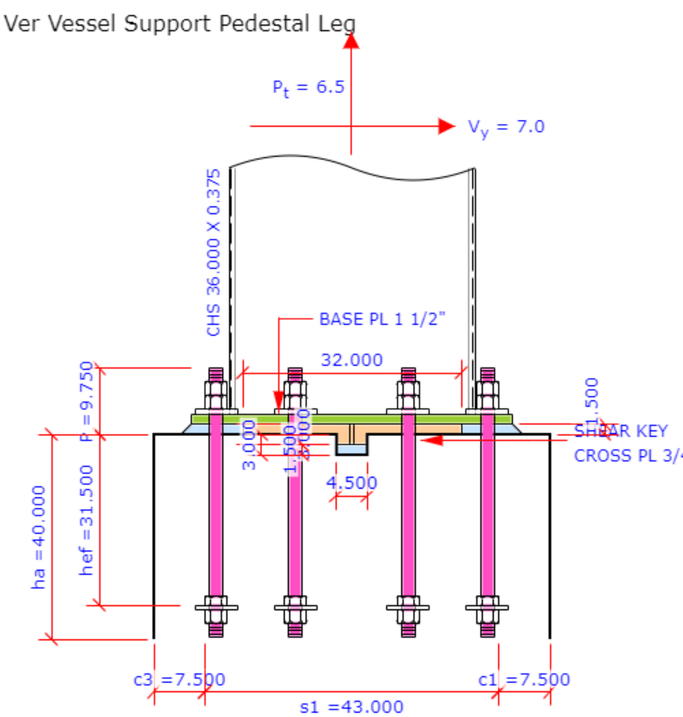
Result Summary - Overall	Anchorage Design	Code=ACI 318-19

Result Summary - Overall	geometries & weld limitations = **PASS**	limit states max ratio = **0.19**	**PASS**
Vertical Vessel Leg Anchor	geometries & weld limitations = **PASS**	limit states max ratio = **0.06**	**PASS**
Vertical Vessel Leg Shear Key	geometries & weld limitations = **PASS**	limit states max ratio = **0.19**	**PASS**
Vertical Vessel Leg Base Plate	geometries & weld limitations = **PASS**	limit states max ratio = **0.13**	**PASS**

Sketch	Anchorage Design	Code=ACI 318-19





Vertical Vessel Anchor Forces Calculation

Max Tensile / Shear Load on Vertical Vessel Single Pedestal Leg

Anchor bolt bolt circle diameter D_{bc} = from user input = 605.98 [in]

No of pedestal leg N_a = from user input = 9

Circular pattern leg section modulus S = = 1363.46 [in]

Factored <u>single</u> leg shear load $V_{ua} = \dfrac{V_u}{N_a}$ = **7.00** [kips]

Anchor Tensile - Uplift LCB by Wind

Factored base moment - wind M_{uw} = from user input = 1158.7 [kip-ft]

Vessel empty weight D_e = from user input = 490.50 [kips]

When $\dfrac{M_{uw}}{S} < 0.9\dfrac{D_e}{N_a}$, there is no tensile load mobilized on anchor

Factored <u>single</u> leg tensile load $N_{uaw} = \dfrac{M_{uw}}{S} - 0.9\dfrac{D_e}{N_a}$ = 0.00 [kips]

Anchor Tensile - Uplift LCB by Seismic

Factored base moment - seismic M_{us} = from user input = 62227.0 [kip-ft]

Vessel operating weight D_o = from user input = 5411.3 [kips]

Factored <u>single</u> leg tensile load $N_{uas} = \dfrac{M_{us}}{S} - 0.9\dfrac{D_o}{N_a}$ = 6.54 [kips]

Factored <u>single</u> anchor tensile load - max $N_{ua} = \max(N_{uaw}, N_{uas})$ = **6.54** [kips]

Anchor Forces Calculation

Anchor Tensile Force Calculation

User Input

Anchor edge distance

c_{1u} = 7.500 [in] c_{2u} = 7.500 [in]

c_{3u} = 7.500 [in] c_{4u} = 7.500 [in]

Anchor out-out spacing

s_{1u} = 43.000 [in] s_{2u} = 43.000 [in]

Anchor embedment depth

h_{ef} = 31.500 [in]

Anchor Load on Vertical Vessel Pedestal Leg

Refer to Vertical Vessel Anchor Forces Calculation for details of anchor forces calculation shown below

Axial force

Axial P = -6.54 [kips] in tension

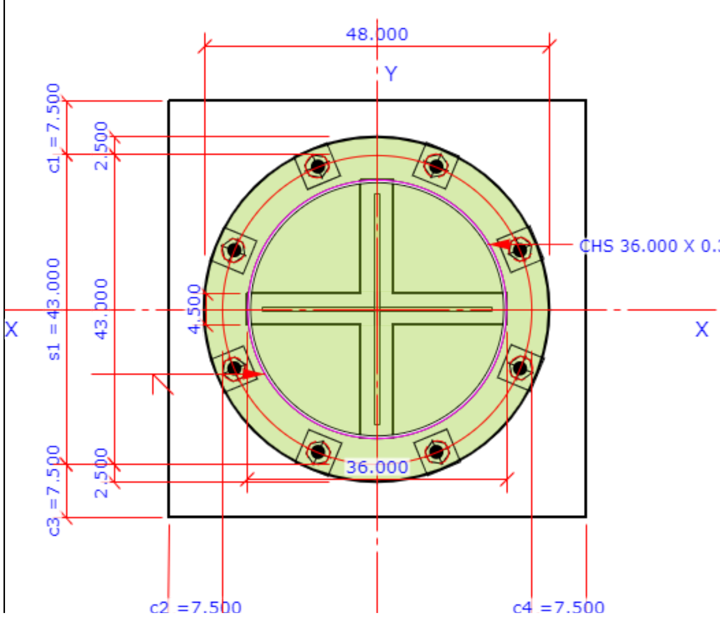
Shear forces

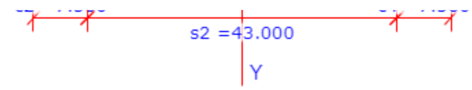
V_y = 7.00 [kips] V_x = 0.00 [kips]

Moment forces

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

Anchor Layout Plan

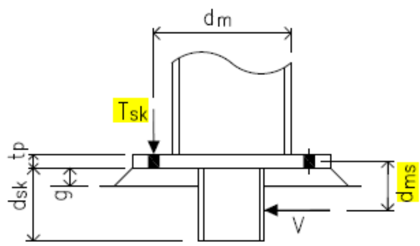




Load Case 1 - Anchor Additional Tension From Moment Caused by V_y

Shear key shear force	V_u = from ver vessel anchor load calc	= 7.00	[kips]

Refer to sketch below , shear key's shear reaction takes moment to base plate center line and this moment will cause additional tensile force on anchors



Base plate & grout thickness	t_p = 1.500 [in]	g = 1.500	[in]
Shear key depth	d_{sk} = 3.000 [in]		
Shear key shear V to base plate center moment arm distance	$d_{ms} = 0.5(d_{sk} - g) + g + 0.5\, t_p$	= 3.000	[in]
Moment by shear key shear	$M_u = V_u\, d_{ms}$	= 1.75	[kip-ft]
Anchor out-out spacing - in shear direction	s_1 = from user input	= 43.000	[in]
Column depth - in shear direction	d = sect Custom Sect	= 36.000	[in]
Exterior anchhor moment arm	$d_m = d + 0.5(s_1 - d)$	= 39.500	[in]
Anchor number along exterior anchhor	n_{bw} = from user input	= 2	
Single anchor tension from moment caused by shear key	$T_{sk} = \dfrac{M_u}{d_m \times n_{bw}}$	= **0.27**	[kips]

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

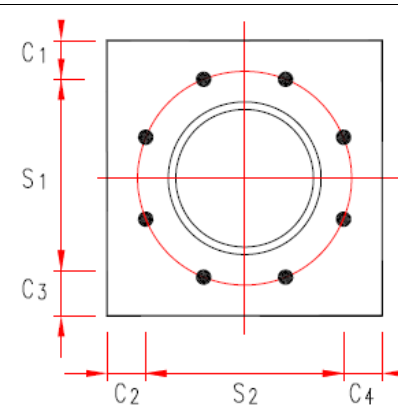
Anchor group edge distances are re-calculated to the most exterior anchors as user input edge distances is edge distances to the bolt circle diameter

Anchor Group Dimensions

Anchor bolt circle dia & pedestal dia	D_{bc} = 43.000	[in]	D_{pd} = 58.000	[in]
Anchor spacing	s_1 = 39.727	[in]	s_2 = 39.727	[in]
Anchor edge distance	c_1 = 9.137	[in]	c_2 = 9.137	[in]
	c_3 = 9.137	[in]	c_4 = 9.137	[in]

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

Max anchor spacing within the tensile anchors group	s_{1max} = 16.455	[in]	s_{2max} = 39.727	[in]



Anchor embedment depth - from user input	h_{ef} = from user input	= 31.500	[in]
Anchors are located less than 1.5h_{ef} f...	= Yes		

from three or more edges				
Max of edge distances not exceeding $1.5h_{ef}$	$c_{a,max}$ =	= 9.137	[in]	
Max spacing between anchors within the group	s =	= 39.727	[in]	
Anchor embedment depth - adjusted	h_{ef} = max ($c_{a,max}$/1.5 , s /3)	= **13.242**	[in]	ACI 318-19 17.6.2.1.2

Vertical Vessel Anchor Bolt P_t =-6.5 kip V =7.0 kip Code=ACI 318-19

Result Summary geometries & weld limitations = **PASS** limit states max ratio = **0.06** **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 = 9.137 [in]	c_2 = 9.137	[in]	
	c_3 = 9.137 [in]	c_4 = 9.137	[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)	= **9.137**	[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	= C2		
Min anchor spacing	s = from user input	= **16.455**	[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	= **31.500**	[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)

TER	TIONAL D IN	UT	TYPE 2 AD H AT M OF OR	*ASCE ANCHORAGE DESIGN REPORT* **MINIMUM DIMENSIONS** (Note 1)	SLEEVES

ANCHOR ROD DIAME	EFFECTIVE CROSS-SECT AREA OF ANCHOR RO TENSION (Note 3)	HEAVY HEX HEAD/N WIDTH	ANCHOR THRE LENGT BOTTO ANCH		h_{ef}	EDGE DISTANCE c_a (Note 2)		SPAC -ING	SLEEVES (See Note 1 (d))		
			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.

Anchor Rod Tensile Resistance ratio = 1.1 / 108.8 = **0.01** **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

Anchor group axial tensile force	P = from user load input	= -6.54	[kips]	in tension
No of anchors in the group	n_t =	= 8		

Refer to Anchor Forces Calculation section above , shear key's shear reaction takes moment to base plate center line and this moment will cause additional tensile force on anchors

Single anchor tension from moment caused by shear key reaction force	T_{sk} = from Anchor Forces Calculation above	= 0.27	[kips]	in tension
Single anchor tensile force	$T = T_{sk} - P / n_t$	= **1.08**	[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 = **0.01**	> T	OK	

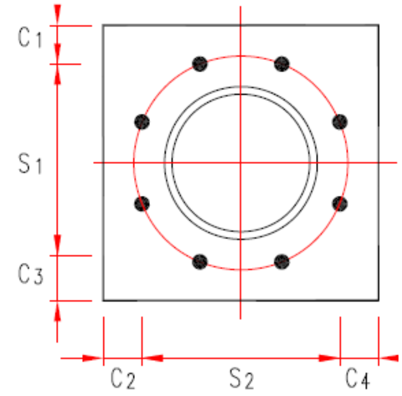
Anchor Concrete Tensile Breakout Resistance ratio = 6.5 / 104.8 = **0.06** **PASS**

Anchor embedment depth-adjusted	h_{ef} = from Anchor Forces Calculation above	= 13.242	[in]	
Conc strength & lightweight conc factor	f_c = 4.4 [ksi]	λ = 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"	= 78.22	[kips]	ACI 318-19 17.6.2.2.1
	$16\lambda\sqrt{f_c}\,h_{ef}^{5/3}$ If 11" ≤ h_{ef} ≤ 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} = 43.000	[in]	D_{pd} = 58.000	[in]
Anchor spacing	s_1 = 39.727	[in]	s_2 = 39.727	[in]
Anchor edge distance	c_1 = 9.137	[in]	c_2 = 9.137	[in]
	c_3 = 9.137	[in]	c_4 = 9.137	[in]
Anchor embedment depth-adjusted	h_{ef} = from calc above	= 13.242	[in]	
Anchor group projected conc failure area	A_{NC1} =	= 3364.1	[in^2]	



	$A_{Nco} = 9\,h_{ef}^2$	= 1578.2	[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	= 8		
	$A_{Nc} = \min(A_{Nc1}, n_t A_{Nco})$	= 3364.1	[in^2]	ACI 318-19 17.6.2.1.1
Eccentricity modification factor	$\Psi_{ec,N}$ = from Anchor Forces Calculation above	= 1.000		ACI 318-19 17.6.2.3.1
Min edge distance	$c_{min} = \min(c_1, c_2, c_3, c_4)$	= 9.137	[in]	
Edge modification factor	$\Psi_{ed,N} = \min[0.7 + \frac{0.3c_{min}}{1.5h_{ef}}, 1.0]$	= 0.838		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	$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$	= 139.72	[kips]	ACI 318-19 17.6.2.1b
Sum of anchors tensile force in anchor group	N_u = from Anchor Forces Calculation above	= **6.54**	[kips]	
Strength reduction factor	ϕ_{tc} = 0.75 supplementary reinft present			ACI 318-19 17.5.3(b)
	$\phi_{tc}N_{cbg}$ = 0.75 x 139.72	= 104.79	[kips]	
Seismic design strength reduction	= x 1.0 not applicable	= **104.79**	[kips]	ACI 318-19 17.10.5.4(b)
	ratio = **0.06**	> N_u	OK	

Anchor Pullout Resistance ratio = 1.1 / 129.5 = **0.01** **PASS**

Anchor head net bearing area & conc strength A_{brg} = 5.32 [in^2] f_c = 4.4 [ksi]

Single bolt pullout resistance $N_p = 8\ A_{brg}\ f_c$ = 185.00 [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

Anchor group axial tensile force P = from user load input = -6.54 [kips] in tension

No of anchors in the group n_t = = 8

Refer to Anchor Forces Calculation section above , shear key's shear reaction takes moment to base plate center line and this moment will cause additional tensile force on anchors

Single anchor tension from moment caused by shear key reaction force T_{sk} = from Anchor Forces Calculation above = 0.27 [kips] in tension

Single anchor tensile force $T = T_{sk} - P\ /\ n_t$ = **1.08** [kips]

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

$\phi_{tc} N_{pn} = \phi_{tc} \Psi_{cP} N_p$ = **129.50** [kips]

Seismic design strength reduction = x 1.0 not applicable = **129.50** [kips] ACI 318-19 17.10.5.4(c)

ratio = **0.01** > T OK

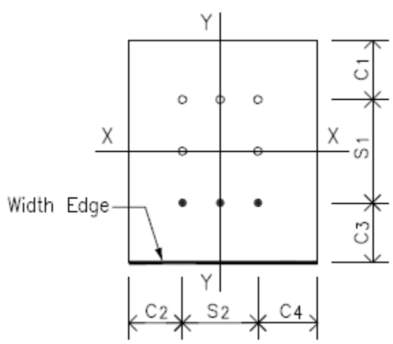
Anchor Side Blowout Resistance ratio = 1.1 / 83.4 = **0.01** **PASS**

Anchor Inputs

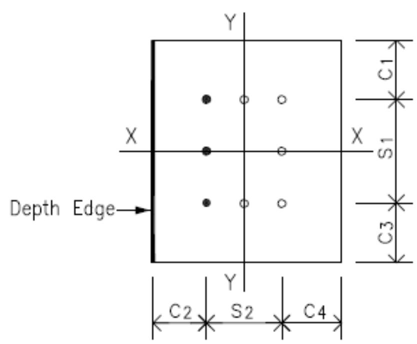
Anchor edge distance c_1 = 9.137 [in] c_2 = 9.137 [in]

c_3 = 9.137 [in] c_4 = 9.137 [in]

Anchor out-out spacing s_1 = 39.727 [in] s_2 = 39.727 [in]



Side Blowout Width Edge



Side Blowout Depth Edge

Side Edges Along X-X Axis - Width Edges

Anchor edge distance in Y direction $c_{a1} = \min(c_1, c_3)$ = 9.137 [in]

Anchor embedment depth h_{ef} = from user input = 31.500 [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 = 39.727 [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 = 4.4 [ksi]

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

Single anchor side blowout capacity $N_{sb} = 160\ c_{a1} \sqrt{A_{brg}}\ \lambda_a \sqrt{f_c}$ = 222.31 [kips] ACI 318-19 17.6.4.1

Single anchor side blowout capacity $N_{sb} = 160\, c_{a1} \sqrt{A_{brg}}\, \lambda \sqrt{f_c}$ = 222.31 [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 $s_b = s_2 / (n_w - 1)$ = 39.727 [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 $= 1 + \frac{s_2}{6c_{a1}}$ = 1.72 — ACI 318-19 17.6.4.2

Group anchor side blowout capacity $N_{sbg} = (1 + \frac{s_2}{6c_{a1}})\, N_{sb}$ = **383.41** [kips]

Max Single Anchor Tensile Force

Anchor group axial tensile force P = from user load input = -6.54 [kips] in tension

No of anchors in the group n_t = = 8

Refer to Anchor Forces Calculation section above , shear key's shear reaction takes moment to base plate center line and this moment will cause additional tensile force on anchors

Single anchor tension from moment caused by shear key reaction force T_{sk} = from Anchor Forces Calculation above = 0.27 [kips] in tension

Single anchor tensile force $T = T_{sk} - P / n_t$ = **1.08** [kips]

No of anchors along side blowout edge n_{bw} = from user input = 2

Tensile force - anchors along potential blowout edge $T_w = n_{bw} \times T$ = **2.17** [kips]

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

$\phi_{tc} N_{sbg} = 0.75 \times 383.41$ = 287.56 [kips]

Seismic design strength reduction = x 1.0 not applicable = **287.56** [kips] — ACI 318-19 17.10.5.4(d)

ratio = **0.01** $> T_w$ 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 $n_t = 8$ $n_{bw} = 2$

Group anchor tensile side blowout capacity $= 287.56 \frac{n_t}{n_{bw}}$ = **1150.2** [kips]

Side Edges Along Y-Y Axis - Depth Edges

Anchor edge distance in X direction $c_{a2} = \min(c_2, c_4)$ = 9.137 [in]

Anchor embedment depth h_{ef} = from user input = 31.500 [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 out-out distance edges along X direction s_1 = from user input = 39.727 [in]

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

Anchor head net bearing area & conc strength $A_{brg} = 5.32$ [in^2] $f_c = 4.4$ [ksi]

Lightweight conc modification factor $\lambda = 1.0$ — ACI 318-19 17.2.4.1

Single anchor side blowout capacity $N_{sb} = 160\, c_{a2} \sqrt{A_{brg}}\, \lambda \sqrt{f_c}$ = 222.31 [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	$s_b = s_1 / (n_d - 1)$	= 39.727	[in]	
Multiple tensile anchors space close and work as group or not	= check if $s_b < 6\ c_{a2}$	= True		ACI 318-19 17.6.4.2
Multiple anchors group factor	$= 1 + \frac{s_1}{6c_{a2}}$	= 1.72		ACI 318-19 17.6.4.2
Group anchor side blowout capacity	$N_{sbg} = (1 + \frac{s_1}{6c_{a2}})\ N_{sb}$	= **383.41**	[kips]	

Max Single Anchor Tensile Force

Anchor group axial tensile force	P = from user load input	= -6.54	[kips]	in tension
No of anchors in the group	$n_t =$	= 8		

Refer to Anchor Forces Calculation section above , shear key's shear reaction takes moment to base plate center line and this moment will cause additional tensile force on anchors

Single anchor tension from moment caused by shear key reaction force	T_{sk} = from Anchor Forces Calculation above	= 0.27	[kips]	in tension
Single anchor tensile force	$T = T_{sk} - P / n_t$	= **1.08**	[kips]	
No of anchors along side blowout edge	n_{bd} = from user input	= 2		
Tensile force - anchors along potential blowout edge	$T_d = n_{bd} \times T$	= **2.17**	[kips]	
Strength reduction factor	$\phi_{tc} = 0.75$ supplementary reinft present			ACI 318-19 17.5.3(b)
	$\phi_{tc} N_{sbg} = 0.75 \times 383.41$	= 287.56	[kips]	
Seismic design strength reduction	= x 1.0 not applicable	= **287.56**	[kips]	ACI 318-19 17.10.5.4(d)
	ratio = **0.01**	> 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	$n_t = 8$	$n_{bd} = 2$		
Group anchor tensile side blowout capacity	$= 287.56 \frac{n_t}{n_{bd}}$	= **1150.2**	[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	$c_{a1} = \min(c_1, c_3)$	= 9.137	[in]	
	$c_{a2} = \min(c_2, c_4)$	= 9.137	[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	$N_{sb1} = (1 + \frac{c_{a2}}{c_{a1}})\ /4 \times N_{sb}$	= 111.16	[kips]	

Max Single Anchor Tensile Force

Anchor group axial tensile force	P = from user load input	= -6.54	[kips]	in tension
No of anchors in the group	$n_t =$	= 8		

Refer to Anchor Forces Calculation section above , shear key's shear reaction takes moment to base plate center line and this moment will cause additional tensile force on anchors

Single anchor tension from moment caused by shear key reaction force	T_{sk} = from Anchor Forces Calculation above	= 0.27	[kips]	in tension
Single anchor tensile force	$T = T_{sk} - P / n_t$	= **1.08**	[kips]	
Strength reduction factor	$\phi_{tc} = 0.75$ supplementary reinft present			ACI 318-19 17.5.3(b)

	$\phi_{tc} N_{sb}$ =	0.75 x 111.16	= 83.37	[kips]	
Seismic design strength reduction	=	x 1.0 not applicable	= **83.37**	[kips]	ACI 318-19 17.10.5.4(d)
	ratio =	**0.01**	> 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	= 8	
Anchor rod tensile resistance	$n_t \phi N_{sa}$ =	8 x 108.75	= 870.00	[kips]
Anchor concrete breakout resistance	ϕN_{cbg} =	from anchor conc breakout calc above	= 104.79	[kips]
Anchor pullout resistance	$n_t \phi N_{pm}$ =	8 x 129.50	= 1036.0	[kips]
Anchor side blowout resistance	ϕN_{sbg} =	from anchor side blowout calc above	= 1150.2	[kips]
Anchor group governing tensile resistance	ϕN_n =	minimum of above values	= **104.79**	[kips]

Anchor Shear Resistance and Tension - Shear Interaction **N/A**

There is no shear load from user load input or shear key is used and all shear is taken by shear key, so
Anchor Shear Resistance and Tension - Shear Interaction checks are Not Applicable

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

There is no shear load applied to anchor/anchor group, so Seismic Shear check is NOT required

Shear Key - Load Case 1 V_y P_t =6.5 kip V_y =7.0 kip Code=ACI 318-19

Result Summary geometries & weld limitations = **PASS** limit states max ratio = **0.19** **PASS**

Shear Key Dimensions Check **PASS**

Shear Lug Dimensions

Shear lug embedment depth and grout thickness	d_{sl} = 3.000 [in]	g = 1.500 [in]	
Shear lug embedment depth excluding grout	$h_{sl} = d_{sl} - g$		= 1.500 [in]
Anchor out-out spacing in shear direction	s_1 = from user input		= 39.727 [in]

c_{sl} is anchor center to shear lug center distance in shear direction, when there are more than one row of anchor, take the average distance of multiple rows of anchor

Anchor center to shear lug center distance in shear direction	$c_{sl} = 0.25\ D_{bc}$	= 10.750 [in]
Anchor embedment depth	h_{ef} = from user input	= 31.500 [in]

ACI 318-19 Fig. R17.11.1.1a

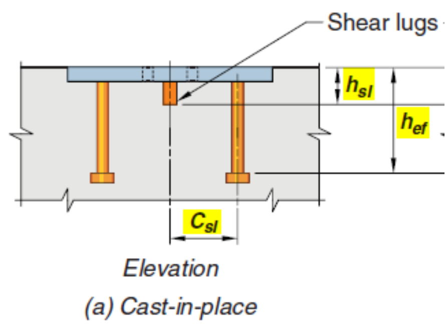
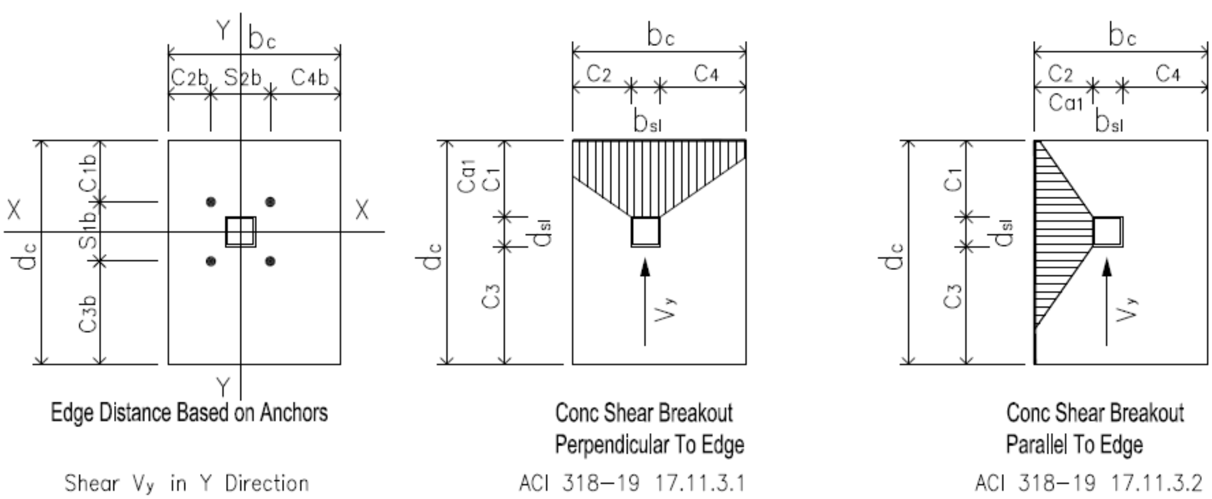


Fig. R17.11.1.1a—Examples of attachments wit

Check if $h_{ef} / h_{sl} \geq 2.5$	$= h_{ef} / h_{sl}$	= 21.00 ≥ 2.5	OK	ACI 318-19 17.11.1.1.8 (a)
Check if $h_{ef} / c_{sl} \geq 2.5$	$= h_{ef} / c_{sl}$	= 2.93 ≥ 2.5	OK	17.11.1.1.8 (b)

Shear Key Conc Breakout Strength ratio = 7.0 / 36.1 = **0.19** **PASS**

Conc. Shear Breakout Resistance - Perpendicular To Edge



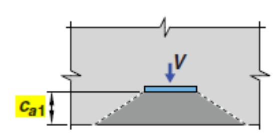
Anchor Dimensions

Anchor edge distance	c_{1b} = 9.137 [in]	c_{2b} = 9.137 [in]
	c_{3b} = 9.137 [in]	c_{4b} = 9.137 [in]
Anchor out-out spacing	s_{1b} = 39.727 [in]	s_{2b} = 39.727 [in]

Shear Lug Dimensions

Shear lug width & depth	b_{sl} = 32.000 [in]	d_{sl} = 0.750 [in]	
Shear lug embed depth	d = from user input		= 3.000 [in]

ACI 318-19 Fig. R17.11.3.1



Grout thickness g = from user input = 1.500 [in]

Shear lug effective height $h_{ef,sl} = d - g$ = 1.500 [in]

Shear lug effective area $A_{ef,sl} = b_{sl} \times h_{ef,sl}$ = 48.00 [in^2]

Shear lug edge distance c_1 = 28.626 [in] c_2 = 13.001 [in]

c_3 = 28.626 [in] c_4 = 13.001 [in]

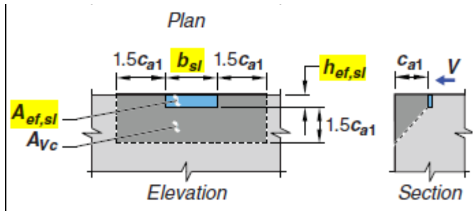


Fig. R17.11.3.1—Example of A_{Vc} for a shear lug near an edge.

Shear lug edge distance c_1 = from user input = 28.626 [in]

Limiting c_{a1} when anchors are influenced by 3 or more edges = Yes ACI 318-19 17.7.2.1.2

Shear lug edge distance - adjusted $c_{a1} = c_1$ needs to be adjusted = **26.667** [in]

b_{sl} = 32.000 [in] $1.5c_1$ = 40.000 [in]

$A_{Vc1} = [\min(c_2, 1.5c_1) + b_{sl} + \min(c_4, 1.5c_1)] \times \min(h_{ef,sl} + 1.5c_{a1}, h_a) - A_{ef,sl}$ = 2272.0 [in^2] ACI 318-19 17.11.3.1.1

Projected area of single anchor failure surface $A_{Vco} = 4.5\, c_{a1}^2$ = 3200.0 [in^2] ACI 318-19 17.7.2.1.3

Projected area of anchor group failure surface $A_{Vc} = \min(A_{Vc1}, 1 \times A_{Vco})$ = 2272.0 [in^2] ACI 318-19 17.7.2.1.1

Shear lug edge distance c_{a1} = 26.667 [in]

Conc strength & lightweight factor f_c = 4.4 [ksi] λ = 1.0

Shear lug shear breakout strength $V_b = 9\,\lambda \sqrt{f_c}\, c_{a1}^{1.5}$ = 81.74 [kips] ACI 318-19 17.11.3.1 ACI 318-19 17.7.2.2.1b

---

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 $\Psi_{ed,V} = \min[\,(0.7 + 0.3\, c_2 / 1.5c_1),\, 1.0\,]$ = 0.798 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

Shear lug edge distance & conc thickness c_{a1} = 26.667 [in] h_a = 40.000 [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

Shear force in demand V_u = from user input = **7.00** [kips]

Strength reduction factor ϕ_{vc} = 0.65 ACI 318-19 17.11.1.1.6

Concrete breakout resistance $V_{cb} = \phi_{vc} \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_b$ = **36.10** [kips] ACI 318-19 17.7.2.1b

ratio = **0.19** > V_u OK

Conc. Shear Breakout Resistance - Parallel To Edge

Shear lug edge distance $c_{a1} = \min(c_2, c_4)$ = 13.001 [in]

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

Shear lug edge distance - adjusted $c_{a1} = c_1$ needs NOT to be adjusted = **13.001** [in]

c_1 = 28.626 [in] c_3 = 28.626 [in]

d_{sl} = 0.750 [in] $1.5c_{a1}$ = 19.501 [in]

$A_{Vc1} = [\min(c_1, 1.5c_{a1}) + d_{sl} + \min(c_3, 1.5c_{a1})] \times \min(h_{ef,sl} + 1.5c_{a1}, h_a) - A_{ef,sl}$ = 833.69 [in^2] ACI 318-19 17.11.3.1.1

Projected area of single anchor failure surface $A_{Vco} = 4.5\, c_{a1}^2$ = 760.56 [in^2] ACI 318-19 17.7.2.1.3

Projected area of anchor group failure surface $A_{Vc} = \min(A_{Vc1}, 1 \times A_{Vco})$ = 760.56 [in^2] ACI 318-19 17.7.2.1.1

Shear lug edge distance c_{a1} = 13.001 [in]

Shear lug edge distance	c_{a1} = 13.001 [in]			
Conc strength & lightweight factor	f_c = 4.4 [ksi]	λ = 1.0		
Shear lug shear breakout strength	$V_b = 9\ \lambda \sqrt{f_c}\ c_{a1}^{1.5}$	= 27.82	[kips]	ACI 318-19 17.11.3.1 ACI 318-19 17.7.2.2.1b
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 17.7.2.1 (c)
Conc cracking modification factor	$\Psi_{c,V}$ =	= 1.20		ACI 318-19 17.7.2.5.1
Shear lug edge distance & conc thickness	c_{a1} = 13.001 [in]	h_a = 40.000	[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
Shear force in demand	V_u = from user input	= **7.00**	[kips]	
Strength reduction factor	ϕ_{vc} = 0.65			ACI 318-19 17.11.1.1.6

For shear parallel to an edge, V_{cb-p} shall be permitted to be twice the value of the shear perpendicular to an edge V_{cb} — ACI 318-19 17.7.2.1 (c)

Concrete breakout resistance	$V_{cb-p} = 2x\ \phi_{vc} \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_b$	= **43.41**	[kips]	ACI 318-19 17.11.3.2
	ratio = **0.16**	> V_u	OK	

Shear Key Conc Bearing Strength ratio = 7.00 / 229.42 = **0.03** **PASS**

Shear lug sect - cross plate	w = 32.000 [in]	t_p = 0.750	[in]	
Shear lug embed depth	d = from user input	= 3.000	[in]	
Grout thickness	g = from user input	= 1.500	[in]	
Conc compressive strength	f_c = from user input	= 4.4	[ksi]	

Shear Lug Bearing Factor

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
No of anchors in tension	n_t =	= 8		
Tensile load in anchor group	P_u = from user load input	= -6.54	[kips]	
Shear lug bearing factor	$\Psi_{b,sl} = 1 + \frac{P_u}{n_t N_{sa}} \leq 1.0$	= 0.99		ACI 318-19 17.11.2.2.1a

Shear in Y Direction

Shear in Y direction	V_y = from user input	= **7.00**	[kips]	
Shear lug bearing area	$A_{ef,sl}$ = w (d - g)	= 48.00	[in^2]	
Shear lug bearing strength	$V_{b,sl} = 1.7\ f_c A_{ef,sl} \Psi_{b,sl}$	= 352.96	[kips]	ACI 318-19 17.11.2.1
Resistance factor-LRFD	ϕ = 0.65			ACI 318-19 17.11.1.1.4
	$\phi V_{b,sl}$ =	= **229.42**	[kips]	
	ratio = **0.03**	> V_y	OK	

Shear Key Flexural Strength		ratio = 1.31 / 736.88	= **0.00**	**PASS**
Shear key sect - cross plate	w = 32.000 [in]	t_p = 0.750	[in]	
Cross plate section modulus	$Z_s = (t_p \times w^2) / 4 + (w \times t_p^2) / 4$	= 196.50	[in^3]	
Cross plate section modulus	$Z_w = (t_p \times w^2) / 4 + (w \times t_p^2) / 4$	= 196.50	[in^3]	
	Z_s = 196.50 [in^3]	Z_w = 196.50	[in^3]	
Shear key steel yield strength	F_y = 50.0 [ksi]			
Shear key embed depth	d = from user input	= 3.000	[in]	
Grout thickness	g = from user input	= 1.500	[in]	
Shear Key Flexure in Y Direction				
Shear in Y direction	V_y = from user input	= 7.00	[kips]	
Moment caused by shear	$M_s = V_y [g + 0.5(d - g)]$	= **1.31**	[kip-ft]	
Shear key flexural capacity	$M_n = F_y Z_s$	= 818.75	[kip-ft]	
Resistance factor-LRFD	ϕ = 0.90			
	ϕM_n =	= **736.88**	[kip-ft]	
	ratio = **0.00**	> M_s	OK	

Shear Key Shear Strength		ratio = 7.00 / 648.00	= **0.01**	**PASS**
Shear key sect - cross plate	w = 32.000 [in]	t_p = 0.750	[in]	
Shear key embed depth	d = from user input	= 3.000	[in]	
Grout thickness	g = from user input	= 1.500	[in]	
Shear key steel yield strength	F_y = 50.0 [ksi]			
Shear in Y Direction				
Shear in Y direction	V_y = from user input	= **7.00**	[kips]	
Plate shear area	$A_w = w \times t_p$	= 24.00	[in^2]	
Shear key shear strength	$V_n = 0.6 F_y A_w$	= 720.00	[kips]	
Resistance factor-LRFD	ϕ = 0.90			
	ϕV_n =	= **648.00**	[kips]	
	ratio = **0.01**	> V_y	OK	

Shear Key to Base PL Fillet Weld ratio = 7.00 / 446.04 = **0.02** **PASS**

Shear key depth & grout thickness	d_{sk} = 3.000 [in]	g = 1.500 [in]	
Shear key reaction to base plate bottom surface distance	$e_x = 0.5(d_{sk} - g) + g$	= **2.250** [in]	

Weld Group Forces

	Shear V = 7.00 [kips]	Axial P = 0.00 [kips]	
Shear force to base plate ecc	e_x =	= 2.250 [in]	
Weld length	L =	= 32.000 [in]	
Shear force in demand	V_u = from user input	= **7.00** [kips]	

Fillet Weld Strength Calc

Fillet weld leg size	w = $^5/_{16}$ [in]	load angle θ = 0.0 [°]	
Electrode strength	F_{EXX} = 70.0 [ksi]	strength coeff C_1 = 1.00	AISC 15th Table 8-3
Number of weld line	n = 2 for double fillet		
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.00	AISC 15th Page 8-9
Fillet weld shear strength	$r_w = 0.6\ (C_1 \times 70\ \text{ksi})\ 0.707\ w\ n\ C_2$	= 18.56 [kip/in]	AISC 15th Eq 8-1
Base metal - shear plate	thickness t = 0.750 [in]	tensile F_u = 65.0 [ksi]	
Base metal - shear plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked			AISC 15th J2.4
Base metal shear rupture	$r_b = 0.6\ F_u\ t$	= 29.25 [kip/in]	AISC 15th Eq J4-4
Weld stress reduction factor due to less base metal strength	$C_3 = r_b / r_w$ when $r_b < r_w$	= 1.000	
Table 8-4 Coefficient C for Eccentrically Loaded Weld Group			AISC 15th Table 8-4
	$a = e_x / L$	= 0.07	
	C = C value in Table 8-4 when k=0	= 3.717	
Weld coefficients	C = 3.717	C_1 = 1.000	
		C_3 = 1.000	
Weld size & length	D = 5.000 [1/16]	L = 32.000 [in]	
Weld strength	$R_n = C\ C_1\ C_3\ D\ L$	= 594.73 [kips]	AISC 15th Table 8-4
Resistance factor-LRFD	ϕ = 0.75		
	ϕR_n =	= **446.04** [kips]	
	ratio = **0.02**	> V_u OK	

Vertical Vessel Base Plate P_t =6.5 kip M_x =0.0 kip-ft Code=ACI 318-19

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.13** **PASS**

Base Plate Thickness Check ratio = 0.191 / 1.500 = **0.13** **PASS**

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

Circular anchor bolt circle dia & column OD	D_{bc} = 43.000 [in]	OD = 36.000	[in]	
Max anchors tensile force	$T_u = T_1$	= 0.82	[kips]	
T_u to CHS wall moment lever arm	$x = 0.5\ (D_{bc} - OD)$	= 3.500	[in]	
Base plate width & strength	B = 7.000 [in]	F_y = 50.0	[ksi]	
Base plate thickness	t_p = from user input	= **1.500**	[in]	
	$t_{req\text{-}t} = 2.11 \left(\frac{T_u\, x}{B\, F_y} \right)^{0.5}$	= **0.191**	[in]	Eq 3.4.7a
	ratio = **0.13**	< t_p	OK	