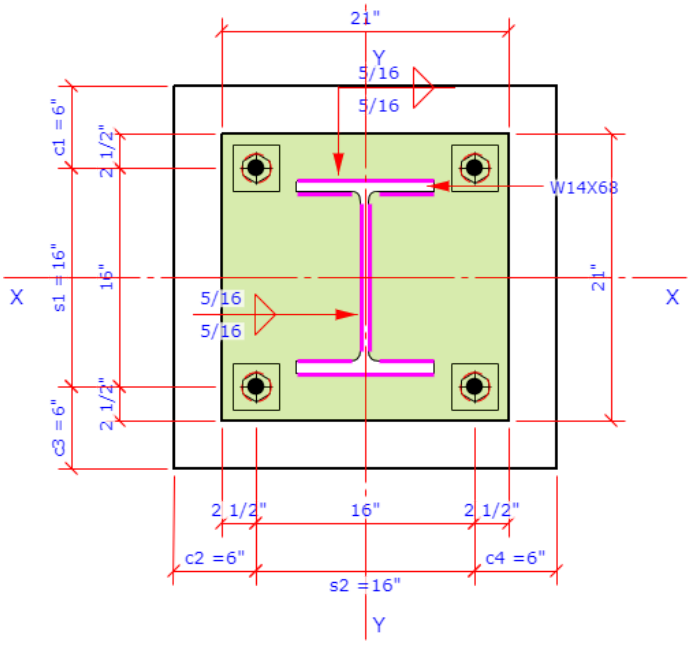
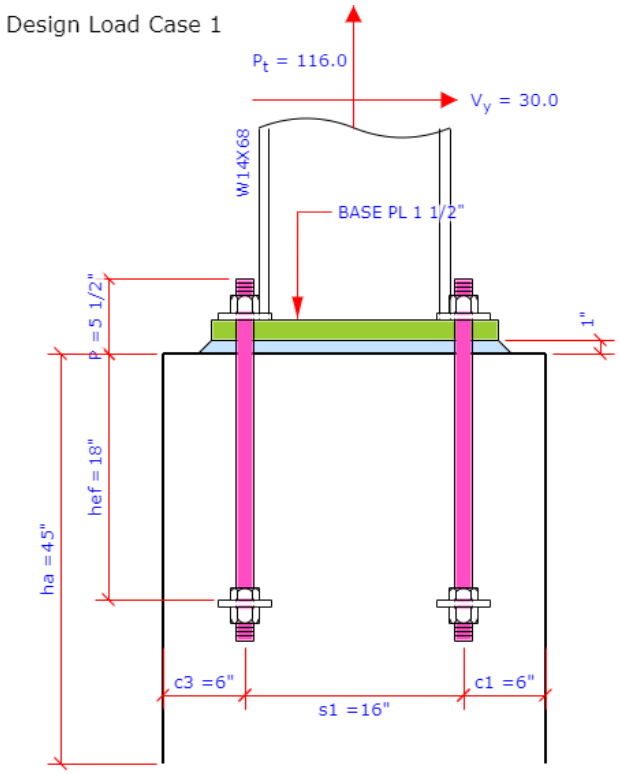
Result Summary - Overall	Anchorage Design	Code=ACI 318-19

Result Summary - Overall	geometries & weld limitations = **PASS**		limit states max ratio = **0.98**	**PASS**
Anchor Bolt - LC 1	**P + V_y + M_x**	geometries & weld limitations = **PASS**	limit states max ratio = **0.80**	**PASS**
Base Plate - LC 1	**P + M_x**	geometries & weld limitations = **PASS**	limit states max ratio = **0.98**	**PASS**

Sketch	Anchorage Design	Code=ACI 318-19





Anchor Forces Calculation

Anchor Tensile Force Calculation

User Input

Anchor edge distance

c_{1u} = 6.000 [in] c_{2u} = 6.000 [in]

c_{3u} = 6.000 [in] c_{4u} = 6.000 [in]

Anchor out-out spacing

s_{1u} = 16.000 [in] s_{2u} = 16.000 [in]

Anchor embedment depth

h_{ef} = 18.000 [in]

Design Load - Load Case 1

Axial force

Axial P = -116.00 [kips] in tension

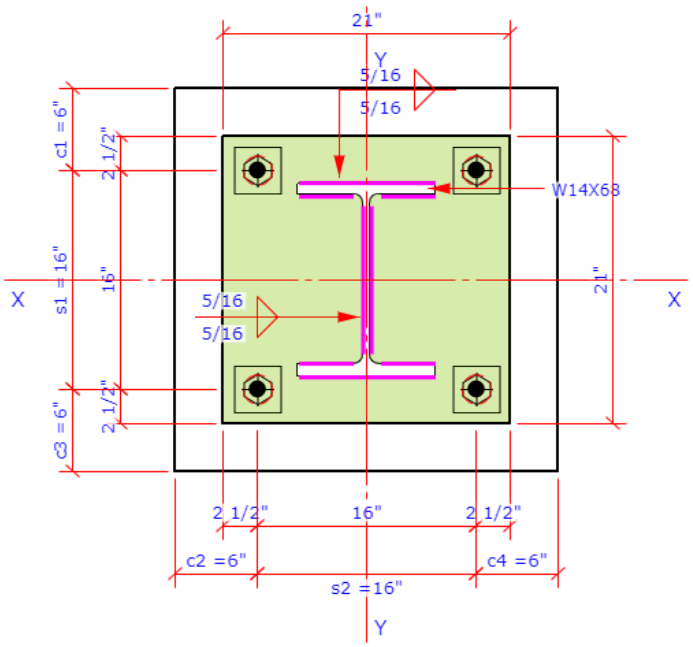
Shear forces

V_y = 30.00 [kips] V_x = 0.00 [kips]

Moment forces

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

Anchor Layout Plan



Anchor Bolt - Load Case 1 **P + V_y + M_x** P_t=116.0 kip V_y=30.0 kip M_x=0.0 kip-ft Code=ACI 318-19

Result Summary geometries & weld limitations = **PASS** limit states max ratio = **0.80** **PASS**

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 Gr55	d_a = 1 1/4	[in]	

Min Anchor Spacing

Min anchor spacing required	$s_{min} = 6 \times d_a$	= **7.500**	[in]	ACI 318-19 Table 17.9.2(a)
Anchor bolt pattern	= from user input	= 4B		
Min anchor spacing	s = from user input	= **16.000**	[in]	
		≥ 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		Post-installed expansion and undercut anchors	Post-installed screw anchors
	Not torqued	Torqued		
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 = 29.0 / 54.5 = **0.53** **PASS**

Anchor rod effective section area	A_{se} = 0.97 [in^2]	f_{uta} = 75.0	[ksi]	
Anchor rod steel strength in tension	$N_{sa} = A_{se} f_{uta}$	= 72.68	[kips]	ACI 318-19 17.6.1.2

Max Single Anchor Tensile Force

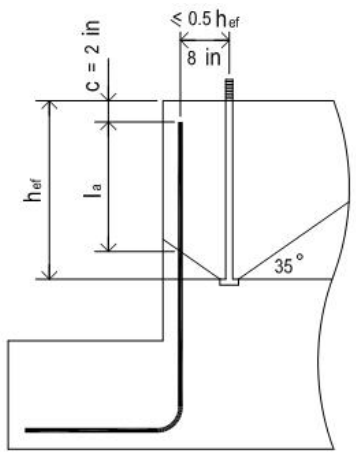
Anchor group axial tensile force	P = from user load input	= -116.00	[kips]	in tension
No of anchors in the group	n_t =	= 4		
Single anchor tensile force	$T = P / n_t$	= **29.00**	[kips]	
Strength reduction factor	ϕ_{ts} = 0.75			ACI 318-19 17.5.3(a)
	$\phi_{ts} N_{sa}$ = 0.75 x 72.68	= **54.51**	[kips]	
	ratio = **0.53**	> T	OK	

Anchor Reinforcement Tensile Breakout Resistance

ratio = 116.0 / 273.3 = **0.42** **PASS**

Concrete & rebar strength	f_c = 4.5 [ksi]	f_y = 60.0	[ksi]	
Ver rebar no & area	bar no = #8	A_s = 0.79	[in^2]	
	d_b = 1"			
	Ψ_e = 1.0	Ψ_r = 1.0		ACI 318-19 Table 25.4.3.2
	Ψ_o = 1.0	Ψ_c = 0.90		
Hook rebar development length	$l_{dh} = \max\left(\frac{f_y \Psi_e \Psi_r \Psi_o \Psi_c}{55 \lambda \sqrt{f_c}} d_b^{1.5}, 8d_b, 6"\right)$	= **14.636**	[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	= 18.000	[in]	
Avg ver. bar center to anchor rod center distance	d_{ar} = from user input	= 2.740	[in]	
Min rebar development length required	$l_{min} = \max(8d_b, 6")$	ACI 318-19 25.4.3.1 = **8.000**	[in]	
Actual rebar development length	$l_a = h_{ef}$ - top cover (2") - d_{ar} tan35	= **14.081**	[in]	
	ratio = **0.57**	> l_{min}	OK	ACI 318-19 25.4.3.1

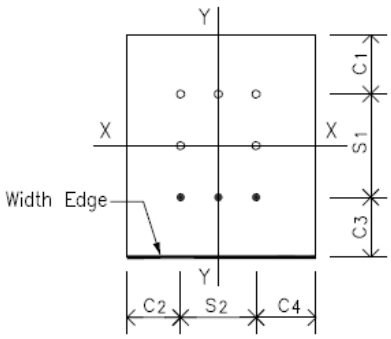
Anchor group tensile load	N_u = from user load input	= **116.00**	[kips]	
No of ver rebar effective to resist anchor tension	n_v = from user input	= 8.0		
Rebar resistance factor	ϕ_s = 0.75			ACI 318-19 17.5.3

when $l_a < l_{dh}$, anchor reinforcement rebar won't develope full yield strength f_y and N_r need to apply a reduction factor l_a / l_{dh}

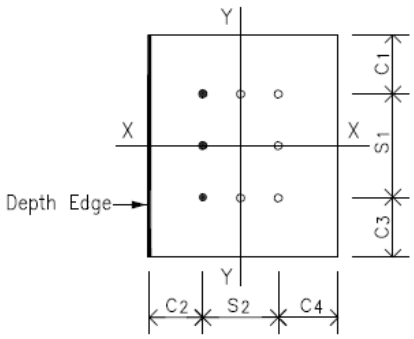
Anchor reinft breakout resistance	$\phi N_n = \phi_s n_v A_s f_y \frac{l_a}{l_{dh}}$	= **273.26**	[kips]	ACI 318-19 17.5.2.1 (a)
	ratio = **0.42**	> N_u	OK	

Anchor Pullout Resistance					ratio = 29.0 / 56.4	= **0.51**	**PASS**
Anchor head net bearing area & conc strength	A_{brg} = 2.24	[in^2]	f_c = 4.5	[ksi]			
Single bolt pullout resistance	$N_p = 8\, A_{brg}\, f_c$		= 80.53	[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		= -116.00	[kips]	in tension		
No of anchors in the group	n_t =		= 4				
Single anchor tensile force	$T = P / n_t$		= **29.00**	[kips]			
Strength reduction factor	ϕ_{tc} = 0.70	pullout strength is always Condition B			ACI 318-19 17.5.3(c)		
	$\phi_{tc} N_{pn} = \phi_{tc} \Psi_{cP} N_p$		= **56.37**	[kips]			
Seismic design strength reduction	= x 1.0	not applicable	= **56.37**	[kips]	ACI 318-19 17.10.5.4(c)		
	ratio = **0.51**		> T	OK			

Anchor Side Blowout Resistance					ratio = 29.0 / 36.1	= **0.80**	**PASS**
Anchor Inputs							
Anchor edge distance	c_1 = 6.000	[in]	c_2 = 6.000	[in]			
	c_3 = 6.000	[in]	c_4 = 6.000	[in]			
Anchor out-out spacing	s_1 = 16.000	[in]	s_2 = 16.000	[in]			



Side Blowout Width Edge



Side Blowout Depth Edge

Side Edges Along X-X Axis - Width Edges

Anchor edge distance in Y direction	$c_{a1} = \min(c_1, c_3)$	= 6.000	[in]		
Anchor embedment depth	h_{ef} = from user input	= 18.000	[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	= 16.000	[in]		
Anchor number along X direction	n_w = from user input	= 2			
Anchor head net bearing area & conc strength	A_{brg} = 2.24 [in^2]	f_c = 4.5	[ksi]		
Lightweight conc modification factor	λ = 1.0			ACI 318-19 17.2.4.1	
Single anchor side blowout capacity	$N_{sb} = 160\, c_{a1} \sqrt{A_{brg}}\, \lambda \sqrt{f_c}$	= 96.32	[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	$s_h = s_2 / (n_w - 1)$	= 16.000	[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	$= 1 + \frac{s_2}{6c_{a1}}$	= 1.44		ACI 318-19 17.6.4.2
Group anchor side blowout capacity	$N_{sbg} = (1 + \frac{s_2}{6c_{a1}})\ N_{sb}$	= **139.13**	[kips]	

Max Single Anchor Tensile Force

Anchor group axial tensile force	P = from user load input	= -116.00	[kips]	in tension
No of anchors in the group	n_t =	= 4		
Single anchor tensile force	$T = P / n_t$	= **29.00**	[kips]	
No of anchors along side blowout edge	n_{bw} = from user input	= 2		
Tensile force - anchors along potential blowout edge	$T_w = n_{bw} \times T$	= **58.00**	[kips]	
Strength reduction factor	ϕ_{tc} = 0.75 supplementary reinft present			ACI 318-19 17.5.3(b)
	$\phi_{tc} N_{sbg}$ = 0.75 x 139.13	= 104.35	[kips]	
Seismic design strength reduction	= x 1.0 not applicable	= **104.35**	[kips]	ACI 318-19 17.10.5.4(d)
	ratio = **0.56**	> 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	$= 104.35 \frac{n_t}{n_{bw}}$	= **208.69**	[kips]	

Side Edges Along Y-Y Axis - Depth Edges

Anchor edge distance in X direction	$c_{a2} = \min(c_2, c_4)$	= 6.000	[in]	
Anchor embedment depth	h_{ef} = from user input	= 18.000	[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 out-out distance edges along X direction	s_1 = from user input	= 16.000	[in]	
Anchor number along X direction	n_d = from user input	= 2		
Anchor head net bearing area & conc strength	A_{brg} = 2.24 [in^2]	f_c = 4.5	[ksi]	
Lightweight conc modification factor	λ = 1.0			ACI 318-19 17.2.4.1
Single anchor side blowout capacity	$N_{sb} = 160\ c_{a2} \sqrt{A_{brg}}\ \lambda \sqrt{f_c}$	= 96.32	[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	$s_b = s_1 / (n_d - 1)$	= 16.000	[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	$= 1 + \frac{s_1}{6c_{a2}}$	= 1.44		ACI 318-19 17.6.4.2
Group anchor side blowout capacity	$N_{sbg} = (1 + \frac{s_1}{6c_{a2}})\ N_{sb}$	= **139.13**	[kips]	

Max Single Anchor Tensile Force

Anchor group axial tensile force	P = from user load input		= -116.00	[kips]	in tension
No of anchors in the group	n_t =		= 4		
Single anchor tensile force	T = P / n_t		= **29.00**	[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} x T		= **58.00**	[kips]	
Strength reduction factor	ϕ_{tc} = 0.75	supplementary reinft present			ACI 318-19 17.5.3(b)
	$\phi_{tc} N_{sbg}$ = 0.75 x 139.13		= 104.35	[kips]	
Seismic design strength reduction	= x 1.0	not applicable	= **104.35**	[kips]	ACI 318-19 17.10.5.4(d)
	ratio = **0.56**		> 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} = 2		
Group anchor tensile side blowout capacity	= 104.35 $\frac{n_t}{n_{bd}}$		= **208.69**	[kips]	

Corner Single Anchor Side Blowout

Check on 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)		= 6.000	[in]	
	c_{a2} = min (c_2, c_4)		= 6.000	[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 x N_{sb}		= 48.16	[kips]	

Max Single Anchor Tensile Force

Anchor group axial tensile force	P = from user load input		= -116.00	[kips]	in tension
No of anchors in the group	n_t =		= 4		
Single anchor tensile force	T = P / n_t		= **29.00**	[kips]	
Strength reduction factor	ϕ_{tc} = 0.75	supplementary reinft present			ACI 318-19 17.5.3(b)
	$\phi_{tc} N_{sb}$ = 0.75 x 48.16		= 36.12	[kips]	
Seismic design strength reduction	= x 1.0	not applicable	= **36.12**	[kips]	ACI 318-19 17.10.5.4(d)
	ratio = **0.80**		> 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 54.51	= 218.03	[kips]	
Anchor concrete breakout resistance	ϕN_n = from anchor reinft tensile breakout resistance calc above	= 273.26	[kips]	
Anchor pullout resistance	$n_t \phi N_{pm}$ = 4 x 56.37	= 225.49	[kips]	
Anchor side blowout resistance	ϕN_{sbg} = from anchor side blowout calc above	= 208.69	[kips]	
Anchor group governing tensile resistance	ϕN_n = minimum of above values	= **208.69**	[kips]	

Anchor Rod Shear Resistance

ratio = 30.0 / 90.7 = **0.33** **PASS**

Shear load on anchor group	V_u = from user load input	= **30.00**	[kips]	
Anchor rod effective section area	A_{se} = 0.97 [in^2]	f_{uta} = 75.0	[ksi]	
No of anchors in the group resisting shear	n_s = from user input	= 4		
Anchor rod steel strength in tension	$V_{sa} = n_s 0.6 A_{se} f_{uta}$	= 174.42	[kips]	ACI 318-19 17.7.1.2b
Strength reduction factor	ϕ_{vs} = 0.65			ACI 318-19 17.5.3(a)
	$\phi_{vs} V_{sa}$ =	= 113.37	[kips]	
Reduction due to built-up grout pad	= x 0.80 applicable	= **90.70**	[kips]	ACI 318-19 17.7.1.2.1
	ratio = **0.33**	> V_u	OK	

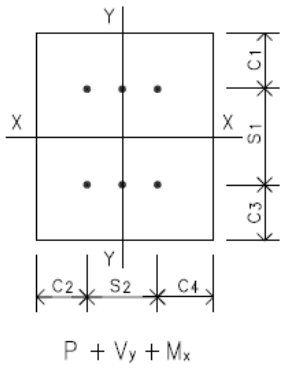
Concrete Shear Breakout Resistance

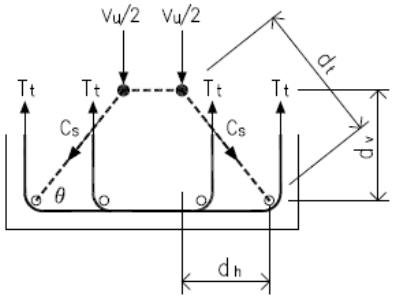
ratio = 21.2 / 45.2 = **0.47** **PASS**

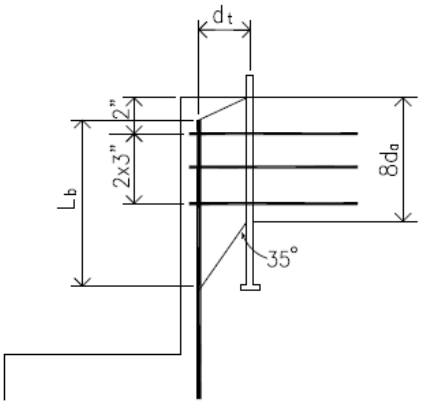
Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinforcement

ACI 318-19 Table 21.2.1 (g)

STM strength reduction factor	ϕ_{st} = 0.75				
Anchor edge distance	c_1 = 6.000	[in]	c_2 = 6.000	[in]	
	c_3 = 6.000	[in]	c_4 = 6.000	[in]	
Anchor dia & embedment depth	d_a = 1.250	[in]	h_{ef} = 18.000	[in]	
Shear load on anchor group	V_u = from user load input		= **30.00**	[kips]	







Refer to sketch above for the terms and notations used below

Strut-and-Tie model geometry	$d_v = c_1$ - 2.75" (70mm)	= 3.250	[in]		
	$d_h = \min(c_2, c_4)$ - 2.75" (70mm)	= 3.250	[in]		
	$\theta = \tan^{-1}(d_v / d_h)$	= 45.0	[°]		
	$d_t = \sqrt{d_v^2 + d_h^2}$	= 4.596	[in]		
Strut compression force	$C_s = \frac{0.5\, V_u}{\sin\theta}$	= **21.21**	[kips]		

Strut Bearing Strength

No of anchors in the group resisting shear	n_s = from user input	= 4			
Concrete & rebar strength	f_c = 4.5 [ksi]	f_y = 60.0	[ksi]		
Strut compressive strength	$f_{ce} = 0.85\, f_c$	= 3.8	[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	$C_r = n_s \times \phi_{st} \times f_{ce} \times A_{brg}$	= 143.44	[kips]		
	ratio = **0.21**	> V_u	OK		

Bearing of Ver Reinforcement Rebar

Anchor & ver rebar dia	d_a = 1.250 [in]	d_b = 1.000	[in]		
Anchor bearing area	$A_{brg} = [\, l_e + 1.5d_t - 0.5(d_a + d_b)\,] \times d_b$	= 15.77	[in^2]		
Anchor bearing resistance	$C_r = \phi_{st} \times f_{ce} \times A_{brg}$	= 45.24	[kips]		
	ratio = **0.47**	> 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)	= 12			
Rebar resistance factor	ϕ_s = 0.75				17.5.3 (a)
Hor rebar area & strength	A_s = 0.31 [in^2]	f_{yh} = 60.0	[ksi]		
Single tie bar tension resistance	$T_r = \phi_s f_{yh} A_s$	= 13.95	[kips]		
Total tie bar tension resistance	$\phi_s V_{nb} = n \times T_r$	= **167.40**	[kips]	17.5.2.1 (b)	
	ratio = **0.18**	> 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} = 18.000	[in]		
	$12d_a = 12 \times 1.250$	= **15.000**	[in]		
Anchor embedment depth	h_{ef} = from user input	= **18.000**	[in]		
	ratio = **0.83**	> $12d_a$	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	= 90.70	[kips]
Anchor reinft shear breakout resistance	ϕV_{nb} = from anchor reinft shear breakout calc	= 167.40	[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	= **90.70**	[kips]

Anchor Tension and Shear Interaction

ratio = 0.89 / 1.20 = **0.74** **PASS**

Anchor group tensile load	N_u = from user load input	= 116.00	[kips]	
Anchor group shear load	V_u = from user load input	= 30.00	[kips]	
Anchor group governing tensile resistance	ϕN_n = from calc in above section	= 208.69	[kips]	
Anchor group governing shear resistance	ϕV_n = from calc in above section	= 90.70	[kips]	
Consider anchor tension-shear interaction	check if $\frac{N_u}{\phi N_n} > 0.2$ and $\frac{V_u}{\phi V_n} > 0.2$	= Yes		ACI 318-19 17.8.3
	anchor tension-shear interaction shall be considered			
	$= \frac{N_u}{\phi N_n} + \frac{V_u}{\phi V_n}$	= **0.89**		ACI 318-19 17.8.3
	ratio = **0.74**	< 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_t=116.0 kip M_x=0.0 kip-ft Code=ACI 318-19

Result Summary geometries & weld limitations = **PASS** limit states max ratio = **0.98** **PASS**

Base Plate Thickness Check

ratio = 1.487 / 1.519 = **0.98** **PASS**

Column sect W14X68	d = 14.000	[in]	b_f = 10.000	[in]
Base plate width & depth	B = 21.000	[in]	N = 21.000	[in]
Pedestal width & depth	b_c = 28.000	[in]	d_c = 28.000	[in]

Base plate area $A_1 = B \times N$ = 441.00 [in^2]

Pedestal area $A_2 = b_c \times d_c$ = 784.00 [in^2]

ACI 318-19 Table 14.5.6.1

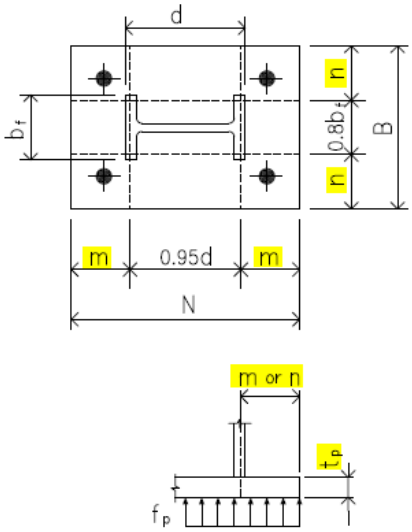
$k = \min (\sqrt{A_2 / A_1}, 2)$ = 1.333

AISC Design Guide 1 - 3.1.2 on Page 15

Base plate cantilever dimension $m = (N - 0.95\, d) / 2$ = 3.850 [in]

$n = (B - 0.8\, b_f) / 2$ = 6.500 [in]

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



Factored tension force T_u = from user input = **116.00** [kips]

No of anchors in tension n_t = anchor bolt pattern - 4B = 4

Anchor rod effective section area A_{se} = 0.97 [in^2] f_{uta} = 75.0 [ksi]

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

Anchor rod tensile resistance $T_r = \phi_{ts}\, n_t\, A_{se}\, f_{uta}$ = **218.03** [kips] ACI 318-19 17.6.1.2

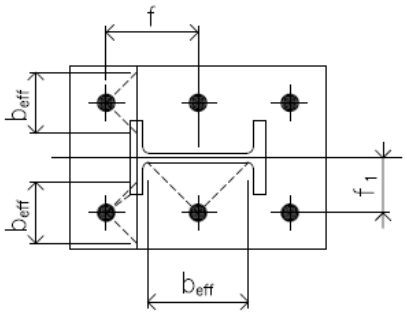
ratio = **0.53** > T_u OK

Anchor out-out spacing	s_1 = 16.000	[in]	s_2 = 16.000	[in]
Column sect W14X68	t_w = 0.415	[in]		

Anchor to column center distance $f = s_1 / 2$ = 8.000 [in]

Anchor to column web center distance $f_1 = s_2 / 2$ = 8.000 [in]

Factored tensile force in single anchor $T_b = T_u / n_t$ = 29.00 [kips]



BASE PLATE SUBJECT TO TENSILE LOAD

Bending to Column Flange X-X

AISC Design Guide 1

Moment lever arm $a = f - 0.5\, d + 0.5\, t_f$ = 1.360 [in] Eq 3.4.6 on Page 26

No of anchors in tension n_1 = from user input bolt pattern = 2

Moment to column flange $M_u = n_1 T_b \times a$ = 6.57 [kip-ft]

Base plate width B = from user input = 21.000 [in]

Base plate strength & strength reduction factor F_y = 36.0 [ksi] ϕ_b = 0.90

Base plate required thickness $t_{1s} = \left(\frac{4\, M_u}{B\, \phi_b\, F_y} \right)^{0.5}$ = 0.681 [in] Eq 3.3.13a on Page 25

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

Base plate required thickness	$t_1 = t_{1s}$		= **0.681**	[in]
	ratio = **0.45**		< t_p	OK

Bending in Weak Axis Direction Y-Y

Single anchor tensile force	$T_b = T_u / n_t$		= 29.00	[kips]
Base plate width & edge	w = 21.000	[in]	e_h = 2.500	[in]
Base plate overhang	n = 6.500	[in]		
T_b lever arm & plate effective width	x_1 = 4.000	[in]	b_{e1} = 6.500	[in]
Base plate thickness & strength	t_p = 1.500	[in]	F_y = 36.0	[ksi]

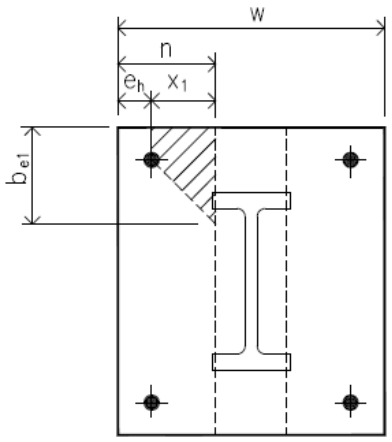


Plate moment due to anchor tensile	$M_{pl} = \frac{T_b x_1}{b_{e1}}$		= **1.487**	[kip-ft/in]
Plate moment resistance	$\phi M_n = 0.9 F_y t_p^2 /4$		= **1.519**	[kip-ft/in]
	ratio = **0.98**		> M_{pl}	OK

Column Flange Fillet Weld Limitation — **PASS**

Min Fillet Weld Size

Thinner part joined thickness	t =		= 0.720	[in]
Min fillet weld size allowed	w_{min} =		= **0.250**	[in] AISC 15th Table J2.4
Fillet weld size provided	w =		= **0.313**	[in]
			≥ w_{min}	OK

Min Fillet Weld Length

Fillet weld size provided	w =		= 0.313	[in]
Min fillet weld length allowed	$L_{min} = 4 \times w$		= **1.250**	[in] AISC 15th J2.2b
Min fillet weld length	$L = 0.5\, b_{fc} - k_{1c}$		= **3.937**	[in]
			≥ L_{min}	OK

Column Web Fillet Weld Limitation — **PASS**

Min Fillet Weld Size

Thinner part joined thickness	t =		= 0.415	[in]
Min fillet weld size allowed	w_{min} =		= **0.188**	[in] AISC 15th Table J2.4
Fillet weld size provided	w =		= **0.313**	[in]
			≥ w_{min}	OK

Min Fillet Weld Length

Fillet weld size provided	w =		= 0.313	[in]
Min fillet weld length allowed	$L_{min} = 4 \times w$		= **1.250**	[in] AISC 15th J2.2b
Min fillet weld length	$L = d_b - 2\, k_b$		= **10.874**	[in]
			≥ L_{min}	OK

W Shape Column Flange to Base Plate Weld					ratio = 4.67 / 20.88	= **0.22**	**PASS**
Column section W14X68	d_c = 14.000	[in]		b_{fc} = 10.000	[in]		
	t_{fc} = 0.720	[in]		k_{1c} = 1.063	[in]		
Forces on W shape flange weld	P_t = -116.00	[kips] (T)		V_x = 0.00	[kips]		
Flange fillet weld length-double fillet	$L = (b_{fc} + b_{fc} - 2k_{1c})/2$ as double fillet			= 8.937	[in]		
Column flange to base plate fillet weld size	w = from user input			= $^5/_{16}$	[in]		
W shape and flange area	A = 20.00	[in^2]		A_f = 7.20	[in^2]		
Tensile axial force taken by w shape one side flange	$P_{ft} = P_t A_f / A$			= -41.76	[kips]		
Weld tensile stress by P_{ft}	$f_t = P_{ft} / L$			= -4.67	[kip/in]		
Weld shear stress-dbl fillet by V_x	$f_v = V_x / 2L$			= 0.00	[kip/in]		
Weld stress combined - max	$f_{max} = (f_t^2 + f_v^2)^{0.5}$			= **4.673**	[kip/in]	AISC 15th	Eq 8-11
Weld stress load angle	$\theta = \tan^{-1} (f_t / f_v)$			= 90.0	[°]		
Fillet Weld Strength Calc							
Fillet weld leg size	w = $^5/_{16}$	[in]	load angle θ = 90.0		[°]		
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	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$			= 1.50		AISC 15th	Page 8-9
Fillet weld shear strength	R_{n-w} = 0.6 (C_1 x 70 ksi) 0.707 w n C_2			= 27.84	[kip/in]	AISC 15th	Eq 8-1
Base metal - column flange	thickness t = 0.720	[in]	tensile F_u = 65.0		[ksi]		
Base metal - column flange is in tension, tensile rupture as per AISC 15th Eq J4-2 is checked						AISC 15th	J2.4
Base metal tensile rupture	$R_{n-b} = F_u t$			= 46.80	[kip/in]	AISC 15th	Eq J4-2
Double fillet linear shear strength	$R_n = \min (R_{n-w}, R_{n-b})$			= **27.838**	[kip/in]	AISC 15th	Eq 9-2
Resistance factor-LRFD	ϕ = 0.75					AISC 15th	Eq 8-1
	ϕR_n =			= **20.879**	[kip/in]		
	ratio = **0.22**			> f_{max}	OK		

W Shape Column Web to Base Plate Weld ratio = 3.66 / 17.63 = **0.21** **PASS**

The moment $M_{x\text{-}x}$, $M_{y\text{-}y}$ and V_x are all taken care by W shape flange weld , so W shape web to base plate weld only takes shear in strong axis direction V_y and part of axial tensile load P_w

Forces on column web weld	P_t = 116.00 [kips] (T)	V_y = 30.00	[kips]	
Column section W14X68	d_c = 14.000 [in]	k_c = 1.563	[in]	
	t_{wc} = 0.415 [in]			
Fillet weld length - double fillet	$L = d_c - 2\,k_c$	= 10.874	[in]	
W shape and web area	A = 20.00 [in^2]	A_w = 4.51	[in^2]	
Tensile axial force taken by w shape web	$P_w = P_t A_w / A$	= 26.17	[kips]	
Column web to base plate fillet weld size	w = from user input	= $^5/_{16}$	[in]	
Weld tensile stress from axial load	$f_a = P_w / L$	= 2.41	[kip/in]	
Weld shear stress from shear load	$f_v = V_y / L$	= 2.76	[kip/in]	
Weld stress combined - max	$f_{max} = (f_a^2 + f_v^2)^{0.5}$	= **3.661**	[kip/in]	AISC 15th Eq 8-11
Weld stress load angle	$\theta = \tan^{-1} (f_a / f_v)$	= 41.1	[°]	

Fillet Weld Strength Calc

Fillet weld leg size	w = $^5/_{16}$ [in]	load angle θ = 41.1	[°]	
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	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.27		AISC 15th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w}$ = 0.6 (C_1 x 70 ksi) 0.707 w n C_2	= 23.51	[kip/in]	AISC 15th Eq 8-1

Base metal - column web	thickness t = 0.415 [in]	tensile F_u = 65.0	[ksi]	
Base metal - column web is in tension, tensile rupture as per AISC 15th Eq J4-2 is checked				AISC 15th J2.4
Base metal tensile rupture	$R_{n\text{-}b} = F_u t$	= 26.98	[kip/in]	AISC 15th Eq J4-2

Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w}, R_{n\text{-}b})$	= **23.505**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_n =	= **17.629**	[kip/in]	
	ratio = **0.21**	> f_{max}	OK	