CivilBay Concrete Anchorage Design v1.2.7 User Manual

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2.0 QUICK START

2.1 Software Installation

- After downloading the ZIP file the user can unzip the file and save it to user's computer.
- The extracted files are in two folders, one US Code folder, and another one Canadian Code folder. Each folder contains seven compiled Excel files in EXE format.
- User can double click these EXE files and open them just as normal Excel files.
- The 14-day trial will start the same date when user tries any of these compiled Excel files.
- During trial period the software provides full functions except that the user can not save the file, but the user can print the file to printer and get a hard copy of the calculation for verification.
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- All the compiled Excel files will fully function as long as they can find the license key in the same folder.
- The license key is created using the Computer ID sent by the user and it only works on that computer where the Computer ID is retrieved from.

2.3 Concrete Anchorage Design v1.2.7 Modules

- **01 US Code ACI 318-08**

02-01-01 Headed Anchor Bolt ACI 318-08.exe

→ Headed anchor bolt design using ACI 318-08 code

02-01-01 Headed Welded Stud ACI 318-08.exe

→ Headed welded stud design using ACI 318-08 code

02-02-01 Base Plate (LRFD) & Anchor Bolt (ACI 318-08) Design With Anchor Reinft - PIN.exe

→ One input to design both base plate and anchor bolt using ACI 318-08 code
In anchor bolt design Anchor Reinforcement is used to replace concrete tension/shear breakout strength.
In base plate design the column base is assumed to be PIN connection and doesn't have moment.

02-02-02 Base Plate (LRFD) & Anchor Bolt (ACI 318-08) Design No Anchor Reinft - PIN.exe

→ One input to design both base plate and anchor bolt using ACI 318-08 code
In anchor bolt design NO Anchor Reinforcement is used.
In base plate design the column base is assumed to be PIN connection and doesn't have moment.

02-02-03 Base Plate (LRFD) & Anchor Bolt (ACI 318-08) Design With Anchor Reinft - MC.exe

→ One input to design both base plate and anchor bolt using ACI 318-08 code
In anchor bolt design Anchor Reinforcement is used to replace concrete tension/shear breakout strength.
In base plate design the column base is assumed to be Moment connection and carries moment.

02-02-04 Base Plate (LRFD) & Anchor Bolt (ACI 318-08) Design No Anchor Reinft - MC.exe

→ One input to design both base plate and anchor bolt using ACI 318-08 code
In anchor bolt design NO Anchor Reinforcement is used.
In base plate design the column base is assumed to be Moment connection and carries moment.

02-03-01 Shear Key ACI 349-06.exe

→ Shear lug design using ACI 349-06 code

- **02 US Code ACI318M-08 SI Unit**

02-01-03 Headed Anchor Bolt ACI 318M-08 SI Unit.exe

→ Headed anchor bolt design using ACI 318M-08 code in SI metric unit

02-01-03 Headed Welded Stud ACI 318M-08 SI Unit.exe

→ Headed welded stud design using ACI 318M-08 code in SI metric unit

- **03 Canadian Code**

02-01-02 Headed Anchor Bolt CSA A23.3-04.exe

→ Headed anchor bolt design using CSA A23.3-04 code

02-01-02 Headed Welded Stud CSA A23.3-04.exe

→ Headed welded stud design using CSA A23.3-04 code

02-02-05 Base Plate & Anchor Bolt (CSA A23.3-04) Design With Anchor Reinft - PIN.exe

→ One input to design both base plate and anchor bolt using CSA A23.3-04 code
In anchor bolt design Anchor Reinforcement is used to replace concrete tension/shear breakout strength.
In base plate design the column base is assumed to be PIN connection and doesn't have moment.

02-02-06 Base Plate & Anchor Bolt (CSA A23.3-04) Design No Anchor Reinft - PIN.exe

→ One input to design both base plate and anchor bolt using CSA A23.3-04 code
In anchor bolt design NO Anchor Reinforcement is used.
In base plate design the column base is assumed to be PIN connection and doesn't have moment.

02-02-07 Base Plate & Anchor Bolt (CSA A23.3-04) Design With Anchor Reinft - MC.exe

→ One input to design both base plate and anchor bolt using CSA A23.3-04 code
In anchor bolt design Anchor Reinforcement is used to replace concrete tension/shear breakout strength.
In base plate design the column base is assumed to be Moment connection and carries moment.

02-02-08 Base Plate & Anchor Bolt (CSA A23.3-04) Design No Anchor Reinft - MC.exe

→ One input to design both base plate and anchor bolt using CSA A23.3-04 code
In anchor bolt design NO Anchor Reinforcement is used.
In base plate design the column base is assumed to be Moment connection and carries moment.

02-03-02 Shear Key ACI 349M-06.exe

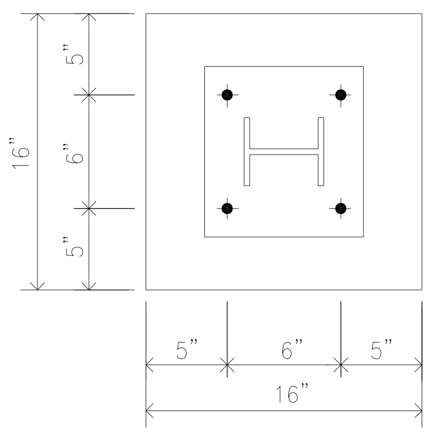
→ Shear lug design using ACI 349M-06 code (metric unit)

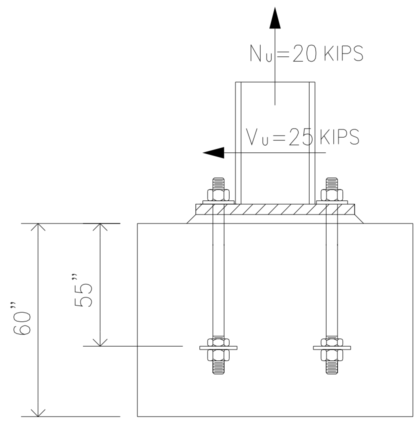
3.0 REVISION HISTORY

Date	Version	Revision Details
2011-12-30	1.2.7	▪ Bug fixed in 3 edges h_{ef} adjustment routine as per ACI 318-08 D.5.2.3 ▪ Bug fixed in 3 edges c_{a1} adjustment routine as per ACI 318-08 D.6.2.4 ▪ User now has the option of defining different rebar yield strength for vertical and horizontal anchor reinforcement ▪ User now has the option to set uncracked concrete for increased Ψ value as per ACI 318-08 D.5.2.6, D5.3.6 and D.6.2.7 ▪ Anchor bolt and anchor stud design as per ACI 318M-08 in SI metric unit is added
2011-12-22	1.2.5	Spreadsheet running speed is greatly improved and the latency between data entries is eliminated. The spreadsheet now runs the same speed as native Excel file.
2011-12-16	1.2.1	User now can select 180 degree hook or hairpin as vertical tensile anchor reinforcement top anchorage option and reduce the required anchor bolt embedment depth h_{ef} In previous versions the vertical tensile anchor reinforcement top anchorage can be only straight bar. In order to provide enough development length on both sides of the concrete breakout failure plane, the anchor bolt embedment depth h_{ef} has to be very deep to provide enough straight bar development length l_d. if 180 degree hook or hairpin is used, the required development length l_{dh} is much less compared to l_d and thus reduce the required anchor bolt embedment depth h_{ef}
2011-12-12	1.2.0	Bug in anchor bolt side face blowout check fixed
2011-12-09	1.0.1	Bug in anchor bolt side face blowout check fixed
2011-11-12	1.0.0	First released

4.0 DESIGN EXAMPLES

Example 01: Anchor Bolt + Anchor Reinft + Tension & Shear + ACI 318-08 Code





N_u= 20 kips (Tension)		V_u = 25 kips			
Concrete	f_c'= 4 ksi	Rebar f_y = 60 ksi			
Pedestal size	16” x 16”				
Anchor bolt	F1554 Grade 36	1.0” dia	Hex Head	h_{ef} = 55”	h_a =60”
Seismic design category >= C					
Anchor reinforcement		Tension → 8-No 8 ver. bar			
		Shear → 2-layer, 4-leg No 4 hor. bar			

Provide built-up grout pad

ANCHOR BOLT DESIGN **Combined Tension and Shear**

Anchor bolt design based on | **Code Abbreviation**

ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D | *ACI 318-08*

PIP STE05121 Anchor Bolt Design Guide-2006 | *PIP STE05121*

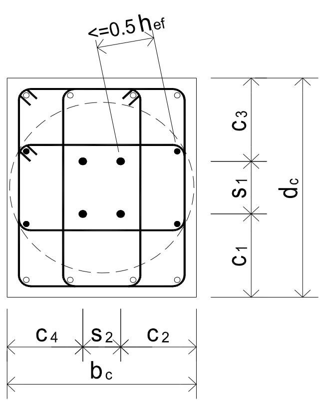
Code Reference

Assumptions | *ACI 318-08*

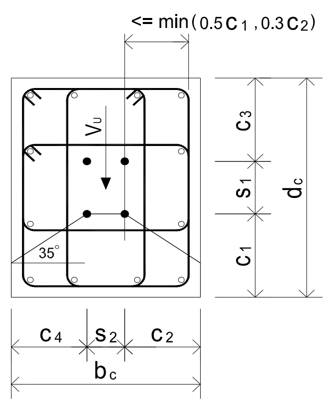
1. Concrete is cracked
2. Condition A - supplementary reinforcement is provided — D.4.4 (c)
3. Load combinations shall be as per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2 — D.4.4
4. Anchor reinft strength is used to replace concrete tension / shear breakout strength as per ACI318-08 Appendix D clause D.5.2.9 and D.6.2.9 — D.5.2.9 & D.6.2.9
5. For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
6. Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft
7. Anchor bolt washer shall be tack welded to base plate for all anchor bolts to transfer shear — *AISC Design Guide 1* section 3.5.3

Anchor Bolt Data

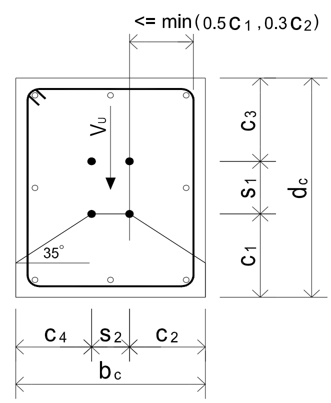
		set N_u = 0 if it's compression					
Factored tension for design	$\mathbf{N_u}$ =	**20.0**	[kips]		= 89.0	[kN]	
Factored shear	V_u =	25.0	[kips]		= 111.2	[kN]	
Factored shear for design	$\mathbf{V_u}$ =	**25.0**	[kips]	V_u = 0 if shear key is provided			
Concrete strength	f'_c =	4.0	[ksi]		= 27.6	[MPa]	
Anchor bolt material	=	F1554 Grade 36					
Anchor tensile strength	f_{uta} =	58	[ksi]		= 400	[MPa]	*ACI 318-08*
		Anchor is ductile steel element					D.1
Anchor bolt diameter	$\mathbf{d_a}$ =	**1**	[in]		= 25.4	[mm]	*PIP STE05121*
Bolt sleeve diameter	d_s =	3.0	[in]				Page A -1 Table 1
Bolt sleeve height	h_s =	10.0	[in]				
				min required			
Anchor bolt embedment depth	h_{ef} =	55.0	[in]	12.0	OK		Page A -1 Table 1
Pedestal height	h =	60.0	[in]	58.0	OK		
Pedestal width	b_c =	16.0	[in]				
Pedestal depth	d_c =	16.0	[in]				



Ver. Reinft For Tension



Hor. Ties For Shear - 4 Legs



Hor. Ties For Shear - 2 Legs

				min required		
Bolt edge distance c_1	c_1 =	5.0	[in]	4.5	OK	**Code Reference**
Bolt edge distance c_2	c_2 =	5.0	[in]	4.5	OK	*PIP STE05121*
Bolt edge distance c_3	c_3 =	5.0	[in]	4.5	OK	Page A -1 Table 1
Bolt edge distance c_4	c_4 =	5.0	[in]	4.5	OK	
Outermost bolt line spacing s_1	s_1 =	6.0	[in]	4.0	OK	Page A -1 Table 1
Outermost bolt line spacing s_2	s_2 =	6.0	[in]	4.0	OK	

ACI 318-08

To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within $0.5h_{ef}$ from the outmost anchor's centerline. In this design $0.5h_{ef}$ value is limited to 8 in. — RD.5.2.9

$0.5h_{ef}$ = **8.0** [in]

No of ver. rebar that are effective for resisting anchor tension n_v = 8

Ver. bar size No. 8 : 1.000 [in] dia single bar area A_s = 0.79 [in^2]

To be considered effective for resisting anchor shear, hor. reinft shall be located within min($0.5c_1$, $0.3c_2$) from the outmost anchor's centerline — RD.6.2.9

min($0.5c_1$, $0.3c_2$) = **1.5** [in]

No of tie leg that are effective to resist anchor shear n_{leg} = 4 ?

No of tie layer that are effective to resist anchor shear n_{lay} = 2 ?

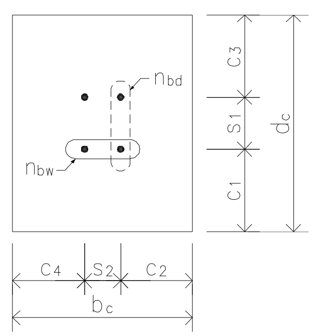
Hor. tie bar size No. 4 : 0.500 [in] dia single bar area A_s = 0.20 [in^2]

For anchor reinft shear breakout strength calc: 100% hor. tie bars develop full yield strength ?

Rebar yield strength f_y = 60 [ksi] suggest 60 = 414 [MPa]

No of bolt carrying tension n_t = 4

No of bolt carrying shear n_s = 4



Bolt No Input for Side-Face Blowout Check Use

For side-face blowout check use

No of bolt along width edge n_{bw} = 2

No of bolt along depth edge n_{bd} = 2

Anchor head type = Hex ?

Anchor effective cross sect area A_{se} = 0.606 [in^2]

Bearing area of head A_{brg} = **1.163** [in^2]

A_{brg} [in^2] not applicable

Bolt 1/8" (3mm) corrosion allowance	=	No	?	
Provide shear key ?	=	No	?	*ACI 318-08*
Seismic design category >= C	=	Yes	?	D.3.3.3
Provide built-up grout pad ?	=	Yes	?	D.6.1.3

Strength reduction factors

Anchor reinforcement	ϕ_s = 0.75			D.5.2.9 & D.6.2.9
Anchor rod - ductile steel	$\phi_{t,s}$ = 0.75		$\phi_{v,s}$ = 0.65	D.4.4(a)
Concrete - condition A	$\phi_{t,c}$ = 0.75		$\phi_{v,c}$ = 0.75	D.4.4(c)

CONCLUSION			**Code Reference**
Abchor Rod Embedment, Spacing and Edge Distance		**OK**	*ACI 318-08*
Min Rquired Anchor Reinft. Development Length	ratio = 0.25	**OK**	12.2.1
Overall	ratio = **0.70**	**OK**	
Tension			
Anchor Rod Tensile Resistance	ratio = 0.19	**OK**	
Anchor Reinft Tensile Breakout Resistance	ratio = 0.09	**OK**	
Anchor Pullout Resistance	ratio = 0.26	**OK**	
Side Blowout Resistance	ratio = 0.27	**OK**	
Shear			
Anchor Rod Shear Resistance	ratio = 0.57	**OK**	
Anchor Reinft Shear Breakout Resistance			
Strut Bearing Strength	ratio = 0.59	**OK**	
Tie Reinforcement	ratio = 0.46	**OK**	
Conc. Pryout Not Govern When h_{ef} >= $12d_a$		**OK**	
Tension Shear Interaction			
Tension Shear Interaction	ratio = 0.70	**OK**	

Ductility

Tension	**Non-ductile**	**Shear**	**Ductile**	*ACI 318-08*

Seismic Design Requirement **NG** D.3.3.4

SDC>= C, ACI318-08 D.3.3.5 or D.3.3.6 must be satisfied for non-ductile design

CACULATION

ACI 318-08

Anchor Rod Tensile Resistance $\phi_{t,s} N_{sa} = \phi_{t,s} n_t A_{se} f_{uta}$ = 105.4 [kips] D.5.1.2 (D-3)

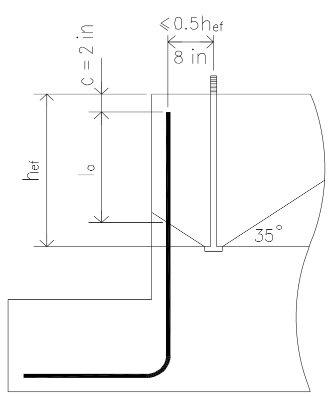
ratio = 0.19 > N_u **OK**

Anchor Reinft Tensile Breakout Resistance

Min tension development length for ver. #8 bar l_d = = 47.4 [in] 12.2.1, 12.2.2, 12.2.4

Actual development lenngth $l_a = h_{ef}$ - c (2 in) - 8 in x tan35 = 47.4 [in]

> 12.0 **OK** 12.2.1



ACI 318-08

$N_{rb} = \phi_s \times f_y \times n_v \times A_s \times (l_a / l_d$, if $l_a < l_d)$ = 284.2 [kips] 12.2.5

Seismic design strength reduction = x 0.75 applicable = 213.1 [kips] D.3.3.3

ratio = 0.09 > N_u **OK**

					Code Reference
Anchor Pullout Resistance					*ACI 318-08*
Single bolt pullout resistance	N_p	$= 8\ A_{brg}\ f_c'$	= 37.2	[kips]	D.5.3.4 (D-15)
	$N_{cpr} = \phi_{t,c}\ N_{pn}$	$= \phi_{t,c}\ n_t\ \Psi_{c,p}\ N_p$	= 104.2	[kips]	D.5.3.1 (D-14)
Seismic design strength reduction		= x 0.75 applicable	= 78.2	[kips]	D.3.3.3
	ratio	= 0.26	> N_u	**OK**	
	$\Psi_{c,p}$	= 1 for cracked conc			D.5.3.6
	$\phi_{t,c}$	= 0.70 pullout strength is always Condition B			D.4.4(c)

Side Blowout Resistance

Failure Along Pedestal Width Edge

Tensile load carried by anchors close to edge which may cause side-face blowout

along pedestal width edge	N_{buw}	$= N_u \times n_{bw} / n_t$	= 10.0	[kips]	RD.5.4.2
	c	$= \min(c_1, c_3)$	= 5.0	[in]	
Check if side blowout applicable	h_{ef}	= 55.0 [in]			
		> 2.5c side bowout is applicable			D.5.4.1
Check if edge anchors work as a	s_{22}	= 6.0 [in]	$s = s_2$ = 6.0	[in]	
a group or work individually		< 6c edge anchors work as a group			D.5.4.2
Single anchor SB resistance	$\phi_{t,c}\ N_{sb}$	$= \phi_{t,c}\left(160\,c\sqrt{A_{brg}}\right)\lambda\sqrt{f'_c}$	= 40.9	[kips]	D.5.4.1 (D-17)
Multiple anchors SB resistance	$\phi_{t,c}N_{sbg,w}$	=			
work as a group - applicable		$= (1+s/\ 6c) \times \phi_{t,c}\ N_{sb}$	= 49.1	[kips]	D.5.4.2 (D-18)
work individually - not applicable		$= n_{bw} \times \phi_{t,c}\ N_{sb} \times [1+(c_2 \text{ or } c_4)/\ c] / 4$	= 0.0	[kips]	D.5.4.1
Seismic design strength reduction		= x 0.75 applicable	= 36.8	[kips]	D.3.3.3
	ratio	= 0.27	> N_{buw}	**OK**	

Failure Along Pedestal Depth Edge

Tensile load carried by anchors close to edge which may cause side-face blowout

along pedestal depth edge	N_{bud}	$= N_u \times n_{bd} / n_t$	= 10.0	[kips]	RD.5.4.2
	c	$= \min(c_2, c_4)$	= 5.0	[in]	
Check if side blowout applicable	h_{ef}	= 55.0 [in]			
		> 2.5c side bowout is applicable			D.5.4.1
Check if edge anchors work as a	s_{11}	= 6.0 [in]	$s = s_1$ = 6.0	[in]	
a group or work individually		< 6c edge anchors work as a group			D.5.4.2
Single anchor SB resistance	$\phi_{t,c}\ N_{sb}$	$= \phi_{t,c}\left(160\,c\sqrt{A_{brg}}\right)\lambda\sqrt{f'_c}$	= 40.9	[kips]	D.5.4.1 (D-17)
Multiple anchors SB resistance	$\phi_{t,c}N_{sbg,d}$	=			
work as a group - applicable		$= (1+s/\ 6c) \times \phi_{t,c}\ N_{sb}$	= 49.1	[kips]	D.5.4.2 (D-18)
work individually - not applicable		$= n_{bd} \times \phi_{t,c}\ N_{sb} \times [1+(c_1 \text{ or } c_3)/\ c] / 4$	= 0.0	[kips]	D.5.4.1
Seismic design strength reduction		= x 0.75 applicable	= 36.8	[kips]	D.3.3.3
	ratio	= 0.27	> N_{bud}	**OK**	
Group side blowout resistance	$\phi_{t,c}\ N_{sbg}$	$= \phi_{t,c}\ \min\left(\frac{N_{sbg,w}}{n_{bw}}n_t\ ,\ \frac{N_{sbg,d}}{n_{bd}}n_t\right)$	= 73.7	[kips]	
Govern Tensile Resistance	$\mathbf{N_r}$	$= \phi_{t,c}\min(N_s, N_{rb}, N_{cp}, N_{sbg})$	= **73.7**	[kips]	

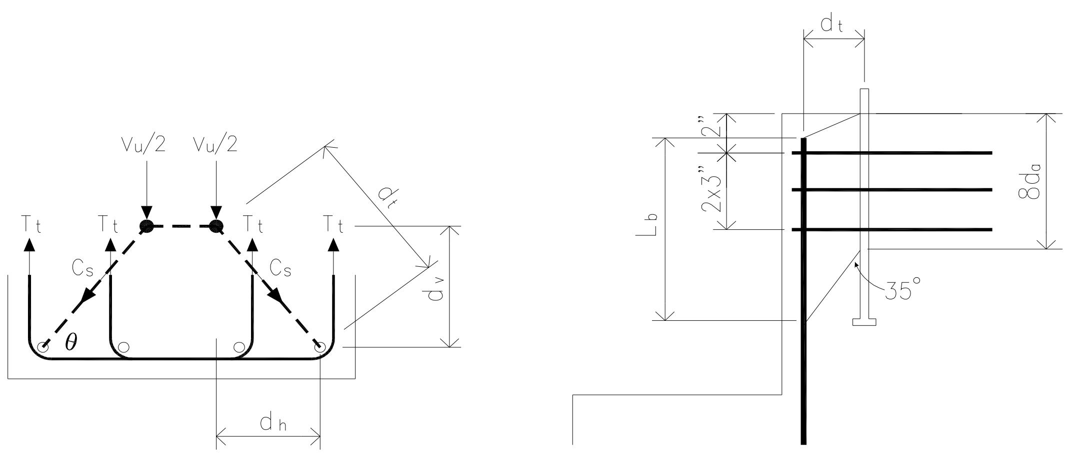
Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear

				Code Reference
				ACI 318-08
Anchor Rod Shear Resistance	$\phi_{v,s} V_{sa} = \phi_{v,s}\, n_s\, 0.6\, A_{se}\, f_{uta}$	= 54.8	[kips]	D.6.1.2 (b) (D-20)
Reduction due to built-up grout pads	= x 0.8 , applicable	= 43.9	[kips]	D.6.1.3
	ratio = 0.57	> V_u	**OK**	

Anchor Reinft Shear Breakout Resistance

Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft

STM strength reduction factor	ϕ_{st} = 0.75			9.3.2.6



Strut-and-Tie model geometry	d_v = 2.250 [in]	d_h = 2.250	[in]	
	θ = 45	d_t = 3.182	[in]	
Strut compression force	$C_s = 0.5\, V_u / \sin\theta$	= 17.7	[kips]	
				ACI 318-08
Strut Bearing Strength				
Strut compressive strength	$f_{ce} = 0.85\, f'_c$	= 3.4	[ksi]	A.3.2 (A-3)
* Bearing of anchor bolt				
Anchor bearing length	$l_e = \min(8d_a, h_{ef})$	= 8.0	[in]	D.6.2.2
Anchor bearing area	$A_{brg} = l_e \times d_a$	= 8.0	[in^2]	
Anchor bearing resistance	$C_r = n_s \times \phi_{st} \times f_{ce} \times A_{brg}$	= 81.6	[kips]	
		> V_u	**OK**	
* Bearing of ver reinft bar				
Ver bar bearing area	$A_{brg} = (l_e + 1.5 \times d_t - d_a/2 - d_b/2) \times d_b$	= 11.8	[in^2]	
Ver bar bearing resistance	$C_r = \phi_{st} \times f_{ce} \times A_{brg}$	= 30.0	[kips]	
	ratio = 0.59	> C_s	**OK**	

Tie Reinforcement **Code Reference**

* For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
* For enclosed tie, at hook location the tie cannot develop full yield strength f_y . Use the pullout resistance in tension of a single hooked bolt as per ACI318-08 Eq. (D-16) as the max force can be developed at hook T_h
* Assume 100% of hor. tie bars can develop full yield strength.

Total number of hor tie bar	$n = n_{leg}$ (leg) x n_{lay} (layer)	= 8		
				ACI 318-08
Pull out resistance at hook	$T_h = \phi_{t,c}\ 0.9\ f_c'\ e_h\ d_a$	= 3.0	[kips]	D.5.3.5 (D-16)
	$e_h = 4.5\ d_b$	= 2.250	[in]	
Single tie bar tension resistance	$T_r = \phi_s \times f_y \times A_s$	= 9.0	[kips]	
Total tie bar tension resistance	$V_{rb} = 1.0 \times n \times Tr$	= 72.0	[kips]	
Seismic design strength reduction	= x 0.75 applicable	= 54.0	[kips]	D.3.3.3
	ratio = 0.46	> V_u	**OK**	

Conc. Pryout Shear Resistance

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} > = $12d_a$, the pryout failure will not govern

$12d_a$ = 12.0 [in]	h_{ef} = 55.0	[in]	
	> $12d_a$	**OK**	

Govern Shear Resistance	$\mathbf{V_r} = \min(\phi_{v,s}V_{sa}, V_{rb})$	= **43.9**	[kips]

Tension Shear Interaction

Check if $N_u > 0.2\phi\ N_n$ and $V_u > 0.2\phi\ V_n$	Yes			D.7.1 & D.7.2
	$N_u / \phi\ N_n + V_u / \phi\ V_n$	= 0.84		D.7.3 (D-32)
	ratio = 0.70	< 1.2	**OK**	

Ductility Tension

$\phi_{t,s}\ N_{sa}$ = 105.4 [kips]

> $\min[N_{rb}, \phi_{t,c}(N_{pn}, N_{sbg})]$ = 73.7 [kips]

Non-ductile

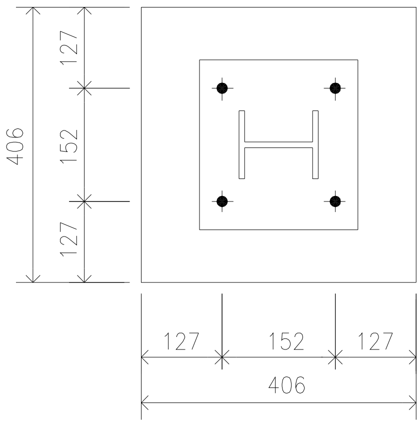
Ductility Shear

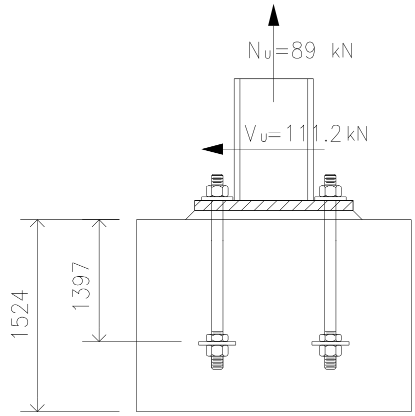
$\phi_{v,s}\ V_{sa}$ = 43.9 [kips]

< V_{rb} = 54.0 [kips]

Ductile

Example 02: Anchor Bolt + Anchor Reinft + Tension & Shear + CSA A23.3-04 Code





N_u= 89 kN (Tension) V_u = 111.2 kN

Concrete f_c'= 27.6 MPa Rebar f_y = 414 MPa

Pedestal size 406mm x 406mm

Anchor bolt F1554 Grade 36 1.0" dia Hex Head h_{ef} = 1397mm h_a =1524mm

Seismic design $I_E\,F_aS_a(0.2)$ >= 0.35

Anchor reinforcement Tension → 8-25M ver. bar

Shear → 2-layer, 4-leg 15M hor. bar

Provide built-up grout pad

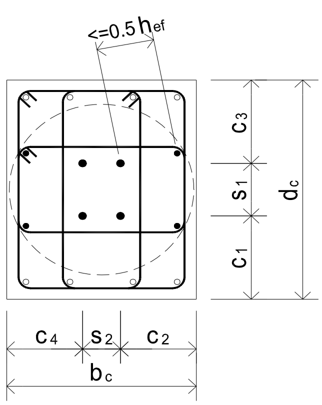
ANCHOR BOLT DESIGN **Combined Tension and Shear**

Anchor bolt design based on	**Code Abbreviation**
CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D	*A23.3-04 (R2010)*
ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary	*ACI318 M-08*
PIP STE05121 Anchor Bolt Design Guide-2006	*PIP STE05121*

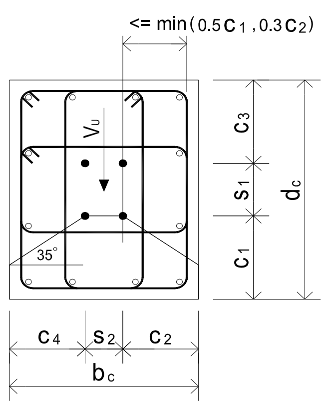
Assumptions	**Code Reference**
1. Concrete is cracked	*A23.3-04 (R2010)*
2. Condition A - supplementary reinforcement is provided	D.5.4 (c)
3. Anchor reinft strength is used to replace concrete tension / shear breakout strength as per	*ACI318 M-08*
ACI318 M-08 Appendix D clause D.5.2.9 and D.6.2.9	D.5.2.9 & D.6.2.9
4. For tie reinft, only the top most 2 or 3 layers of ties (50mm from TOC and 2x75mm after) are effective	
5. Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft	
6. Anchor bolt washer shall be tack welded to base plate for all anchor bolts to transfer shear	*AISC Design Guide 1*
	section 3.5.3

Input Data

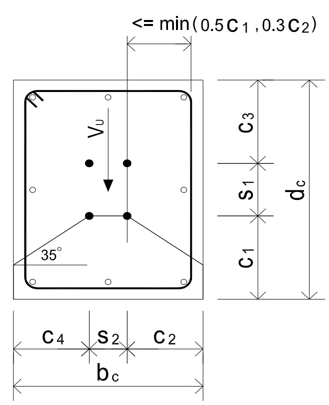
			set N_u = 0 if it's compression				
Factored <u>tension</u> for design	$\mathbf{N_u}$ =	**89.0**	[kN]		= 20.0	[kips]	
Factored shear	V_u =	111.2	[kN]		= 25.0	[kips]	
Factored shear for design	$\mathbf{V_u}$ =	**111.2**	[kN]	V_u = 0 if shear key is provided			
Concrete strength	f'_c =	28	[MPa]		= 4.0	[ksi]	
Anchor bolt material	=	F1554 Grade 36					
Anchor tensile strength	f_{uta} =	58	[ksi]		= 400	[MPa]	*A23.3-04 (R2010)*
		Anchor is ductile steel element					D.2
Anchor bolt diameter	$\mathbf{d_a}$ =	**1**	[in]		= 25.4	[mm]	*PIP STE05121*
Bolt sleeve diameter	d_s =	76	[mm]				Page A -1 Table 1
Bolt sleeve height	h_s =	254	[mm]				
				min required			
Anchor bolt embedment depth	h_{ef} =	1397	[mm]	305	**OK**		Page A -1 Table 1
Pedestal height	h =	1524	[mm]	1473	**OK**		
Pedestal width	b_c =	406	[mm]				
Pedestal depth	d_c =	406	[mm]				



Ver. Reinft For Tension



Hor. Ties For Shear - 4 Legs



Hor. Ties For Shear - 2 Legs

				min required		
Bolt edge distance c_1	c_1 =	127	[mm]	114	OK	**Code Reference**
Bolt edge distance c_2	c_2 =	127	[mm]	114	OK	*PIP STE05121*
Bolt edge distance c_3	c_3 =	127	[mm]	114	OK	Page A -1 Table 1
Bolt edge distance c_4	c_4 =	127	[mm]	114	OK	
Outermost bolt line spacing s_1	s_1 =	152	[mm]	102	OK	Page A -1 Table 1
Outermost bolt line spacing s_2	s_2 =	152	[mm]	102	OK	

ACI318 M-08

To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within $0.5h_{ef}$ from the outmost anchor's centerline. In this design $0.5h_{ef}$ value is limited to 200mm. RD.5.2.9

$0.5h_{ef}$ = **200** [mm]

No of ver. rebar that are effective for resisting anchor tension n_v = 8

Ver. bar size d_b = 25 single bar area A_s = 500 [mm^2]

To be considered effective for resisting anchor shear, hor. reinft shall be located within min($0.5c_1$, $0.3c_2$) from the outmost anchor's centerline min($0.5c_1$, $0.3c_2$) = **38** [mm] RD.6.2.9

No of tie leg that are effective to resist anchor shear n_{leg} = 4 ?

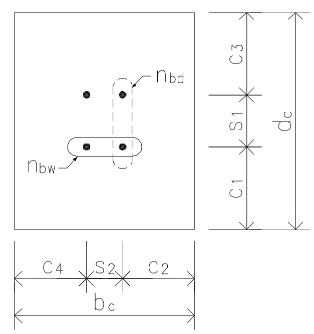
No of tie layer that are effective to resist anchor shear n_{lay} = 2 ?

Hor. bar size d_b = 15 single bar area A_s = 200 [mm^2]

For anchor reinft shear breakout strength calc 100% hor. tie bars develop full yield strength ?

suggest

Rebar yield strength f_y = 414 [MPa] 400 = 60.0 [ksi]



Bolt No Input for Side-Face Blowout Check Use

No of bolt carrying tension n_t = 4

No of bolt carrying shear n_s = 4

For side-face blowout check use

No of bolt along width edge n_{bw} = 2

No of bolt along depth edge n_{bd} = 2

Anchor head type = Hex ?

A_{se} = 391 [mm^2]

Bearing area of head A_{brg} = **750** [mm^2]

A_{brg} [mm^2] not applicable

Bolt 1/8" (3mm) corrosion allowance = No ?

Provide shear key ? = No ?

Seismic region where $I_E F_a S_a(0.2)$>=0.35	= Yes	?	*A23.3-04 (R2010)* D.4.3.5
Provide built-up grout pad ?	= Yes	?	D.7.1.3
Strength reduction factors			
Anchor reinforcement factor	ϕ_{as} = 0.75		D.7.2.9
Steel anchor resistance factor	ϕ_s = 0.85		8.4.3 (a)
Concrete resistance factor	ϕ_c = 0.65		8.4.2

Resistance modification factors

Anchor rod - ductile steel	$R_{t,s}$ = 0.80	$R_{v,s}$ = 0.75	D.5.4(a)
Concrete - condition A	$R_{t,c}$ = 1.15	$R_{v,c}$ = 1.15	D.5.4(c)

CONCLUSION			**Code Reference**
Abchor Rod Embedment, Spacing and Edge Distance		**OK**	*A23.3-04 (R2010)*
Min Rquired Anchor Reinft. Development Length	ratio = 0.25	**OK**	12.2.1
Overall	ratio = **0.71**	**OK**	
Tension			
Anchor Rod Tensile Resistance	ratio = 0.21	**OK**	
Anchor Reinft Tensile Breakout Resistance	ratio = 0.10	**OK**	
Anchor Pullout Resistance	ratio = 0.28	**OK**	
Side Blowout Resistance	ratio = 0.27	**OK**	
Shear			
Anchor Rod Shear Resistance	ratio = 0.58	**OK**	
Anchor Reinft Shear Breakout Resistance			
Strut Bearing Strength	ratio = 0.60	**OK**	
Tie Reinforcement	ratio = 0.30	**OK**	
Conc. Pryout Not Govern When h_{ef} >= $12d_a$		**OK**	
Anchor Rod on Conc Bearing	ratio = 0.21	**OK**	
Tension Shear Interaction			
Tension Shear Interaction	ratio = 0.71	**OK**	
Ductility			
Tension **Non-ductile**	**Shear** **Ductile**		
Seismic Design Requirement		**NG**	D.4.3.6

IeFaSa(0.2)>=0.35, A23.3-04 D.4.3.7 or D.4.3.8 must be satisfied for non-ductile design

CACULATION *A23.3-04 (R2010)*

Anchor Rod Tensile Resistance

$N_{sr} = n_t\, A_{se}\, \phi_s\, f_{uta}\, R_{t,s}$ = 425.3 [kN] D.6.1.2 (D-3)

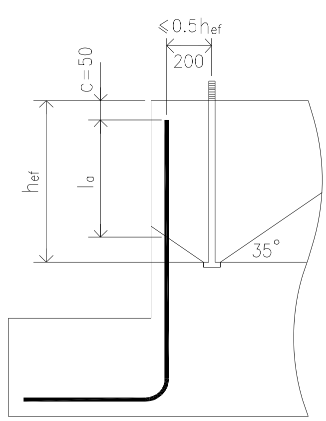
ratio = 0.21 > N_u **OK**

Anchor Reinft Tensile Breakout Resistance

Min tension development length for ver. 25M bar l_d = = 887 [mm] 12.2.3

Actual development lenngth $l_a = h_{ef}$ - c (50mm) - 200mm x tan35 = 1207 [mm]

> 300 **OK** 12.2.1



$N_{rbr} = \phi_{as} \times f_y \times n_v \times A_s \times (l_a / l_d$, if $l_a < l_d)$ = 1242.0 [kN] 12.2.5

Seismic design strength reduction = x 0.75 applicable = 931.5 [kN] D.4.3.5

ratio = 0.10 > N_u **OK**

					Code Reference
Anchor Pullout Resistance					*A23.3-04 (R2010)*
Single bolt pullout resistance	N_{pr} =	$8\ A_{brg}\ \phi_c\ f_c'\ R_{t,c}$	= 107.7	[kN]	D.6.3.4 (D-16)
	N_{cpr} =	$n_t\ \Psi_{c,p}\ N_{pr}$	= 430.7	[kN]	D.6.3.1 (D-15)
Seismic design strength reduction	=	x 0.75 applicable	= 323.1	[kN]	D.4.3.5
	ratio =	0.28	> N_u	**OK**	
	$\Psi_{c,p}$ =	1 for cracked conc			D.6.3.6
	$R_{t,c}$ =	1.00 pullout strength is always Condition B			D.5.4(c)
Side Blowout Resistance					
Failure Along Pedestal Width Edge					
Tensile load carried by anchors close to edge which may cause side-face blowout					*ACI318 M-08*
along pedestal width edge	N_{buw} =	$N_u \times n_{bw} / n_t$	= 44.5	[kN]	RD.5.4.2
	c =	min (c_1, c_3)	= 127	[mm]	
Check if side blowout applicable	h_{ef} =	1397 [mm]			*A23.3-04 (R2010)*
	>	2.5c side bowout is applicable			D.6.4.1
Check if edge anchors work as a	s_{22} =	152 [mm]	s = s_2 = 152	[mm]	
a group or work individually	<	6c edge anchors work as a group			D.6.4.2
Single anchor SB resistance	$N_{sbr,w}$ =	$13.3c\sqrt{A_{brg}}\ \phi_c\ \sqrt{f'_c}\ R_{t,c}$	= 181.7	[kN]	D.6.4.1 (D-18)
Multiple anchors SB resistance	$N_{sbgr,w}$ =				
work as a group - applicable	=	(1+s/ 6c) x $N_{sbr,w}$	= 217.9	[kN]	D.6.4.2 (D-19)
work individually - not applicable	=	$n_{bw} \times N_{sbr,w} \times [1+(c_2 \text{ or } c_4) / c] / 4$	= 0.0	[kN]	D.6.4.1
Seismic design strength reduction	=	x 0.75 applicable	= 163.5	[kN]	D.4.3.5
	ratio =	0.27	> N_{buw}	**OK**	
Failure Along Pedestal Depth Edge					
Tensile load carried by anchors close to edge which may cause side-face blowout					*ACI318 M-08*
along pedestal depth edge	N_{bud} =	$N_u \times n_{bd} / n_t$	= 44.5	[kN]	RD.5.4.2
	c =	min (c_2, c_4)	= 127	[mm]	
Check if side blowout applicable	h_{ef} =	1397 [mm]			*A23.3-04 (R2010)*
	>	2.5c side bowout is applicable			D.6.4.1
Check if edge anchors work as a	s_{11} =	152 [mm]	s = s_1 = 152	[mm]	
a group or work individually	<	6c edge anchors work as a group			D.6.4.2
Single anchor SB resistance	$N_{sbr,d}$ =	$13.3c\sqrt{A_{brg}}\ \phi_c\ \sqrt{f'_c}\ R_{t,c}$	= 181.7	[kN]	D.6.4.1 (D-18)
Multiple anchors SB resistance	$N_{sbgr,d}$ =				
work as a group - applicable	=	(1+s/ 6c) x $\phi_{t,c}\ N_{sbr,d}$	= 217.9	[kN]	D.6.4.2 (D-19)
work individually - not applicable	=	$n_{bd} \times N_{sbr,d} \times [1+(c_1 \text{ or } c_3) / c] / 4$	= 0.0	[kN]	D.6.4.1
Seismic design strength reduction	=	x 0.75 applicable	= 163.5	[kN]	D.4.3.5
	ratio =	0.27	> N_{bud}	**OK**	
Group side blowout resistance	N_{sbgr} =	$\min\left(\dfrac{N_{sbgr,w}}{n_{bw}} n_t\ ,\ \dfrac{N_{sbgr,d}}{n_{bd}} n_t\right)$	= 326.9	[kN]	
Govern Tensile Resistance	$\mathbf{N_r}$ =	min (N_{sr}, N_{rbr}, N_{cpr}, N_{sbgr})	= **323.1**	[kN]	

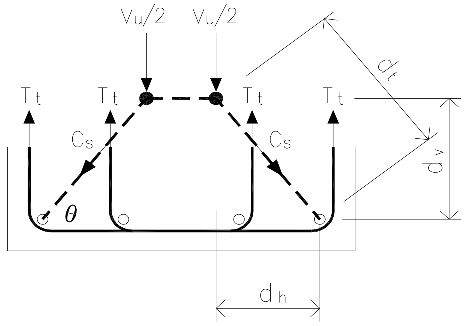
Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear

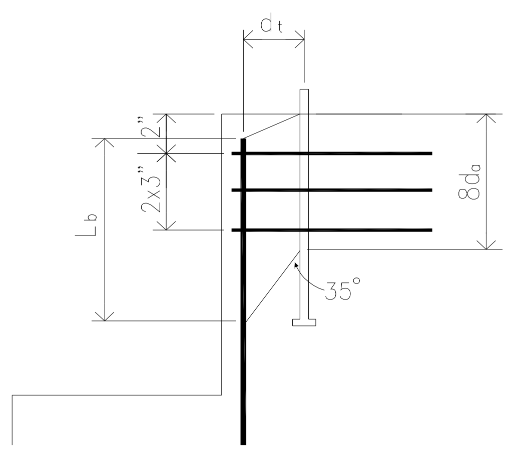
				Code Reference
				A23.3-04 (R2010)
Anchor Rod Shear Resistance	$V_{sr} = n_s A_{se} \phi_s\, 0.6\, f_{uta} R_{v,s}$	= 239.2	[kN]	D.7.1.2 (b) (D-21)
Reduction due to built-up grout pads	= x 0.8 , applicable	= 191.4	[kN]	D.7.1.3
	ratio = 0.58	> V_u	**OK**	

Anchor Reinft Shear Breakout Resistance *ACI318 M-08*

Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft

STM strength reduction factor ϕ_{st} = 0.75 — 9.3.2.6





Strut-and-Tie model geometry	d_v = 57 [mm]	d_h = 57	[mm]	
	θ = 45	d_t = 81	[mm]	
Strut compression force	$C_s = 0.5\, V_u / \sin\theta$	= 78.6	[kN]	

ACI318 M-08

Strut Bearing Strength

Strut compressive strength	$f_{ce} = 0.85\, f'_c$	= 23.5	[MPa]	A.3.2 (A-3)
* Bearing of anchor bolt				
Anchor bearing length	$l_e = \min(8d_a, h_{ef})$	= 203	[mm]	D.6.2.2
Anchor bearing area	$A_{brg} = l_e \times d_a$	= 5161	[mm^2]	
Anchor bearing resistance	$C_r = n_s \times \phi_{st} \times f_{ce} \times A_{brg}$	= 363.3	[kN]	
		> V_u	**OK**	
* Bearing of ver reinft bar				
Ver bar bearing area	$A_{brg} = (l_e + 1.5 \times d_t - d_a/2 - d_b/2) \times d_b$	= 7473	[mm^2]	
Ver bar bearing resistance	$C_r = \phi_{st} \times f_{ce} \times A_{brg}$	= 131.5	[kN]	
	ratio = 0.60	> C_s	**OK**	

Tie Reinforcement				Code Reference

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

* For enclosed tie, at hook location the tie cannot develop full yield strength f_y . Use the pullout resistance in tension of a single J-bolt as per A23.3-04 Annex D Eq. (D-17) as the max force can be developed at hook T_h

* Assume 100% of hor. tie bars can develop full yield strength.

Total number of hor tie bar	$n = n_{leg}$ (leg) x n_{lay} (layer)	= 8		
				A23.3-04 (R2010)
Pull out resistance at hook	$T_h = 0.9\ \phi_c\ f_c'\ e_h\ d_b\ R_{t,c}$	= 16.3	[kN]	D.6.3.5 (D-17)
	$e_h = 4.5\ d_b$	= 68	[mm]	
Single tie bar tension resistance	$T_r = \phi_{as} \times f_y \times A_s$	= 62.1	[kN]	
Total tie bar tension resistance	V_{rbr} = 1.0 x n x Tr	= 496.8	[kN]	
Seismic design strength reduction	= x 0.75 applicable	= 372.6	[kN]	D.4.3.5
	ratio = 0.30	> V_u	**OK**	

Conc. Pryout Shear Resistance

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} > = $12d_a$, the pryout failure will not govern

	$12d_a$ = 305 [mm]	h_{ef} = 1397	[mm]		
		> $12d_a$	**OK**		
				CSA S16-09	
Anchor Rod on Conc Bearing	$B_r = n_s \times 1.4 \times \phi_c \times \min(8d_a, h_{ef}) \times d_a \times f_c'$	= 518.5	[kN]	25.3.3.2	
	ratio = 0.21	> V_u	**OK**		
Govern Shear Resistance	$\mathbf{V_r}$ = min (V_{sr} , V_{rbr} , B_r)	= **191.4**	[kN]		
				A23.3-04 (R2010)	
Tension Shear Interaction					
Check if N_u >0.2 N_r and V_u >0.2 V_r	Yes			D.8.2 & D.8.3	
	$N_u/N_r + V_u/V_r$	= 0.86		D.8.4 (D-35)	
	ratio = 0.71	< 1.2	**OK**		

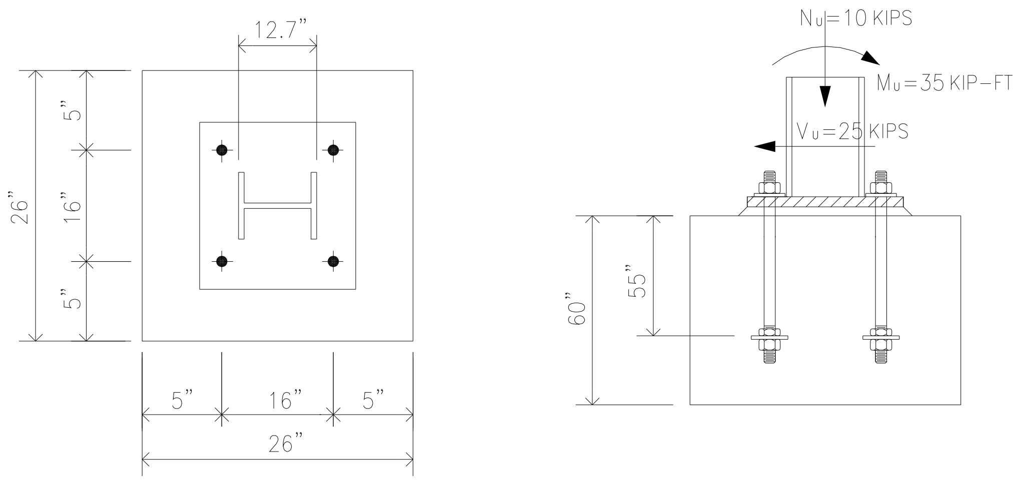
Ductility Tension

N_{sr} = 425.3 [kN]			
> min (N_{rbr} , N_{cpr}, N_{sbgr})	= 323.1	[kN]	
Non-ductile			

Ductility Shear

V_{sr} = 191.4 [kN]			
< min (V_{rbr}, B_r)	= 372.6	[kN]	
Ductile			

Example 03: Anchor Bolt + Anchor Reinft + Tension Shear & Moment + ACI 318-08 Code



M_u = 35 kip-ft	N_u= 10 kips (Compression)		V_u = 25 kips		
Concrete	f_c'= 4 ksi	Rebar f_y = 60 ksi			
Pedestal size	26" x 26"				
Anchor bolt	F1554 Grade 36	1.25" dia	Hex Head	h_{ef} = 55"	h_a =60"

Seismic design category < C

Anchor reinforcement — Tension → 2-No 8 ver. bar

Shear → 2-layer, 2-leg No 4 hor. bar

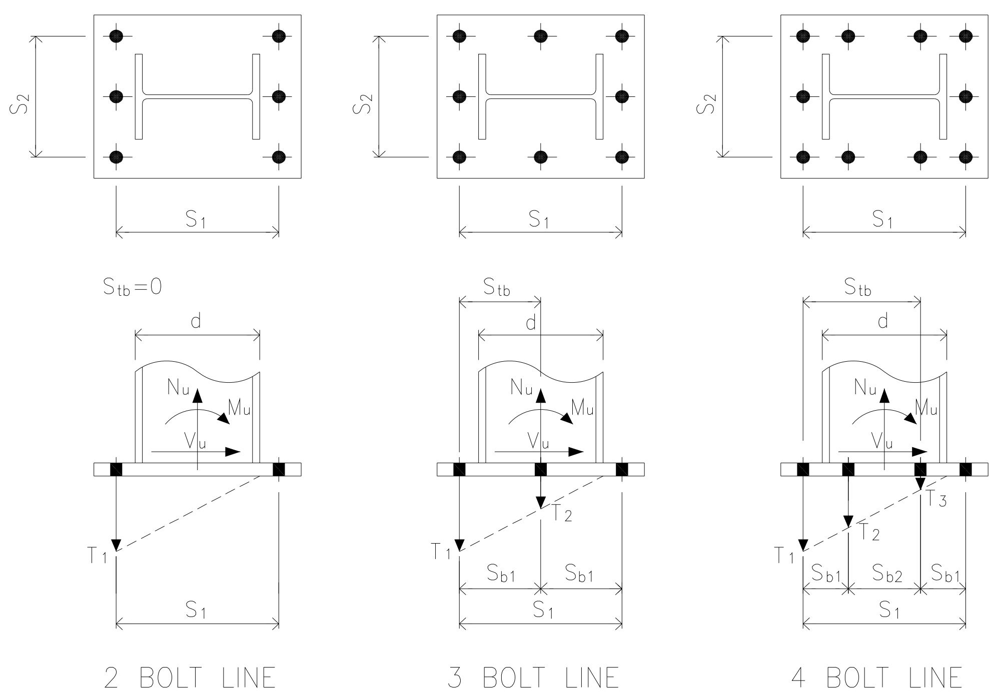
Provide built-up grout pad

ANCHOR BOLT DESIGN **Combined Tension, Shear and Moment**

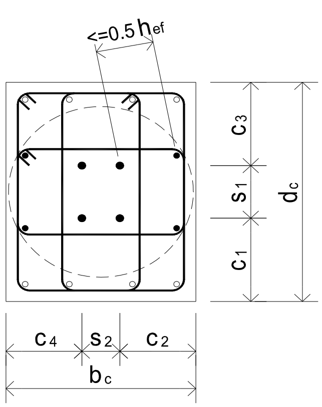
Anchor bolt design based on	**Code Abbreviation**
ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D	*ACI 318-08*
PIP STE05121 Anchor Bolt Design Guide-2006	*PIP STE05121*
	Code Reference
Assumptions	*ACI 318-08*
1. Concrete is cracked	
2. Condition A - supplementary reinforcement is provided	D.4.4 (c)
3. Load combinations shall be as per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2	D.4.4
4. Anchor reinft strength is used to replace concrete tension / shear breakout strength as per ACI318-08 Appendix D clause D.5.2.9 and D.6.2.9	D.5.2.9 & D.6.2.9
5. For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective	
6. Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft	
7. For anchor group subject to moment, the anchor tensile load is designed using elastic analysis and there is no redistribution of the forces between highly stressed and less stressed anchors	D.3.1
8. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output	
9. Shear carried by only half of total anchor bolts due to oversized holes in column base plate	*AISC Design Guide 1* section 3.5.3

Anchor Bolt Data

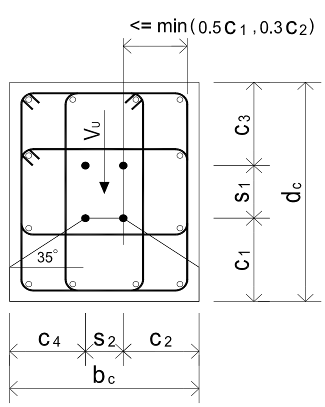
Factored moment	M_u =	35.0	[kip-ft]		= 47.5	[kNm]
Factored tension /compression	N_u =	-10.0	[kips]	in compression	= -44.5	[kN]
Factored shear	V_u =	25.0	[kips]		= 111.2	[kN]
Factored shear for design	$\mathbf{V_u}$ =	**25.0**	[kips]	V_u = 0 if shear key is provided		



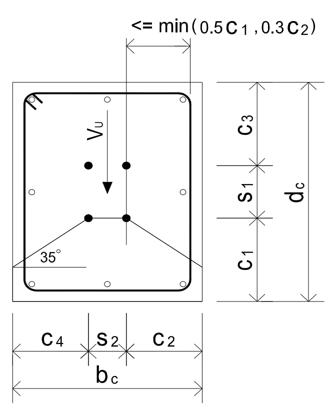
							Code Reference
No of bolt line for resisting moment	=	2 Bolt Line					
No of bolt along outermost bolt line	=	2					
				min required			*PIP STE05121*
Outermost bolt line spacing s_1	s_1 =	16.0	[in]	5.0	**OK**		Page A -1 Table 1
Outermost bolt line spacing s_2	s_2 =	16.0	[in]	5.0	**OK**		
Internal bolt line spacing s_{b1}	s_{b1} =	10.5	[in]	5.0	**OK**		
Internal bolt line spacing s_{b2}	s_{b2} =	0.0	[in]	5.0	**OK**		
Column depth	d =	12.7	[in]				
Concrete strength	f'_c =	4.0	[ksi]			= 27.6 [MPa]	
Anchor bolt material	=	F1554 Grade 36					
Anchor tensile strength	f_{uta} =	58	[ksi]			= 400 [MPa]	*ACI 318-08*
		Anchor is ductile steel element					D.1
Anchor bolt diameter	**d_a** =	**1.25**	[in]			= 31.8 [mm]	*PIP STE05121*
Bolt sleeve diameter	d_s =	3.0	[in]				Page A -1 Table 1
Bolt sleeve height	h_s =	10.0	[in]				
				min required			
Anchor bolt embedment depth	h_{ef} =	55.0	[in]	15.0	**OK**		Page A -1 Table 1
Pedestal height	h =	60.0	[in]	58.0	**OK**		
Pedestal width	b_c =	26.0	[in]				
Pedestal depth	d_c =	26.0	[in]				
Bolt edge distance c_1	c_1 =	5.0	[in]	5.0	**OK**		Page A -1 Table 1
Bolt edge distance c_2	c_2 =	5.0	[in]	5.0	**OK**		
Bolt edge distance c_3	c_3 =	5.0	[in]	5.0	**OK**		
Bolt edge distance c_4	c_4 =	5.0	[in]	5.0	**OK**		



Ver. Reinft For Tension

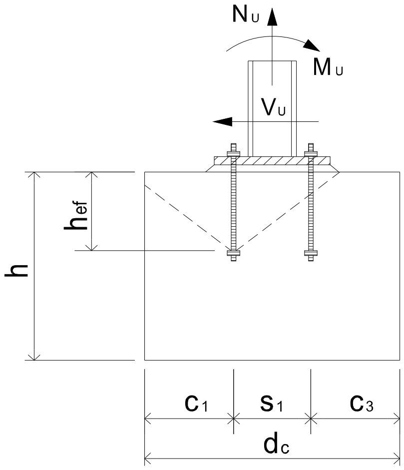


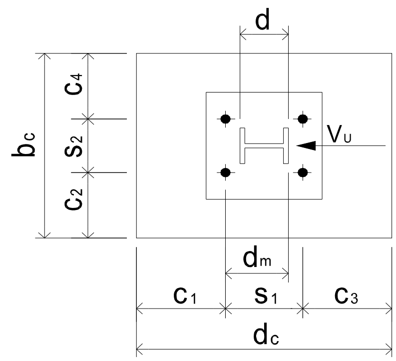
Hor. Ties For Shear - 4 Legs



Hor. Ties For Shear - 2 Legs

Code Reference





ACI 318-08

To be considered effective for resisting anchor tension, ver reinforcing bars shall be located — RD.5.2.9

within $0.5h_{ef}$ from the outmost anchor's centerline. In this design $0.5h_{ef}$ value is limited to 8 in.

$0.5h_{ef}$ = **8.0** [in]

No of ver. rebar that are effective for resisting anchor tension n_v = 2

Ver. bar size No. 8 : 1.000 [in] dia single bar area A_s = 0.79 [in^2]

To be considered effective for resisting anchor shear, hor. reinft shall be located — RD.6.2.9

within min($0.5c_1$, $0.3c_2$) from the outmost anchor's centerline min($0.5c_1$, $0.3c_2$) = **1.5** [in]

No of tie <u>leg</u> that are effective to resist anchor shear n_{leg} = 2 ?

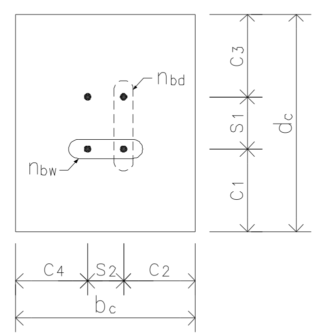
No of tie <u>layer</u> that are effective to resist anchor shear n_{lay} = 2 ?

Hor. tie bar size No. 4 : 0.500 [in] dia single bar area A_s = 0.20 [in^2]

For anchor reinft shear breakout strength calc 100% hor. tie bars develop full yield strength ?

suggest

Rebar yield strength f_y = 60 [ksi] 60 = 414 [MPa]



Bolt No Input for Side-Face Blowout Check Use

Total no of anchor bolt n = 4

No of bolt carrying tension n_t = 2

No of bolt carrying shear n_s = 2

For side-face blowout check use

No of bolt along width edge n_{bw} = 2

Anchor head type = Hex ?

Anchor effective cross sect area A_{se} = 0.969 [in^2]

Bearing area of head A_{brg} = **1.817** [in^2]

A_{brg} [in^2] not applicable

Bolt 1/8" (3mm) corrosion allowance = No ?

Provide shear key ? = No ? — *ACI 318-08*

Seismic design category >= C = No ? — D.3.3.3

Provide built-up grout pad ? = Yes ? — D.6.1.3

					Code Reference
Strength reduction factors					*ACI 318-08*
Anchor reinforcement	ϕ_s = 0.75				D.5.2.9 & D.6.2.9
Anchor rod - ductile steel	$\phi_{t,s}$ = 0.75		$\phi_{v,s}$ = 0.65		D.4.4(a)
Concrete - condition A	$\phi_{t,c}$ = 0.75		$\phi_{v,c}$ = 0.75		D.4.4(c)

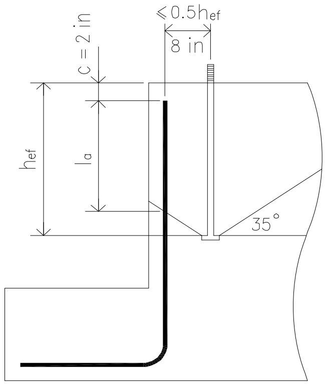
CONCLUSION

Abchor Rod Embedment, Spacing and Edge Distance			**OK**	
Min Rquired Anchor Reinft. Development Length		ratio = 0.25	**OK**	12.2.1
Overall		ratio = **0.89**	**OK**	
Tension				
Anchor Rod Tensile Resistance		ratio = 0.29	**OK**	
Anchor Reinft Tensile Breakout Resistance		ratio = 0.35	**OK**	
Anchor Pullout Resistance		ratio = 0.31	**OK**	
Side Blowout Resistance		ratio = 0.32	**OK**	
Shear				
Anchor Rod Shear Resistance		ratio = 0.71	**OK**	
Anchor Reinft Shear Breakout Resistance				
Strut Bearing Strength		ratio = 0.51	**OK**	
Tie Reinforcement		ratio = 0.69	**OK**	
Conc. Pryout Not Govern When h_{ef} >= $12d_a$			**OK**	
Tension Shear Interaction				
Tension Shear Interaction		ratio = 0.89	**OK**	
Ductility				
	Tension **Non-ductile**	**Shear**	**Ductile**	*ACI 318-08*
Seismic Design Requirement			**OK**	D.3.3.4
	SDC< C, ACI318-08 D.3.3 ductility requirement is NOT required			

CACULATION

Anchor Tensile Force					*ACI 318-08*
Single bolt tensile force	T_1 = 12.42 [kips]	No of bolt for T_1 n_{T1} = 2			
	T_2 = 0.00 [kips]	No of bolt for T_2 n_{T2} = 0			
	T_3 = 0.00 [kips]	No of bolt for T_3 n_{T3} = 0			
Sum of bolt tensile force	$\mathbf{N_u} = \Sigma\, n_i\, T_i$		= **24.8**	[kips]	
Anchor Rod Tensile	$\phi_{t,s} N_{sa} = \phi_{t,s} A_{se} f_{uta}$		= 42.2	[kips]	D.5.1.2 (D-3)
Resistance	ratio = 0.29		> T_1	**OK**	
Anchor Reinft Tensile Breakout Resistance					
Min tension development length	l_d =		= 47.4	[in]	12.2.1, 12.2.2, 12.2.4
for ver. #8 bar					
Actual development lenngth	$l_a = h_{ef}$ - c (2 in) - 8 in x tan35		= 47.4	[in]	
			> 12.0	**OK**	12.2.1

Code Reference



ACI 318-08

	N_{rbr} =	ϕ_s x f_y x n_v x A_s x (l_a / l_d , if l_a < l_d)	= 71.0	[kips]	12.2.5
Seismic design strength reduction	=	x 1.0 not applicable	= 71.0	[kips]	D.3.3.3
	ratio =	0.35	> N_u	**OK**	

Anchor Pullout Resistance

Single bolt pullout resistance	N_p =	8 A_{brg} f_c'	= 58.1	[kips]	D.5.3.4 (D-15)
	$N_{cpr} = \phi_{t,c} N_{pn}$ =	$\phi_{t,c}$ $\Psi_{c,p}$ N_p	= 40.7	[kips]	D.5.3.1 (D-14)
Seismic design strength reduction	=	x 1.0 not applicable	= 40.7	[kips]	D.3.3.3
	ratio =	0.31	> T_1	**OK**	
	$\Psi_{c,p}$ =	1 for cracked conc			D.5.3.6
	$\phi_{t,c}$ =	0.70 pullout strength is always Condition B			D.4.4(c)

Side Blowout Resistance

<u>Failure Along Pedestal Width Edge</u>

Tensile load carried by anchors close to edge which may cause side-face blowout

along pedestal width edge	N_{buw} =	n_{T1} T_1	= 24.8	[kips]	RD.5.4.2
	c =	min (c_1, c_3)	= 5.0	[in]	
Check if side blowout applicable	h_{ef} =	55.0 [in]			
	>	2.5c side bowout is applicable			D.5.4.1
Check if edge anchors work as a	s_{22} =	16.0 [in]	s = s_2 = 16.0	[in]	
a group or work individually	<	6c edge anchors work as a group			D.5.4.2
Single anchor SB resistance	$\phi_{t,c} N_{sb}$ =	$\phi_{t,c}\left(160\,c\sqrt{A_{brg}}\right)\lambda\sqrt{f'_c}$	= 51.2	[kips]	D.5.4.1 (D-17)
Multiple anchors SB resistance	$\phi_{t,c}N_{sbg,w}$ =				
work as a group - applicable	=	(1+s/ 6c) x $\phi_{t,c}$ N_{sb}	= 78.4	[kips]	D.5.4.2 (D-18)
work individually - not applicable	=	n_{bw} x $\phi_{t,c}$ N_{sb} x [1+(c_2 or c_4) / c] / 4	= 0.0	[kips]	D.5.4.1
Seismic design strength reduction	=	x 1.0 not applicable	= 78.4	[kips]	D.3.3.3
	ratio =	0.32	> N_{buw}	**OK**	
Group side blowout resistance	$\phi_{t,c} N_{sbg}$ =	$\phi_{t,c} \dfrac{N_{sbgr,w}}{n_{T1}} n_t$	= 78.4	[kips]	
Govern Tensile Resistance	$\mathbf{N_r}$ =	$\phi_{t,c}$ min ($n_t N_s$, N_{rb}, $n_t N_{cp}$, N_{sbg})	= **71.0**	[kips]	

Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear — **Code Reference**

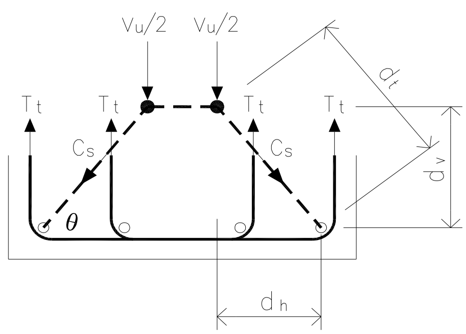
ACI 318-08

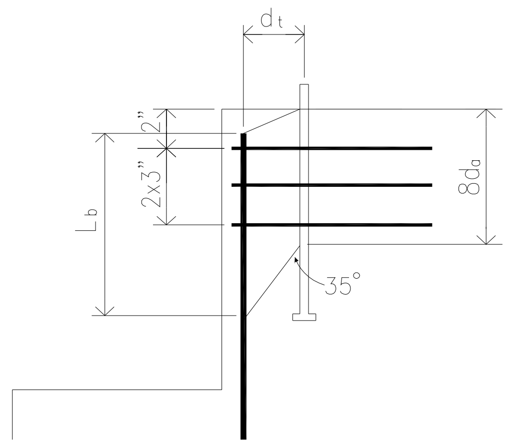
Anchor Rod Shear Resistance	$\phi_{v,s} V_{sa} = \phi_{v,s}\, n_s\, 0.6\, A_{se}\, f_{uta}$	= 43.8	[kips]	D.6.1.2 (b) (D-20)
Reduction due to built-up grout pads	= x 0.8 , applicable	= 35.1	[kips]	D.6.1.3
	ratio = 0.71	> V_u	**OK**	

Anchor Reinft Shear Breakout Resistance

Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft

STM strength reduction factor ϕ_{st} = 0.75 — 9.3.2.6





Strut-and-Tie model geometry	d_v = 2.250	[in]	d_h = 2.250	[in]	
	θ = 45		d_t = 3.182	[in]	
Strut compression force	C_s = 0.5 V_u / sinθ		= 17.7	[kips]	

ACI 318-08

Strut Bearing Strength				
Strut compressive strength	$f_{ce} = 0.85\, f'_c$	= 3.4	[ksi]	A.3.2 (A-3)
* Bearing of anchor bolt				
Anchor bearing length	$l_e = \min(8d_a, h_{ef})$	= 10.0	[in]	D.6.2.2
Anchor bearing area	$A_{brg} = l_e \times d_a$	= 12.5	[in^2]	
Anchor bearing resistance	$C_r = n_s \times \phi_{st} \times f_{ce} \times A_{brg}$	= 63.8	[kips]	
		> V_u	**OK**	
* Bearing of ver reinft bar				
Ver bar bearing area	$A_{brg} = (l_e + 1.5 \times d_t - d_a/2 - d_b/2) \times d_b$	= 13.6	[in^2]	
Ver bar bearing resistance	$C_r = \phi_{st} \times f_{ce} \times A_{brg}$	= 34.8	[kips]	
	ratio = 0.51	> C_s	**OK**	

Code Reference
ACI 318-08

Tie Reinforcement

* For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
* For enclosed tie, at hook location the tie cannot develop full yield strength f_y . Use the pullout resistance in tension of a single hooked bolt as per ACI318-08 Eq. (D-16) as the max force can be developed at hook T_h
* Assume 100% of hor. tie bars can develop full yield strength.

Total number of hor tie bar	$n = n_{leg}$ (leg) x n_{lay} (layer)	= 4			
Pull out resistance at hook	$T_h = \phi_{t,c}\ 0.9\ f_c'\ e_h\ d_a$	= 3.0	[kips]		D.5.3.5 (D-16)
	$e_h = 4.5\ d_b$	= 2.250	[in]		
Single tie bar tension resistance	$T_r = \phi_s \times f_y \times A_s$	= 9.0	[kips]		
Total tie bar tension resistance	$V_{rb} = 1.0 \times n \times Tr$	= 36.0	[kips]		
Seismic design strength reduction	= x 1.0 not applicable	= 36.0	[kips]		D.3.3.3
	ratio = 0.69	> V_u		OK	

Conc. Pryout Shear Resistance

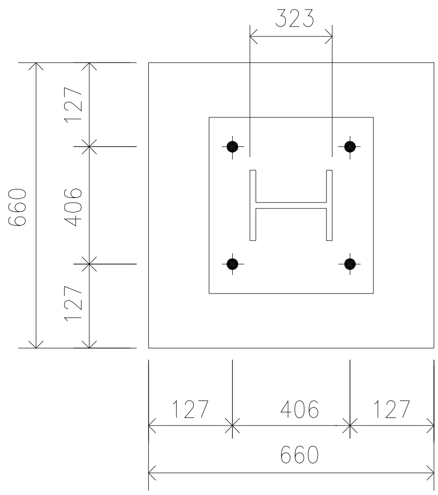
The pryout failure is only critical for short and stiff anchors. It is reasonable to assume that for general cast-in place headed anchors with $h_{ef} >= 12d_a$, the pryout failure will not govern

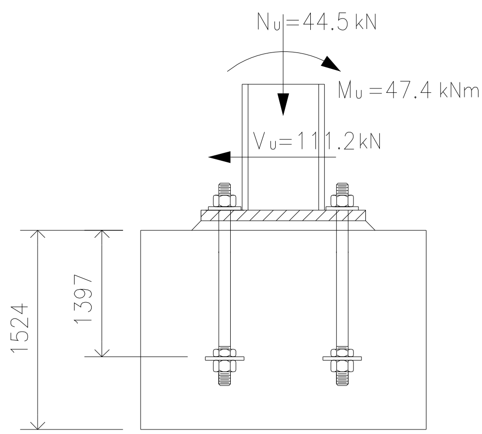
	$12d_a = 15.0$ [in]	$h_{ef} = 55.0$	[in]		
		> $12d_a$		OK	
Govern Shear Resistance	$\mathbf{V_r} = \min(\phi_{v,s}V_{sa}, V_{rb})$	= **35.1**	[kips]		

Tension Shear Interaction

Check if $N_u > 0.2\phi\ N_n$ and $V_u > 0.2\phi\ V_n$	Yes				D.7.1 & D.7.2
	$N_u / \phi\ N_n + V_u / \phi\ V_n$	= 1.06			D.7.3 (D-32)
	ratio = 0.89	< 1.2		OK	
Ductility Tension	$\phi_{t,s}\ N_{sa} = 42.2$ [kips]				
	> $\phi_{t,c}\ \min(N_{rb}, N_{pn}, N_{sbg})$	= 40.7	[kips]		
	Non-ductile				
Ductility Shear	$\phi_{v,s}\ V_{sa} = 35.1$ [kips]				
	< V_{rb}	= 36.0	[kips]		
	Ductile				

Example 04: Anchor Bolt + Anchor Reinft + Tension Shear & Moment + CSA A23.3-04 Code





M_u = 47.4 kNm	N_u= -44.5 kN (Compression)		V_u = 111.2 kN		
Concrete	f_c'= 27.6 MPa	Rebar	f_y = 414 MPa		
Pedestal size	660mm x 660mm				
Anchor bolt	F1554 Grade 36	1.25" dia	Hex Head	h_{ef} = 1397mm	h_a =1524mm

Seismic design $I_E\,F_aS_a(0.2) < 0.35$

Anchor reinforcement Tension → 2-25M ver. bar

Shear → 2-layer, 2-leg 15M hor. bar

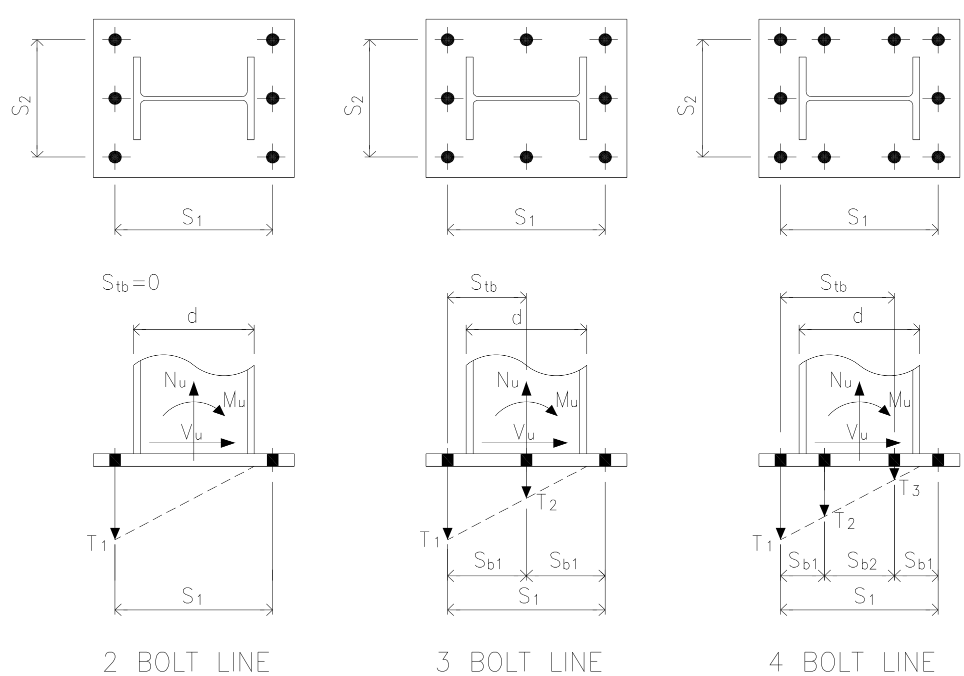
Provide built-up grout pad

ANCHOR BOLT DESIGN **Combined Tension, Shear and Moment**

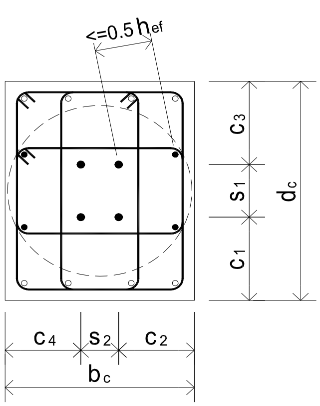
Anchor bolt design based on	**Code Abbreviation**
CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D	*A23.3-04 (R2010)*
ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary	*ACI318 M-08*
PIP STE05121 Anchor Bolt Design Guide-2006	*PIP STE05121*
	Code Reference
Assumptions	
1. Concrete is cracked	*A23.3-04 (R2010)*
2. Condition A - supplementary reinforcement is provided	D.5.4 (c)
3. Anchor reinft strength is used to replace concrete tension / shear breakout strength as per ACI318 M-08 Appendix D clause D.5.2.9 and D.6.2.9	*ACI318 M-08* D.5.2.9 & D.6.2.9
4. For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective	
5. Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft	*A23.3-04 (R2010)*
6. For anchor group subject to moment, the anchor tensile load is designed using elastic analysis and there is no redistribution of the forces between highly stressed and less stressed anchors	D.4.1
7. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output	
8. Shear carried by only half of total anchor bolts due to oversized holes in column base plate	*AISC Design Guide 1* section 3.5.3

Anchor Bolt Data

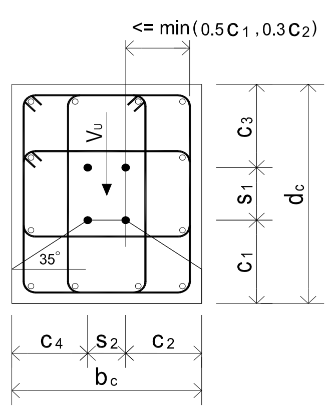
Factored moment	M_u =	47.4	[kNm]		= 35.0	[kip-ft]
Factored tension /compression	N_u =	-44.5	[kN]	in compression	= -10.0	[kips]
Factored shear	V_u =	111.2	[kN]		= 25.0	[kips]
Factored shear for design	$\mathbf{V_u}$ =	**111.2**	[kN]	V_u = 0 if shear key is provided		



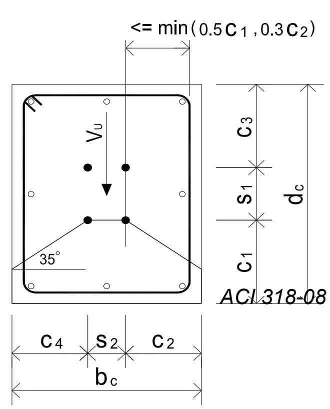
								Code Reference
No of bolt line for resisting moment		=	2 Bolt Line					
No of bolt along outermost bolt line		=	2					
					min required			
Outermost bolt line spacing s_1	s_1	=	406	[mm]	127	**OK**		*PIP STE05121*
Outermost bolt line spacing s_2	s_2	=	406	[mm]	127	**OK**		Page A -1 Table 1
Internal bolt line spacing s_{b1}	s_{b1}	=	267	[mm]	127	**OK**		
Internal bolt line spacing s_{b2}	s_{b2}	=	0	[mm]	127	**OK**		
Column depth	d	=	323	[mm]				
Concrete strength	f'_c	=	28	[MPa]		= 4.0	[ksi]	
Anchor bolt material		=	F1554 Grade 36					
Anchor tensile strength	f_{uta}	=	58	[ksi]		= 400	[MPa]	*A23.3-04 (R2010)*
			Anchor is ductile steel element					D.2
Anchor bolt diameter	d_a	=	**1.25**	[in]		= 31.8	[mm]	*PIP STE05121*
Bolt sleeve diameter	d_s	=	76	[mm]				Page A -1 Table 1
Bolt sleeve height	h_s	=	254	[mm]				
					min required			
Anchor bolt embedment depth	h_{ef}	=	1397	[mm]	381	**OK**		Page A -1 Table 1
Pedestal height	h	=	1524	[mm]	1473	**OK**		
Pedestal width	b_c	=	660	[mm]				
Pedestal depth	d_c	=	660	[mm]				
Bolt edge distance c_1	c_1	=	127	[mm]	127	**OK**		Page A -1 Table 1
Bolt edge distance c_2	c_2	=	127	[mm]	127	**OK**		
Bolt edge distance c_3	c_3	=	127	[mm]	127	**OK**		
Bolt edge distance c_4	c_4	=	127	[mm]	127	**OK**		



Ver. Reinft For Tension



Hor. Ties For Shear - 4 Legs



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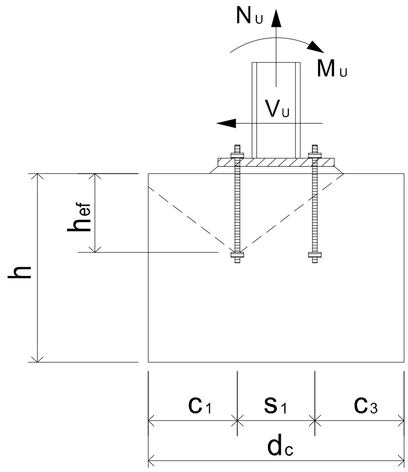
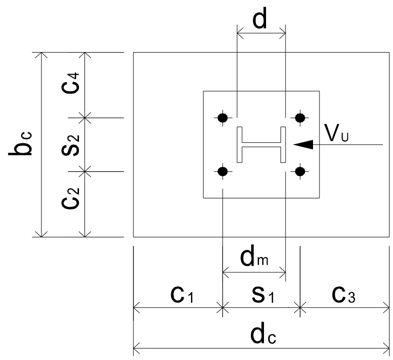
Hor. Ties For Shear - 2 Legs

Code Reference





ACI318 M-08

To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within $0.5h_{ef}$ from the outmost anchor's centerline. In this design $0.5h_{ef}$ value is limited to 200mm. — RD.5.2.9

$0.5h_{ef}$ = **200** [mm]

No of ver. rebar that are effective for resisting anchor tension — n_v = 2

Ver. bar size — d_b = 25 — single bar area A_s = 500 [mm^2]

To be considered effective for resisting anchor shear, hor. reinft shall be located within min($0.5c_1$, $0.3c_2$) from the outmost anchor's centerline — RD.6.2.9

min($0.5c_1$, $0.3c_2$) = **38** [mm]

No of tie <u>leg</u> that are effective to resist anchor shear — n_{leg} = 2 ?

No of tie <u>layer</u> that are effective to resist anchor shear — n_{lay} = 2 ?

Hor. bar size — d_b = 15 — single bar area A_s = 200 [mm^2]

For anchor reinft shear breakout strength calc — 100% hor. tie bars develop full yield strength ?

Rebar yield strength — f_y = 414 [MPa] — suggest 400 — = 60.0 [ksi]

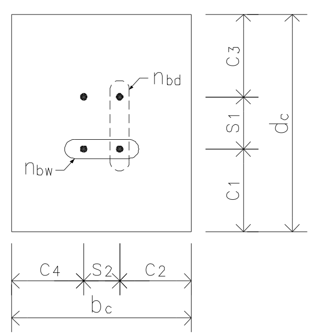
Total no of anchor bolt — n = 4

No of bolt carrying tension — n_t = 2

No of bolt carrying shear — n_s = 2

For side-face blowout check use

No of bolt along width edge — n_{bw} = 2



Bolt No Input for Side-Face Blowout Check Use

Anchor head type = Hex ?

A_{se} = 625 [mm^2]

Bearing area of head — A_{brg} = **1172** [mm^2]

A_{brg} [mm^2] not applicable

A23.3-04 (R2010)

Bolt 1/8" (3mm) corrosion allowance = No ?

Provide shear key ? = No ?

Seismic region where $I_E F_a S_a(0.2)$>=0.35 = No ? — D.4.3.5

Provide built-up grout pad ? = Yes ? — D.7.1.3

					Code Reference
Strength reduction factors					*A23.3-04 (R2010)*
Anchor reinforcement factor	ϕ_{as} = 0.75				D.7.2.9
Steel anchor resistance factor	ϕ_s = 0.85				8.4.3 (a)
Concrete resistance factor	ϕ_c = 0.65				8.4.2
Resistance modification factors					
Anchor rod - ductile steel	$R_{t,s}$ = 0.80		$R_{v,s}$ = 0.75		D.5.4(a)
Concrete - condition A	$R_{t,c}$ = 1.15		$R_{v,c}$ = 1.15		D.5.4(c)
CONCLUSION					
Abchor Rod Embedment, Spacing and Edge Distance				**OK**	
Min Rquired Anchor Reinft. Development Length			ratio = 0.25	**OK**	12.2.1
Overall			ratio = **0.90**	**OK**	
Tension					
Anchor Rod Tensile Resistance			ratio = 0.32	**OK**	
Anchor Reinft Tensile Breakout Resistance			ratio = 0.36	**OK**	
Anchor Pullout Resistance			ratio = 0.33	**OK**	
Side Blowout Resistance			ratio = 0.32	**OK**	
Shear					
Anchor Rod Shear Resistance			ratio = 0.73	**OK**	
Anchor Reinft Shear Breakout Resistance					
Strut Bearing Strength			ratio = 0.52	**OK**	
Tie Reinforcement			ratio = 0.45	**OK**	
Conc. Pryout Not Govern When h_{ef} >= $12d_a$				**OK**	
Anchor Rod on Conc Bearing			ratio = 0.27	**OK**	
Tension Shear Interaction					
Tension Shear Interaction			ratio = 0.90	**OK**	
Ductility					*A23.3-04 (R2010)*
	Tension	**Non-ductile**	**Shear**	**Ductile**	
Seismic Design Requirement				**OK**	D.4.3.6

IeFaSa(0.2)<0.35, A23.3-04 D.4.3.3 ductility requirement is NOT required

CACULATION

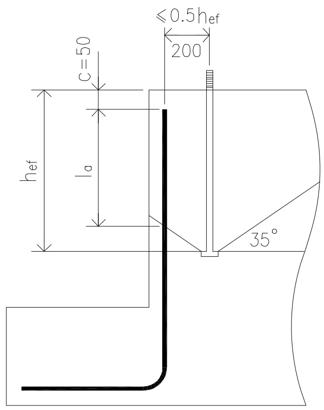
Anchor Tensile Force

Single bolt tensile force	T_1 = 55.2	[kN]	No of bolt for T_1 n_{T1} = 2		
	T_2 = 0.0	[kN]	No of bolt for T_2 n_{T2} = 0		
	T_3 = 0.0	[kN]	No of bolt for T_3 n_{T3} = 0		
Sum of bolt tensile force	$\mathbf{N_u} = \Sigma n_i T_i$		= **110.3**	[kN]	
Anchor Rod Tensile	$N_{sr} = A_{se} \phi_s f_{uta} R_{t,s}$		= 170.0	[kN]	D.6.1.2 (D-3)
Resistance	ratio = 0.32		> T_1	**OK**	

Anchor Reinft Tensile Breakout Resistance

Min tension development length	l_d =	= 887	[mm]	12.2.3
for ver. 25M bar				

						Code Reference
Actual development lenngth	l_a	= h_{ef} - c (50mm) - 200mm x tan35	= 1207	[mm]		*A23.3-04 (R2010)*
			> 300		**OK**	12.2.1



	N_{rbr}	= ϕ_{as} x f_y x n_v x A_s x (l_a / l_d , if l_a < l_d)	= 310.5	[kN]		12.2.5
Seismic design strength reduction		= x 1.0 not applicable	= 310.5	[kN]		D.4.3.5
	ratio	= 0.36	> N_u		**OK**	

Anchor Pullout Resistance

Single bolt pullout resistance	N_{pr}	= 8 A_{brg} ϕ_c f_c' $R_{t,c}$	= 168.2	[kN]		D.6.3.4 (D-16)
	N_{cpr}	= $\Psi_{c,p}$ N_{pr}	= 168.2	[kN]		D.6.3.1 (D-15)
Seismic design strength reduction		= x 1.0 not applicable	= 168.2	[kN]		D.4.3.5
	ratio	= 0.33	> T_1		**OK**	
	$\Psi_{c,p}$	= 1 for cracked conc				D.6.3.6
	$R_{t,c}$	= 1.00 pullout strength is always Condition B				D.5.4(c)

Side Blowout Resistance

Failure Along Pedestal Width Edge						*ACI318 M-08*
Tensile load carried by anchors close to edge which may cause side-face blowout						
along pedestal width edge	N_{buw}	= n_{T1} T_1	= 110.3	[kN]		RD.5.4.2
	c	= min (c_1, c_3)	= 127	[mm]		
Check if side blowout applicable	h_{ef}	= 1397 [mm]				*A23.3-04 (R2010)*
		> 2.5c side bowout is applicable				D.6.4.1
Check if edge anchors work as a	s_{22}	= 406 [mm]	s = s_2 = 406	[mm]		
a group or work individually		< 6c edge anchors work as a group				D.6.4.2
Single anchor SB resistance	$N_{sbr,w}$	= $13.3c\sqrt{A_{brg}}\ \phi_c\ \sqrt{f'_c}\ R_{t,c}$	= 227.1	[kN]		D.6.4.1 (D-18)
Multiple anchors SB resistance	$N_{sbgr,w}$	=				
work as a group - applicable		= (1+s/ 6c) x $N_{sbr,w}$	= 348.1	[kN]		D.6.4.2 (D-19)
work individually - not applicable		= n_{bw} x $N_{sbr,w}$ x [1+(c_2 or c_4) / c] / 4	= 0.0	[kN]		D.6.4.1
Seismic design strength reduction		= x 1.0 not applicable	= 348.1	[kN]		D.4.3.5
	ratio	= 0.32	> N_{buw}		**OK**	
Group side blowout resistance	N_{sbgr}	= $\dfrac{N_{sbgr,w}}{n_{bw}} n_t$	= 348.1	[kN]		
Govern Tensile Resistance	$\mathbf{N_r}$	= min (n_t N_{sr}, N_{rbr}, n_t N_{cpr}, N_{sbgr})	= **310.5**	[kN]		

Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear — **Code Reference**

A23.3-04 (R2010)

Anchor Rod Shear Resistance $V_{sr} = n_s A_{se} \phi_s 0.6 f_{uta} R_{v,s}$ = 191.2 [kN] D.7.1.2 (b) (D-21)

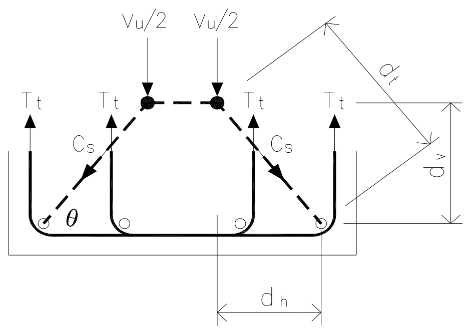
Reduction due to built-up grout pads = x 0.8 , applicable = 153.0 [kN] D.7.1.3

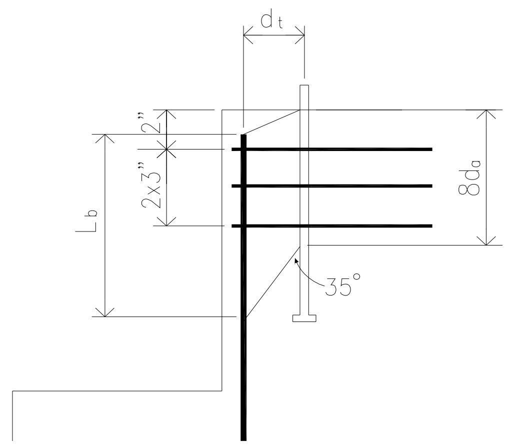
ratio = 0.73 > V_u **OK**

Anchor Reinft Shear Breakout Resistance *ACI318 M-08*

Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft

STM strength reduction factor ϕ_{st} = 0.75 — 9.3.2.6





Strut-and-Tie model geometry d_v = 57 [mm] d_h = 57 [mm]

θ = 45 d_t = 81 [mm]

Strut compression force $C_s = 0.5 V_u / \sin\theta$ = 78.6 [kN]

ACI318 M-08

Strut Bearing Strength

Strut compressive strength $f_{ce} = 0.85 f'_c$ = 23.5 [MPa] A.3.2 (A-3)

* Bearing of anchor bolt

Anchor bearing length $l_e = \min(8d_a, h_{ef})$ = 254 [mm] D.6.2.2

Anchor bearing area $A_{brg} = l_e \times d_a$ = 8065 [mm^2]

Anchor bearing resistance $C_r = n_s \times \phi_{st} \times f_{ce} \times A_{brg}$ = 283.8 [kN]

> V_u **OK**

* Bearing of ver reinft bar

Ver bar bearing area $A_{brg} = (l_e + 1.5 \times d_t - d_a/2 - d_b/2) \times d_b$ = 8664 [mm^2]

Ver bar bearing resistance $C_r = \phi_{st} \times f_{ce} \times A_{brg}$ = 152.4 [kN]

ratio = 0.52 > C_s **OK**

Code Reference

Tie Reinforcement

* For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
* For enclosed tie, at hook location the tie cannot develop full yield strength f_y . Use the pullout resistance in tension of a single J-bolt as per A23.3-04 Annex D Eq. (D-17) as the max force can be developed at hook T_h
* Assume 100% of hor. tie bars can develop full yield strength.

					A23.3-04 (R2010)
Total number of hor tie bar	n = n_{leg} (leg) x n_{lay} (layer)	= 4			
Pull out resistance at hook	T_h = $0.9\ \phi_c\ f_c'\ e_h\ d_b\ R_{t,c}$	= 16.3	[kN]		D.6.3.5 (D-17)
	e_h = $4.5\ d_b$	= 68	[mm]		
Single tie bar tension resistance	T_r = ϕ_{as} x f_y x A_s	= 62.1	[kN]		
Total tie bar tension resistance	V_{rbr} = 1.0 x n x Tr	= 248.4	[kN]		
Seismic design strength reduction	= x 1.0 not applicable	= 248.4	[kN]		D.4.3.5
	ratio = 0.45	> V_u		OK	

Conc. Pryout Shear Resistance

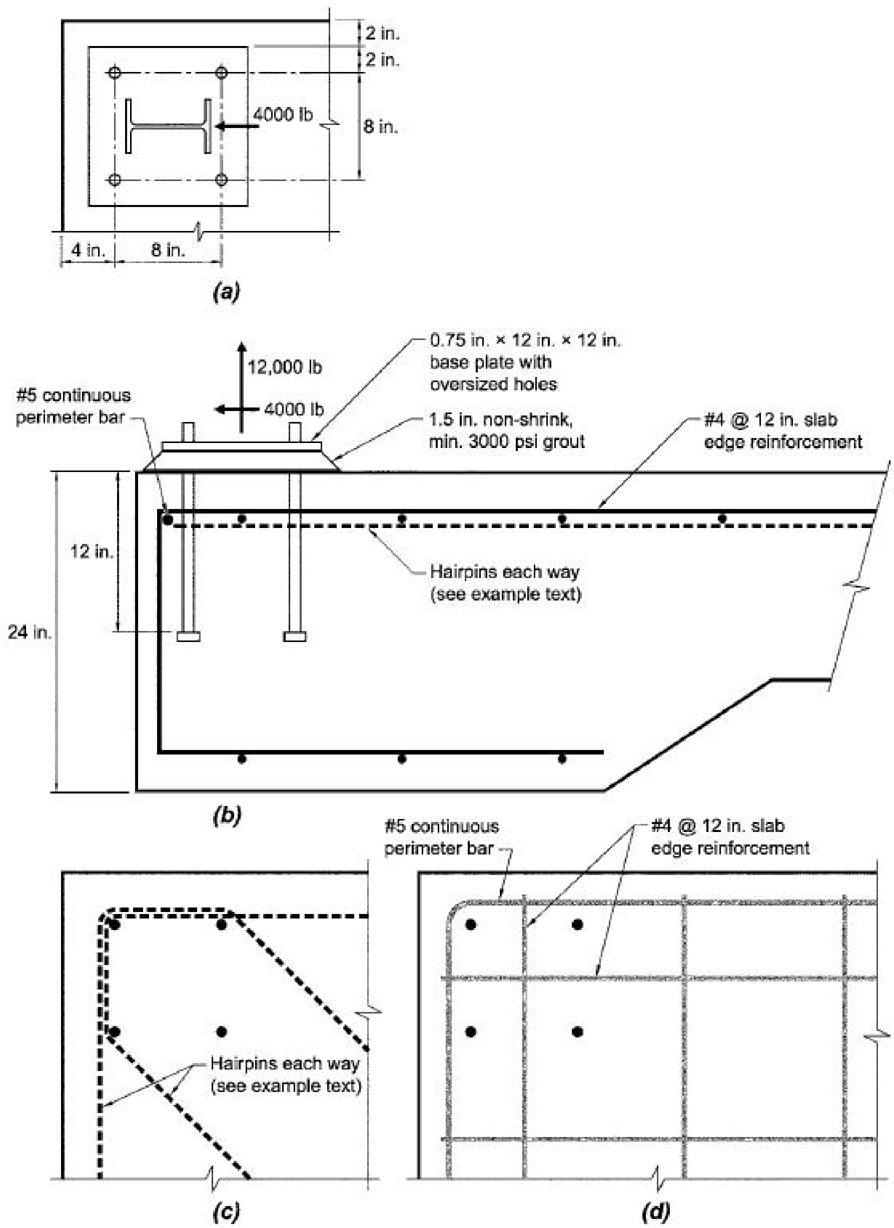
The pryout failure is only critical for short and stiff anchors. It is reasonable to assume that for general cast-in place headed anchors with h_{ef} > = $12d_a$, the pryout failure will not govern

	$12d_a$ = 381 [mm]	h_{ef} = 1397	[mm]		
		> $12d_a$		OK	*CSA S16-09*
Anchor Rod on Conc Bearing	B_r = n_s x 1.4 x ϕ_c x min($8d_a$, h_{ef}) x d_a x f_c'	= 405.1	[kN]		25.3.3.2
	ratio = 0.27	< V_u		OK	
Govern Shear Resistance	$\mathbf{V_r}$ = min (V_{sr} , V_{rbr} , B_r)	= **153.0**	[kN]		

Tension Shear Interaction					*A23.3-04 (R2010)*
Check if N_u >0.2 N_r and V_u >0.2 V_r	Yes				D.8.2 & D.8.3
	$N_u/N_r + V_u/V_r$	= 1.08			D.8.4 (D-35)
	ratio = 0.90	< 1.2		OK	
Ductility Tension	N_{sr} = 170.0 [kN]				
	> min (N_{rbr} , N_{cpr}, N_{sbgr})	= 168.2	[kN]		
	Non-ductile				
Ductility Shear	V_{sr} = 153.0 [kN]				
	< min (V_{rbr}, B_r)	= 248.4	[kN]		
	Ductile				

Example 11: Anchor Bolt + No Anchor Reinft + Tension & Shear + ACI 318-08 Code

This example taken from Example 8 on page 71 of *ACI 355.3R-11 Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D*



N_u = 12 kips (tension), V_u=4 kips, f_c' = 3 ksi

Anchor bolt d_a=3/4 in ASTM F1554 Grade 55 h_{ef} =12 in h_a=24 in Anchor head → Hex

Supplementary reinforcement Tension → Condition B Shear → Condition A $\Psi_{c,V}$ =1.2

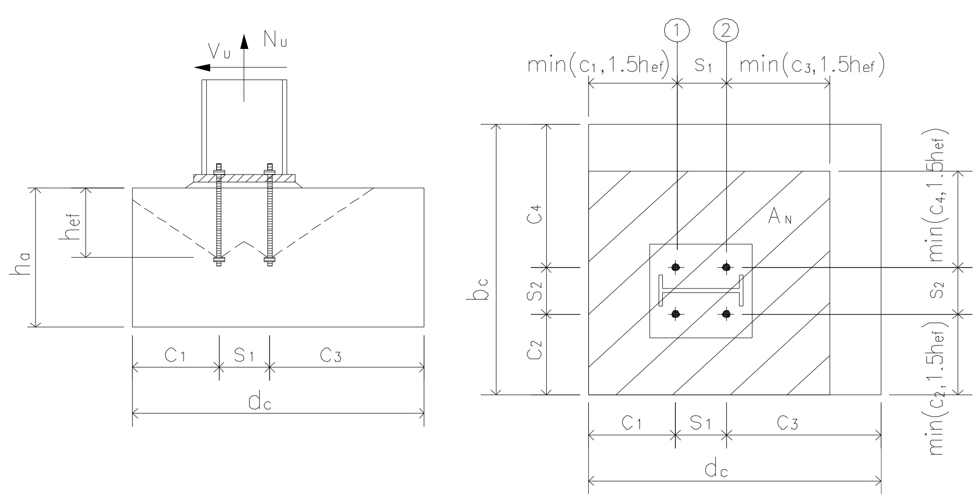
Provide built-up grout pad Seismic is not a consideration

Field welded plate washers to base plate at each anchor

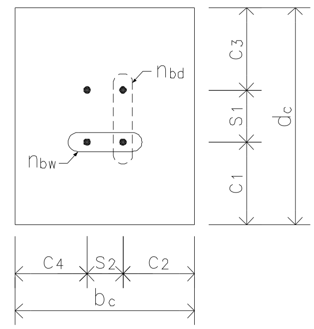
ANCHOR BOLT DESIGN **Combined Tension and Shear**

Anchor bolt design based on						**Code Abbreviation**
ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D						*ACI 318-08*
PIP STE05121 Anchor Bolt Design Guide-2006						*PIP STE05121*

Anchor Bolt Data		set N_u = 0 if it's compression					**Code Reference**
Factored tension for design	$\mathbf{N_u}$ =	**12.0**	[kips]	= 53.4		[kN]	
Factored shear	V_u =	4.0	[kips]	= 17.8		[kN]	
Factored shear for design	$\mathbf{V_u}$ =	**4.0**	[kips]	V_u = 0 if shear key is provided			
Concrete strength	f'_c =	3.0	[ksi]	= 20.7		[MPa]	
Anchor bolt material		F1554 Grade 55					
Anchor tensile strength	f_{uta} =	75	[ksi]	= 517		[MPa]	*ACI 318-08*
		Anchor is ductile steel element					D.1
Anchor bolt diameter	$\mathbf{d_a}$ =	**0.75**	[in]	= 19.1		[mm]	*PIP STE05121*
Bolt sleeve diameter	d_s =	2.0	[in]				Page A -1 Table 1
Bolt sleeve height	h_s =	7.0	[in]				
				min required			
Anchor bolt embedment depth	h_{ef} =	12.0	[in]	9.0	**OK**		Page A -1 Table 1
Concrete thickness	h_a =	24.0	[in]	15.0	**OK**		
Bolt edge distance c_1	c_1 =	4.0	[in]	4.5	**Warn**		Page A -1 Table 1
Bolt edge distance c_2	c_2 =	4.0	[in]	4.5	**Warn**		
Bolt edge distance c_3	c_3 =	100.0	[in]	4.5	**OK**		
Bolt edge distance c_4	c_4 =	100.0	[in]	4.5	**OK**		*ACI 318-08*
$c_i > 1.5h_{ef}$ for at least two edges to avoid reducing of h_{ef} when $N_u > 0$					**Yes**		D.5.2.3
Adjusted h_{ef} for design	h_{ef} =	**12.00**	[in]	9.0	**OK**		D.5.2.3
Outermost bolt line spacing s_1	s_1 =	8.0	[in]	3.0	**OK**		*PIP STE05121*
Outermost bolt line spacing s_2	s_2 =	8.0	[in]	3.0	**OK**		Page A -1 Table 1



Number of bolt at bolt line 1	n_1 = 2			
Number of bolt at bolt line 2	n_2 = 2			
Number of bolt carrying tension	n_t = 4			
Oversized holes in base plate ?	= No	?		
Number of bolt carrying shear	n_s = 4			
For side-face blowout check use				
No of bolt along width edge	n_{bw} = 2			
No of bolt along depth edge	n_{bd} = 2			

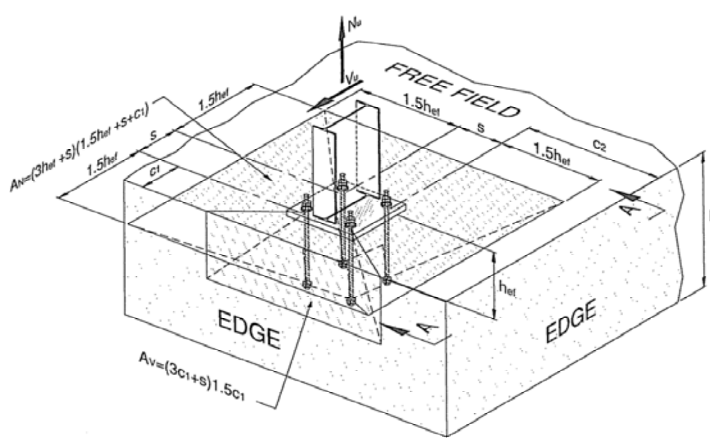


Bolt No Input for Side-Face Blowout Check Use

Anchor head type	= Hex		?		
Anchor effective cross sect area	A_{se} = 0.334	[in^2]			
Bearing area of head	A_{brg} = **0.654**	[in^2]			
	A_{brg}	[in^2]	not applicable		
Bolt 1/8" (3mm) corrosion allowance	No	?			**Code Reference**
Provide shear key ?	No	?			*ACI 318-08*
Seismic design category >= C	No	?			D.3.3.3
Supplementary reinforcement					
For tension	No	Condition B			D.4.4 (c)
For shear	$\Psi_{c,V}$ = 1.2	Condition A	?		D.6.2.7
Provide built-up grout pad ?	Yes	?			D.6.1.3
Strength reduction factors					
Anchor reinforcement	ϕ_s = 0.75				D.5.2.9 & D.6.2.9
Anchor rod - ductile steel	$\phi_{t,s}$ = 0.75		$\phi_{v,s}$ = 0.65		D.4.4 (a)
Concrete	$\phi_{t,c}$ = 0.70	Cdn-B	$\phi_{v,c}$ = 0.75	Cdn-A	D.4.4 (c)

Assumptions

1. Concrete is cracked	
2. Condition B - no supplementary reinforcement provided	D.4.4 (c)
3. Load combinations shall be per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2	D.4.4
4. Tensile load acts through center of bolt group $\Psi_{ec,N}$ =1.0	D.5.2.4
5. Shear load acts through center of bolt group $\Psi_{ec,V}$ =1.0	D.6.2.5
6. Anchor bolt washer shall be tack welded to base plate for all anchor bolts to transfer shear	*AISC Design Guide 1* section 3.5.3



CONCLUSION

Abchor Rod Embedment, Spacing and Edge Distance			**Warn**
Overall		ratio = **0.83**	**OK**
Tension			
Anchor Rod Tensile Resistance		ratio = 0.16	**OK**
Conc. Tensile Breakout Resistance		ratio = 0.58	**OK**
Anchor Pullout Resistance		ratio = 0.27	**OK**
Side Blowout Resistance		ratio = 0.23	**OK**
Shear			
Anchor Rod Shear Resistance		ratio = 0.13	**OK**
Conc. Shear Breakout Resistance		ratio = 0.41	**OK**
Conc. Pryout Shear Resistance		ratio = 0.10	**OK**
Tension Shear Interaction			
Tension Shear Interaction		ratio = 0.83	**OK**
Ductility			
	Tension **Non-ductile**	**Shear**	**Non-ductile**
Seismic Design Requirement			**OK** D.3.3.4

SDC< C, ACI318-08 D.3.3 ductility requirement is NOT required

CALCULATION — **Code Reference** *ACI 318-08*

Anchor Rod Tensile Resistance

$\phi_{t,s} N_{sa} = \phi_{t,s}\, n_t\, A_{se}\, f_{uta}$ = 75.2 [kips] — D.5.1.2 (D-3)

ratio = 0.16 > N_u **OK**

Conc. Tensile Breakout Resistance

$N_b = 24\ \lambda \sqrt{f'_c}\ h_{ef}^{1.5}$ if $h_{ef} < 11"$ or $h_{ef} > 25"$ = 55.1 [kips] — D.5.2.2 (D-7)

$16\ \lambda \sqrt{f'_c}\ h_{ef}^{5/3}$ if $11" \le h_{ef} \le 25"$ — D.5.2.2 (D-8)

Projected conc failure area: $1.5h_{ef}$ = = 18.00 [in]

A_{Nc} = $[s_1+\min(c_1,1.5h_{ef})+\min(c_3,1.5h_{ef})]\times[s_2+\min(c_2,1.5h_{ef})+\min(c_4,1.5h_{ef})]$ = 900.0 [in^2]

$A_{Nco} = 9\ h_{ef}^2$ = 1296.0 [in^2] — D.5.2.1 (D-6)

$A_{Nc} = \min(A_{Nc},\ n_t\, A_{Nco})$ = 900.0 [in^2] — D.5.2.1

Min edge distance: $c_{min} = \min(c_1, c_2, c_3, c_4)$ = 4.0 [in]

Eccentricity effects: $\Psi_{ec,N}$ = 1.0 for no eccentric load — D.5.2.4

Edge effects: $\Psi_{ed,N} = \min[(0.7+0.3c_{min}/1.5h_{ef}),\ 1.0]$ = 0.77 — D.5.2.5

Concrete cracking: $\Psi_{c,N}$ = 1.0 for cracked concrete — D.5.2.6

Concrete splitting: $\Psi_{cp,N}$ = 1.0 for cast-in anchor — D.5.2.7

					Code Reference
					ACI 318-08
Concrete breakout resistance	$\phi_{t,c}\, N_{cbg}$	$= \phi_{t,c} \dfrac{A_{Nc}}{A_{Nco}} \Psi_{ec,N}\, \Psi_{ed,N}\, \Psi_{c,N}\, \Psi_{cp,N}\; N_b$	= 20.5	[kips]	D.5.2.1 (D-5)
Seismic design strength reduction		= x 1.0 not applicable	= 20.5	[kips]	D.3.3.3
	ratio	= 0.58	> N_u	**OK**	
Anchor Pullout Resistance					
Single bolt pullout resistance	N_p	$= 8\, A_{brg}\, f_c'$	= 15.7	[kips]	D.5.3.4 (D-15)
	$\phi_{t,c}\, N_{pn}$	$= \phi_{t,c}\, n_t\, \Psi_{c,p}\, N_p$	= 43.9	[kips]	D.5.3.1 (D-14)
Seismic design strength reduction		= x 1.0 not applicable	= 43.9	[kips]	D.3.3.3
	ratio	= 0.27	> N_u	**OK**	
	$\Psi_{c,p}$	= 1 for cracked conc			D.5.3.6
	$\phi_{t,c}$	= 0.70 pullout strength is always Condition B			D.4.4(c)
Side Blowout Resistance					
Failure Along Pedestal Width Edge					
Tensile load carried by anchors close to edge which may cause side-face blowout					
along pedestal width edge	N_{buw}	$= N_u \times n_{bw} / n_t$	= 6.0	[kips]	RD.5.4.2
	c	$= \min(c_1, c_3)$	= 4.0	[in]	
Check if side blowout applicable	h_{ef}	= 12.0 [in]			
		> 2.5c side bowout is applicable			D.5.4.1
Check if edge anchors work as a	s_{22}	= 8.0 [in] $s = s_2$	= 8.0	[in]	
a group or work individually		< 6c edge anchors work as a group			D.5.4.2
Single anchor SB resistance	$\phi_{t,c}\, N_{sb}$	$= \phi_{t,c} \left(160\, c \sqrt{A_{brg}}\right) \lambda \sqrt{f'_c}$	= 19.8	[kips]	D.5.4.1 (D-17)
Multiple anchors SB resistance	$\phi_{t,c} N_{sbg,w}$	=			
work as a group - applicable		$= (1+s/\,6c) \times \phi_{t,c}\, N_{sb}$	= 26.5	[kips]	D.5.4.2 (D-18)
work individually - not applicable		$= n_{bw} \times \phi_{t,c}\, N_{sb} \times [1+(c_2 \text{ or } c_4)/\,c] / 4$	= 0.0	[kips]	D.5.4.1
Seismic design strength reduction		= x 1.0 not applicable	= 26.5	[kips]	D.3.3.3
	ratio	= 0.23	> N_{buw}	**OK**	
Failure Along Pedestal Depth Edge					
Tensile load carried by anchors close to edge which may cause side-face blowout					
along pedestal depth edge	N_{bud}	$= N_u \times n_{bd} / n_t$	= 6.0	[kips]	RD.5.4.2
	c	$= \min(c_2, c_4)$	= 4.0	[in]	
Check if side blowout applicable	h_{ef}	= 12.0 [in]			
		> 2.5c side bowout is applicable			D.5.4.1
Check if edge anchors work as a	s_{11}	= 8.0 [in] $s = s_1$	= 8.0	[in]	
a group or work individually		< 6c edge anchors work as a group			D.5.4.2
Single anchor SB resistance	$\phi_{t,c}\, N_{sb}$	$= \phi_{t,c} \left(160\, c \sqrt{A_{brg}}\right) \lambda \sqrt{f'_c}$	= 19.8	[kips]	D.5.4.1 (D-17)
Multiple anchors SB resistance	$\phi_{t,c} N_{sbg,d}$	=			
work as a group - applicable		$= (1+s/\,6c) \times \phi_{t,c}\, N_{sb}$	= 26.5	[kips]	D.5.4.2 (D-18)
work individually - not applicable		$= n_{bd} \times \phi_{t,c}\, N_{sb} \times [1+(c_1 \text{ or } c_3)/\,c] / 4$	= 0.0	[kips]	D.5.4.1
Seismic design strength reduction		= x 1.0 not applicable	= 26.5	[kips]	D.3.3.3
	ratio	= 0.23	> N_{bud}	**OK**	

				Code Reference
Group side blowout resistance	$\phi_{t,c} N_{sbg} = \phi_{t,c} \min\left(\frac{N_{sbg,w}}{n_{bw}} n_t , \frac{N_{sbg,d}}{n_{bd}} n_t \right)$	= 52.9	[kips]	*ACI 318-08*
Govern Tensile Resistance	$\mathbf{N_r}$ = min [$\phi_{t,s}$ N_{sa}, $\phi_{t,c}$ (N_{cbg}, N_{pn}, N_{sbg})]	= **20.5**	[kips]	

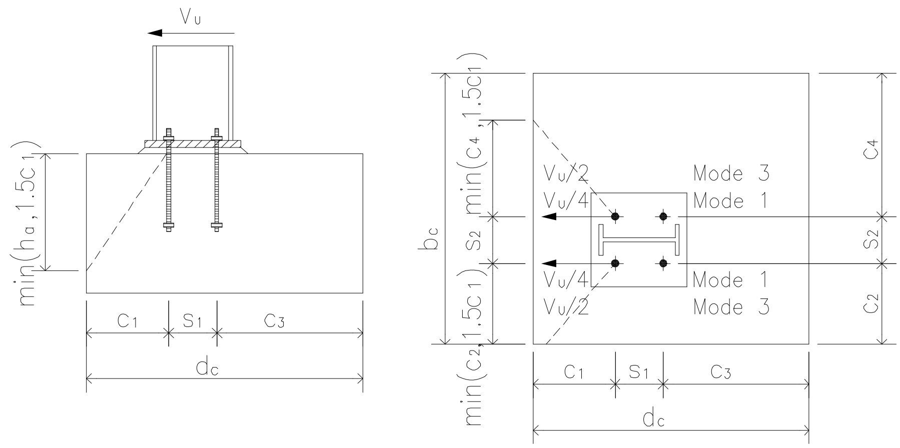
Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear

Anchor Rod Shear Resistance	$\phi_{v,s} V_{sa} = \phi_{v,s}\, n_s\, 0.6\, A_{se} f_{uta}$	= 39.1	[kips]	D.6.1.2 (b) (D-20)
Reduction due to built-up grout pads	= x 0.8 , applicable	= 31.3	[kips]	D.6.1.3
	ratio = 0.13	> V_u	**OK**	

Conc. Shear Breakout Resistance

Mode 1 Failure cone at front anchors, strength check against 0.5 x V_u

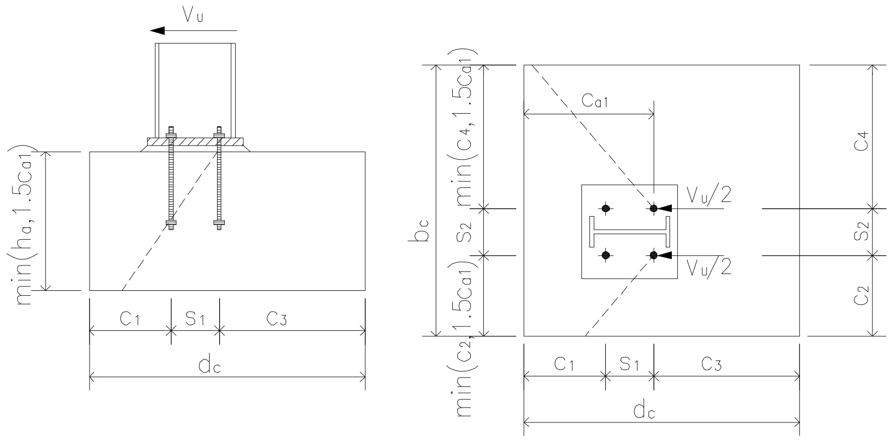
Mode 3 Failure cone at front anchors, strength check against 1.0 x V_u , applicable when oversized holes are used in base plate



Bolt edge distance	c_1 =	= 4.0	[in]	
Limiting c_{a1} when anchors are influenced by 3 or more edges		= No		D.6.2.4
Bolt edge distance - adjusted	$\mathbf{c_1}$ = ca1 needs NOT to be adjusted	= **4.0**	[in]	D.6.2.4
	c_2 =	= 4.0	[in]	
	$1.5c_1$ =	= 6.0	[in]	
	A_{Vc} = [min(c_2,1.5c_1) + s_2 + min(c_4,1.5c_1)] x min(1.5c_1, h_a)	= 108.0	[in^2]	D.6.2.1
	$A_{Vco} = 4.5c_1^2$	= 72.0	[in^2]	D.6.2.1 (D-23)
	A_{Vc} = min (A_{Vc}, n_1 A_{Vco})	= 108.0	[in^2]	D.6.2.1
	l_e = min($8d_a$, h_{ef})	= 6.0	[in]	D.6.2.2
	$V_b = \left[7 \left(\frac{l_e}{d_a} \right)^{0.2} \sqrt{d_a} \right] \lambda \sqrt{f'_c}\; c_1^{1.5}$	= 4.0	[kips]	D.6.2.2 (D-24)
Eccentricity effects	$\Psi_{ec,v}$ = 1.0 shear acts through center of group			D.6.2.5
Edge effects	$\Psi_{ed,v}$ = min[(0.7+0.3c_2/1.5c_1), 1.0]	= 0.90		D.6.2.6
Concrete cracking	$\Psi_{c,v}$ =	= 1.20		D.6.2.7
Member thickness	$\Psi_{h,v}$ = max[(sqrt(1.5c_1 / h_a) , 1.0]	= 1.00		D.6.2.8

					Code Reference
Conc shear breakout					*ACI 318-08*
resistance	V_{cbg1}	$= \phi_{v,c} \dfrac{A_{Vc}}{A_{Vco}} \Psi_{ec,V}\ \Psi_{ed,V}\ \Psi_{c,V}\ \Psi_{h,V}\ V_b$	= 4.9	[kips]	D.6.2.1 (D-22)
Mode 1 is used for checking	$\mathbf{V_{cbg1}}$	$= V_{cbg1}$ x 2.0	= **9.8**	[kips]	

Mode 2 Failure cone at back anchors

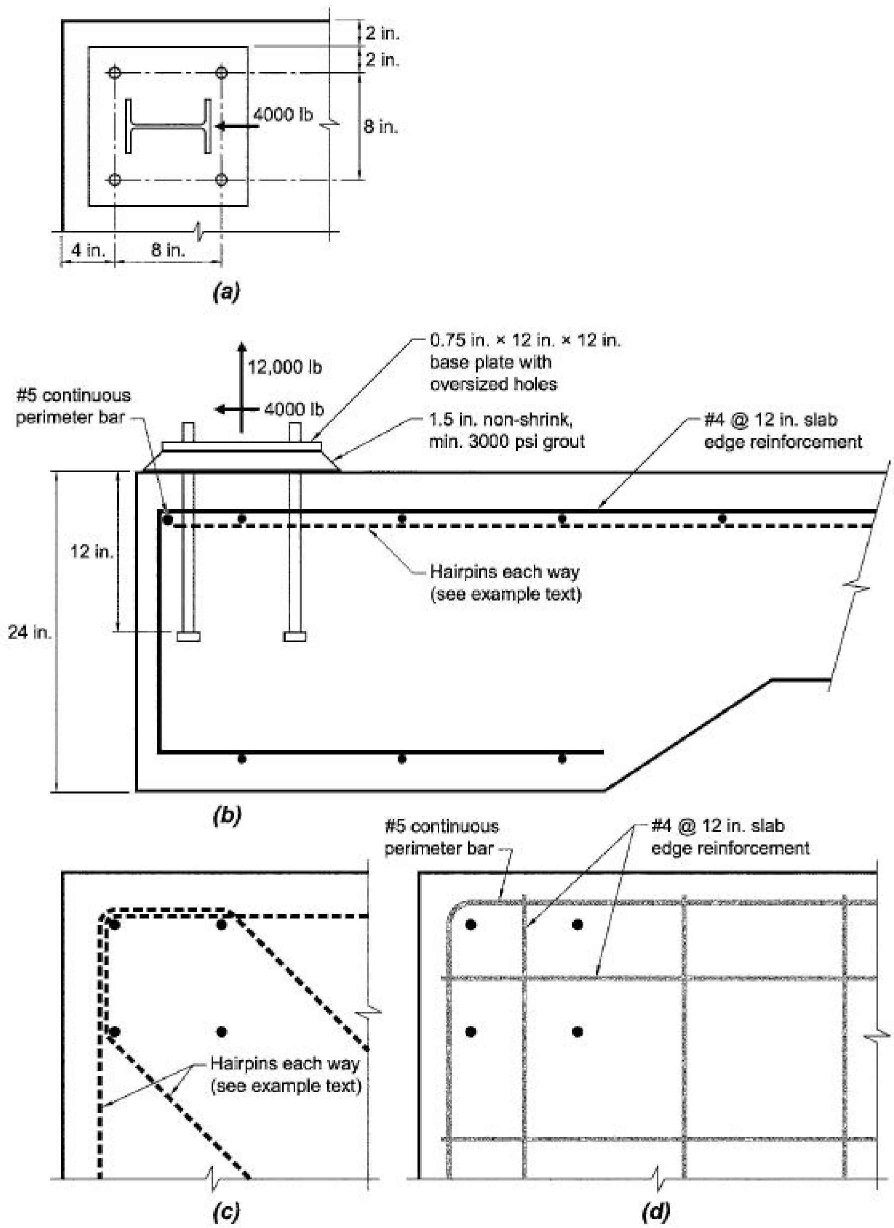


					Code Reference
					ACI 318-08
Bolt edge distance	c_{a1}	$= c_1 + s_1$	= 12.0	[in]	
Limiting c_{a1} when anchors are influenced by 3 or more edges			= No		D.6.2.4
Bolt edge distance - adjusted	$\mathbf{c_{a1}}$	= ca1 needs NOT to be adjusted	= **12.0**	[in]	D.6.2.4
	c_2	=	4.0	[in]	
	$1.5c_{a1}$	=	18.0	[in]	
	A_{Vc}	$= [\min(c_2,1.5c_{a1}) + s_2 + \min(c_4,1.5c_{a1})] \times \min(1.5c_{a1}, h_a)$	= 540.0	[in^2]	D.6.2.1
	A_{Vco}	$= 4.5{c_{a1}}^2$	= 648.0	[in^2]	D.6.2.1 (D-23)
	A_{Vc}	$= \min(A_{Vc},\ n_2 A_{Vco})$	= 540.0	[in^2]	D.6.2.1
	l_e	$= \min(8d_a, h_{ef})$	= 6.0	[in]	D.6.2.2
	V_b	$= \left[7\left(\dfrac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right]\lambda\sqrt{f_c'}\ c_{a1}^{1.5}$	= 20.9	[kips]	D.6.2.2 (D-24)
Eccentricity effects	$\Psi_{ec,v}$	= 1.0 shear acts through center of group			D.6.2.5
Edge effects	$\Psi_{ed,v}$	$= \min[(0.7+0.3c_2/1.5c_{a1}), 1.0]$	= 0.77		D.6.2.6
Concrete cracking	$\Psi_{c,v}$	=	= 1.20		D.6.2.7
Member thickness	$\Psi_{h,v}$	$= \max[(\mathrm{sqrt}(1.5c_{a1} / h_a), 1.0]$	= 1.00		D.6.2.8
Conc shear breakout resistance	V_{cbg2}	$= \phi_{v,c} \dfrac{A_{Vc}}{A_{Vco}} \Psi_{ec,V}\ \Psi_{ed,V}\ \Psi_{c,V}\ \Psi_{h,V}\ V_b$	= 12.0	[kips]	D.6.2.1 (D-22)
Min shear breakout resistance	V_{cbg}	$= \min(V_{cbg1}, V_{cbg2})$	= 9.8	[kips]	
Seismic design strength reduction		= x 1.0 not applicable	= 9.8	[kips]	D.3.3.3
	ratio	= 0.41	> V_u	**OK**	

					Code Reference
Conc. Pryout Shear Resistance					*ACI 318-08*
	k_{cp}	= 2.0			D.6.3
Factored shear pryout resistance	$\phi_{v,c}\ V_{cpg}$	= $\phi_{v,c}\ k_{cp}\ N_{cbg}$	= 41.1	[kips]	D.6.3 (D-31)
	$\phi_{\ v,c}$	= 0.70 pryout strength is always Condition B			D.4.4(c)
Seismic design strength reduction		= x 1.0 not applicable	= 41.1	[kips]	D.3.3.3
	ratio	= 0.10	> V_u	**OK**	
Govern Shear Resistance	$\mathbf{V_r}$	= min [$\phi_{v,s}V_{sa}$, $\phi_{v,c}$ (V_{cbg}, V_{cpg})]	= **9.8**	[kips]	
Tension Shear Interaction					
Check if N_u >0.2ϕ N_n and V_u >0.2ϕ V_n		Yes			D.7.1 & D.7.2
		N_u / ϕ N_n + V_u / ϕ V_n	= 0.99		D.7.3 (D-32)
	ratio	= 0.83	< 1.2	**OK**	
Ductility Tension					
	$\phi_{t,s}\ N_{sa}$	= 75.2 [kips]			
		> $\phi_{t,c}$ min (N_{cbg}, N_{pn}, N_{sbg})	= 20.5	[kips]	
		Non-ductile			
Ductility Shear					
	$\phi_{v,s}\ V_{sa}$	= 31.3 [kips]			
		> $\phi_{v,c}$ min (V_{cbg}, V_{cpg})	= 9.8	[kips]	
		Non-ductile			

Example 12: Anchor Bolt + No Anchor Reinft + Tension & Shear + CSA A23.3-04 Code

This example taken from Example 8 on page 71 of *ACI 355.3R-11 Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D*



N_u = 53.4 kN (tension), V_u=17.8 kN, f_c' = 20.7 MPa

Anchor bolt d_a=3/4 in ASTM F1554 Grade 55 h_{ef} =305mm h_a=610mm Anchor head → Hex

Supplementary reinforcement Tension → Condition B Shear → Condition A $\Psi_{c,V}$ =1.2

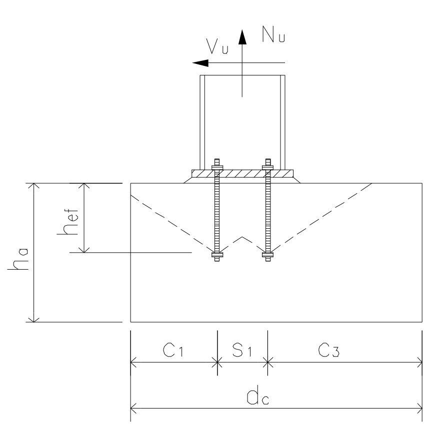
Provide built-up grout pad Seismic is not a consideration

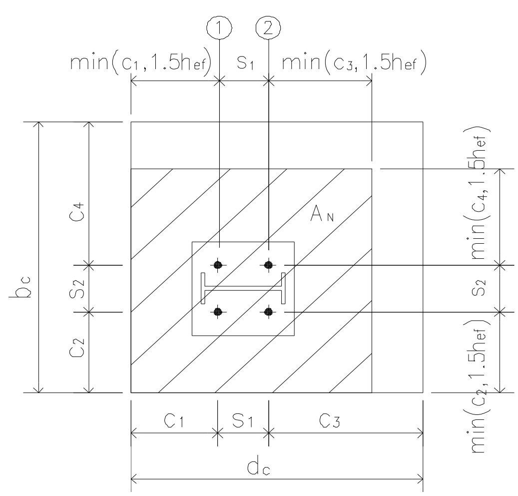
Field welded plate washers to base plate at each anchor

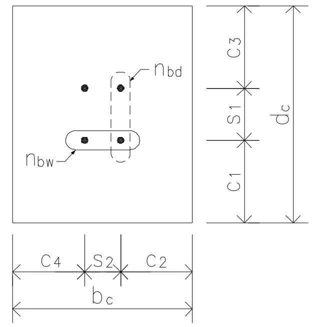
ANCHOR BOLT DESIGN **Combined Tension and Shear**

Anchor bolt design based on	**Code Abbreviation**
CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D	*A23.3-04 (R2010)*
PIP STE05121 Anchor Bolt Design Guide-2006	*PIP STE05121*

Anchor Bolt Data				set N_u = 0 if it's compression				**Code Reference**
Factored tension for design	$\mathbf{N_u}$ =	**53.4**	[kN]		= 12.0	[kips]		
Factored shear	V_u =	17.8	[kN]		= 4.0	[kips]		
Factored shear for design	$\mathbf{V_u}$ =	**17.8**	[kN]	V_u = 0 if shear key is provided				
Concrete strength	f'_c =	21	[MPa]		= 3.0	[ksi]		
Anchor bolt material	=	F1554 Grade 55						
Anchor tensile strength	f_{uta} =	75	[ksi]		= 517	[MPa]		*A23.3-04 (R2010)*
		Anchor is ductile steel element						D.2
Anchor bolt diameter	$\mathbf{d_a}$ =	**0.75**	[in]		= 19.1	[mm]		*PIP STE05121*
Bolt sleeve diameter	d_s =	51	[mm]					Page A -1 Table 1
Bolt sleeve height	h_s =	178	[mm]					
				min required				
Anchor bolt embedment depth	h_{ef} =	305	[mm]	229			OK	Page A -1 Table 1
Concrete thickness	h_a =	610	[mm]	381			OK	
Bolt edge distance c_1	c_1 =	102	[mm]	114			Warn	Page A -1 Table 1
Bolt edge distance c_2	c_2 =	102	[mm]	114			Warn	
Bolt edge distance c_3	c_3 =	2540	[mm]	114			OK	
Bolt edge distance c_4	c_4 =	2540	[mm]	114			OK	*A23.3-04 (R2010)*
$c_i > 1.5h_{ef}$ for at least two edges to avoid reducing of h_{ef} when $N_u > 0$							Yes	D.6.2.3
Adjusted h_{ef} for design	h_{ef} =	**305**	[mm]	229			OK	D.6.2.3
Outermost bolt line spacing s_1	s_1 =	203	[mm]	76			OK	*PIP STE05121*
Outermost bolt line spacing s_2	s_2 =	203	[mm]	76			OK	Page A -1 Table 1





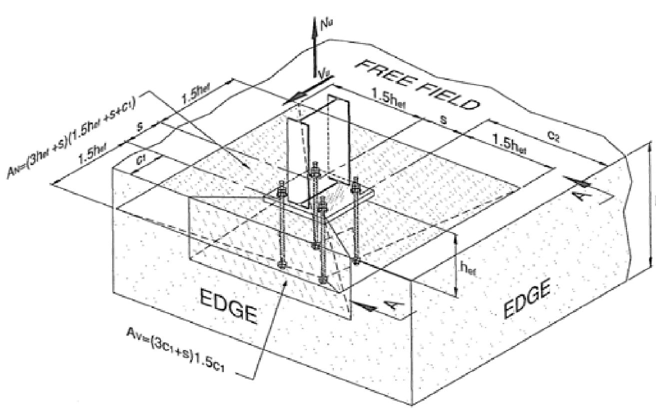


Bolt No Input for Side-Face Blowout Check Use

Number of bolt at bolt line 1	n_1	= 2			
Number of bolt at bolt line 2	n_2	= 2			
Number of bolt carrying tension	n_t	= 4			
Oversized holes in base plate ?		= No	?		
Number of bolt carrying shear	n_s	= 4			
For side-face blowout check use					
No of bolt along width edge	n_{bw}	= 2			
No of bolt along depth edge	n_{bd}	= 2			
Anchor head type		= Hex	?		
	A_{se}	= 215	[mm^2]		
Bearing area of head	A_{brg}	= **422**	[mm^2]		
	A_{brg}		[mm^2] not applicable		
Bolt 1/8" (3mm) corrosion allowance		= No	?		**Code Reference**
Provide shear key ?		= No	?		*A23.3-04 (R2010)*
Seismic region where $I_E F_a S_a(0.2)>=0.35$		= No	?		D.4.3.5
Supplementary reinforcement					
For tension		= No	Condition B		D.5.4 (c)
For shear	$\Psi_{c,V}$	= 1.2	Condition A	?	D.7.2.7
Provide built-up grout pad ?		= Yes	?		D.7.1.3
Strength reduction factors					
Anchor reinforcement factor	ϕ_{as}	= 0.75			D.7.2.9
Steel anchor resistance factor	ϕ_s	= 0.85			8.4.3 (a)
Concrete resistance factor	ϕ_c	= 0.65			8.4.2
Resistance modification factors					
Anchor rod - ductile steel	$R_{t,s}$	= 0.80		$R_{v,s}$ = 0.75	D.5.4(a)
Concrete	$R_{t,c}$	= 1.00	Cdn-B	$R_{v,c}$ = 1.15 Cdn-A	D.5.4(c)

Assumptions

1. Concrete is cracked
2. Condition B for tension - no supplementary reinforcement provided — D.5.4 (c)
3. Tensile load acts through center of bolt group $\Psi_{ec,N}$ =1.0 — D.6.2.4
4. Shear load acts through center of bolt group $\Psi_{ec,V}$ =1.0 — D.7.2.5
5. Anchor bolt washer shall be tack welded to base plate for all anchor bolts to transfer shear — *AISC Design Guide 1* section 3.5.3



CONCLUSION

Abchor Rod Embedment, Spacing and Edge Distance			**Warn**
Overall		ratio = **0.86**	**OK**
Tension			
Anchor Rod Tensile Resistance		ratio = 0.18	**OK**
Conc. Tensile Breakout Resistance		ratio = 0.62	**OK**
Anchor Pullout Resistance		ratio = 0.29	**OK**
Side Blowout Resistance		ratio = 0.24	**OK**
Shear			
Anchor Rod Shear Resistance		ratio = 0.13	**OK**
Conc. Shear Breakout Resistance		ratio = 0.41	**OK**
Conc. Pryout Shear Resistance		ratio = 0.10	**OK**
Anchor Rod on Conc Bearing		ratio = 0.08	**OK**
Tension Shear Interaction			
Tension Shear Interaction		ratio = 0.86	**OK**
Ductility			
	Tension **Non-ductile**	**Shear**	**Non-ductile**
Seismic Design Requirement			**OK** D.4.3.6

IeFaSa(0.2)<0.35, A23.3-04 D.4.3.3 ductility requirement is NOT required

CALCULATION — **Code Reference** *A23.3-04 (R2010)*

Anchor Rod Tensile Resistance

$N_{sr} = n_t A_{se} \phi_s f_{uta} R_{t,s}$ = 303.1 [kN] D.6.1.2 (D-3)

ratio = 0.18 > N_u **OK**

Conc. Tensile Breakout Resistance

$N_{br} = 10\ \phi_c \sqrt{f'_c}\ h_{ef}^{1.5} R_{t,c}$ if $h_{ef} \le 275$ or $h_{ef} \ge 625$ D.6.2.2 (D-7)

$3.9\ \phi_c \sqrt{f'_c}\ h_{ef}^{5/3} R_{t,c}$ if $275 < h_{ef} < 625$ D.6.2.2 (D-8)

= 160.5 [kN]

Projected conc failure area	$1.5h_{ef}$ =	= 458	[mm]	
	A_{Nc} = $[s_1+\min(c_1,1.5h_{ef})+\min(c_3,1.5h_{ef})]\times[s_2+\min(c_2,1.5h_{ef})+\min(c_4,1.5h_{ef})]$	= 5.8E+05	[mm^2]	
	$A_{Nco} = 9\ h_{ef}^2$	= 8.4E+05	[mm^2]	D.6.2.1 (D-6)
	$A_{Nc} = \min(A_{Nc},\ n_t A_{Nco})$	= 5.8E+05	[mm^2]	D.6.2.1
Min edge distance	$c_{min} = \min(c_1, c_2, c_3, c_4)$	= 102	[mm]	
Eccentricity effects	$\Psi_{ec,N}$ = 1.0 for no eccentric load			D.6.2.4
Edge effects	$\Psi_{ed,N} = \min[(0.7+0.3c_{min}/1.5h_{ef}), 1.0]$	= 0.77		D.6.2.5
Concrete cracking	$\Psi_{c,N}$ = 1.0 for cracked concrete			D.6.2.6
Concrete splitting	$\Psi_{cp,N}$ = 1.0 for cast-in anchor			D.6.2.7

					Code Reference
					A23.3-04 (R2010)
Concrete breakout resistance	$N_{cbgr} = \frac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_{br}$		= 85.5	[kN]	D.6.2.1 (D-5)
Seismic design strength reduction	= x 1.0	not applicable	= 85.5	[kN]	D.4.3.5
	ratio = 0.62		> N_u	**OK**	

Anchor Pullout Resistance					
Single bolt pullout resistance	$N_{pr} = 8\, A_{brg}\, \phi_c\, f_c'\, R_{t,c}$		= 46.1	[kN]	D.6.3.4 (D-16)
	$N_{cpr} = n_t\, \Psi_{c,p}\, N_{pr}$		= 184.3	[kN]	D.6.3.1 (D-15)
Seismic design strength reduction	= x 1.0	not applicable	= 184.3	[kN]	D.4.3.5
	ratio = 0.29		> N_u	**OK**	
	$\Psi_{c,p}$ = 1 for cracked conc				D.6.3.6
	$R_{t,c}$ = 1.00	pullout strength is always Condition B			D.5.4(c)

Side Blowout Resistance					
Failure Along Pedestal Width Edge					
Tensile load carried by anchors close to edge which may cause side-face blowout					*ACI318 M-08*
along pedestal width edge	$N_{buw} = N_u \times n_{bw} / n_t$		= 26.7	[kN]	RD.5.4.2
	$c = \min(c_1, c_3)$		= 102	[mm]	
Check if side blowout applicable	h_{ef} = 305 [mm]				*A23.3-04 (R2010)*
	> 2.5c	side bowout is applicable			D.6.4.1
Check if edge anchors work as a	s_{22} = 203 [mm]	$s = s_2$	= 203	[mm]	
a group or work individually	< 6c	edge anchors work as a group			D.6.4.2
Single anchor SB resistance	$N_{sbr,w} = 13.3c\sqrt{A_{brg}}\, \phi_c \sqrt{f'_c}\, R_{t,c}$		= 83.0	[kN]	D.6.4.1 (D-18)
Multiple anchors SB resistance	$N_{sbgr,w}$ =				
work as a group - applicable	$= (1+s/6c) \times N_{sbr,w}$		= 110.5	[kN]	D.6.4.2 (D-19)
work individually - not applicable	$= n_{bw} \times N_{sbr,w} \times [1+(c_2 \text{ or } c_4)/c] / 4$		= 0.0	[kN]	D.6.4.1
Seismic design strength reduction	= x 1.0	not applicable	= 110.5	[kN]	D.4.3.5
	ratio = 0.24		> N_{buw}	**OK**	
Failure Along Pedestal Depth Edge					
Tensile load carried by anchors close to edge which may cause side-face blowout					*ACI318 M-08*
along pedestal depth edge	$N_{bud} = N_u \times n_{bd} / n_t$		= 26.7	[kN]	RD.5.4.2
	$c = \min(c_2, c_4)$		= 102	[mm]	
Check if side blowout applicable	h_{ef} = 305 [mm]				*A23.3-04 (R2010)*
	> 2.5c	side bowout is applicable			D.6.4.1
Check if edge anchors work as a	s_{11} = 203 [mm]	$s = s_1$	= 203	[mm]	
a group or work individually	< 6c	edge anchors work as a group			D.6.4.2
Single anchor SB resistance	$N_{sbr,d} = 13.3c\sqrt{A_{brg}}\, \phi_c \sqrt{f'_c}\, R_{t,c}$		= 83.0	[kN]	D.6.4.1 (D-18)
Multiple anchors SB resistance	$N_{sbgr,d}$ =				
work as a group - applicable	$= (1+s/6c) \times \phi_{t,c}\, N_{sbr,d}$		= 110.5	[kN]	D.6.4.2 (D-19)
work individually - not applicable	$= n_{bd} \times N_{sbr,d} \times [1+(c_1 \text{ or } c_3)/c] / 4$		= 0.0	[kN]	D.6.4.1
Seismic design strength reduction	= x 1.0	not applicable	= 110.5	[kN]	D.4.3.5
	ratio = 0.24		> N_{bud}	**OK**	

				Code Reference
Group side blowout resistance	$N_{sbgr} = \min\left(\frac{N_{sbgr,w}}{n_{bw}} n_t \,,\, \frac{N_{sbgr,d}}{n_{bd}} n_t \right)$	= 221.1	[kN]	*A23.3-04 (R2010)*
Govern Tensile Resistance	$\mathbf{N_r} = \min(N_{sr}, N_{rbr}, N_{cpr}, N_{sbgr})$	= **85.5**	[kN]	

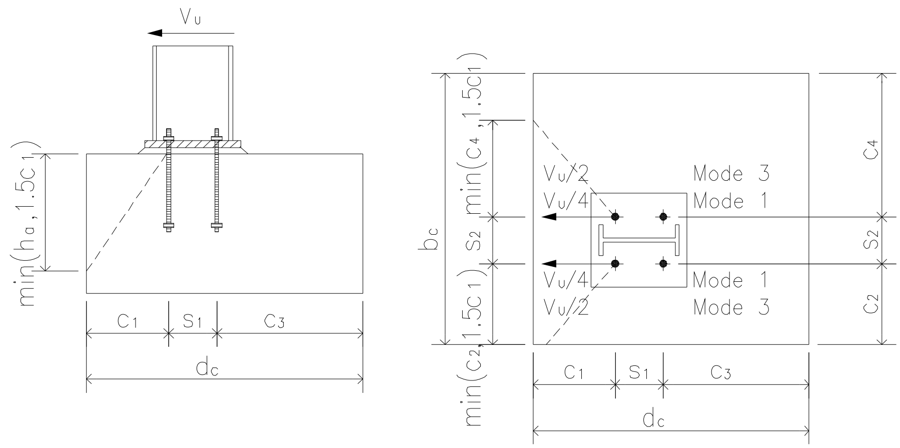
Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear

Anchor Rod Shear Resistance	$V_{sr} = n_s A_{se} \phi_s \, 0.6 \, f_{uta} R_{v,s}$	= 170.5	[kN]	D.7.1.2 (b) (D-21)
Reduction due to built-up grout pads	= x 0.8 , applicable	= 136.4	[kN]	D.7.1.3
	ratio = 0.13	> V_u	**OK**	

Conc. Shear Breakout Resistance

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 oversized holes are used in base plate

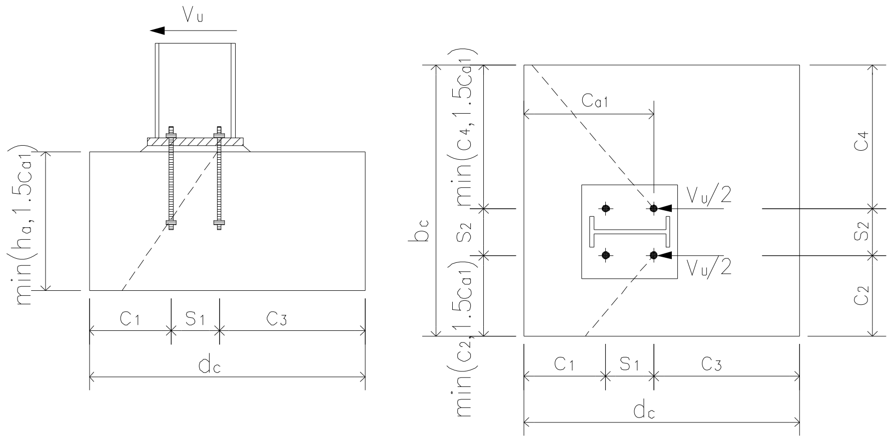


A23.3-04 (R2010)

Bolt edge distance	c_1 =	= 102	[mm]	
Limiting c_{a1} when anchors are influenced by 3 or more edges		= No		D.7.2.4
Bolt edge distance - adjusted	$\mathbf{c_1}$ = ca1 needs NOT to be adjusted	= **102**	[mm]	D.7.2.4
	c_2 =	= 102	[mm]	
	$1.5c_1$ =	= 153	[mm]	
	$A_{Vc} = [\min(c_2, 1.5c_1) + s_2 + \min(c_4, 1.5c_1)] \times \min(1.5c_1, h_a)$	= 7.0E+04	[mm^2]	D.7.2.1
	$A_{Vco} = 4.5 c_1^2$	= 4.7E+04	[mm^2]	D.7.2.1 (D-24)
	$A_{Vc} = \min(A_{Vc}, \; n_1 A_{Vco})$	= 7.0E+04	[mm^2]	D.7.2.1
	$l_e = \min(8d_a \,,\, h_{ef})$	= 152	[mm]	D.3
	$V_{br} = 0.58 \left(\frac{l_e}{d_a} \right)^{0.2} \sqrt{d_a} \; \phi_c \sqrt{f'_c} \; c_{a1}^{1.5} R_{v,c}$	= 13.5	[kN]	D.7.2.2 (D-25)
Eccentricity effects	$\Psi_{ec,v}$ = 1.0 shear acts through center of group			D.7.2.5
Edge effects	$\Psi_{ed,v} = \min[\,(0.7 + 0.3c_2/1.5c_1), 1.0\,]$	= 0.90		D.7.2.6
Concrete cracking	$\Psi_{c,v}$ =	= 1.20		D.7.2.7
Member thickness	$\Psi_{h,v} = \max[\,(\mathrm{sqrt}(1.5c_1 / h_a)\,, 1.0\,]$	= 1.00		D.7.2.8

				Code Reference
Conc shear breakout				*A23.3-04 (R2010)*
resistance	$V_{cbgr1} = \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_{br}$	= 21.9	[kN]	D.7.2.1 (D-23)
Mode 1 is used for checking	$\mathbf{V_{cbgr1}} = V_{cbg1}$ x 2.0	= **43.8**	[kN]	

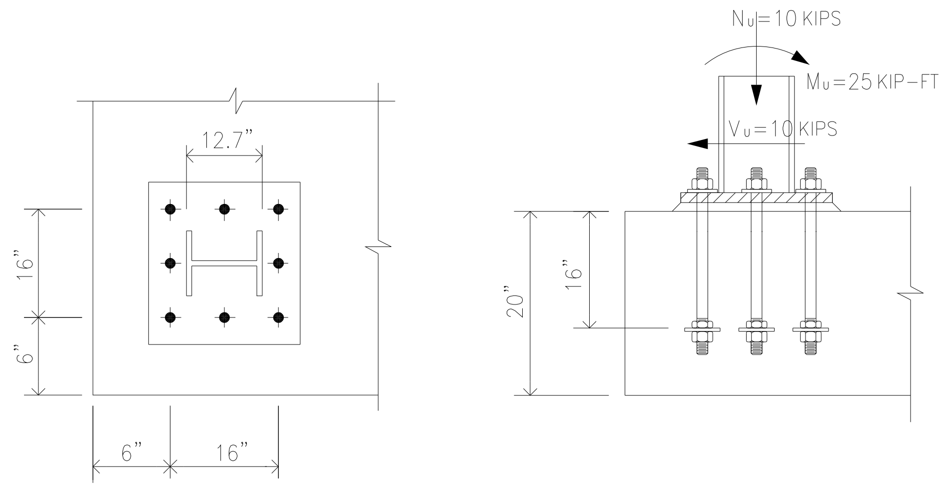
Mode 2 Failure cone at back anchors



				A23.3-04 (R2010)
Bolt edge distance	$c_{a1} = c_1 + s_1$	= 305	[mm]	
Limiting c_{a1} when anchors are influenced by 3 or more edges		= No		D.7.2.4
Bolt edge distance - adjusted	$\mathbf{c_{a1}}$ = ca1 needs NOT to be adjusted	= **305**	[mm]	D.7.2.4
	c_2 =	102	[mm]	
	$1.5c_{a1}$ =	458	[mm]	
	$A_{Vc} = [\min(c_2, 1.5c_{a1}) + s_2 + \min(c_4, 1.5c_{a1})] \times \min(1.5c_{a1}, h_a)$	= 3.5E+05	[mm^2]	D.7.2.1
	$A_{Vco} = 4.5c_{a1}^{\ 2}$	= 4.2E+05	[mm^2]	D.7.2.1 (D-24)
	$A_{Vc} = \min(A_{Vc},\ n_2 A_{Vco})$	= 3.5E+05	[mm^2]	D.7.2.1
	$l_e = \min(8d_a, h_{ef})$	= 152	[mm]	D.3
	$V_{br} = 0.58 \left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\ \phi_c \sqrt{f_c'}\ c_{a1}^{1.5} R_{v,c}$	= 70.0	[kN]	D.7.2.2 (D-25)
Eccentricity effects	$\Psi_{ec,v}$ = 1.0 shear acts through center of group			D.7.2.5
Edge effects	$\Psi_{ed,v} = \min[(0.7+0.3c_2/1.5c_{a1}), 1.0]$	= 0.77		D.7.2.6
Concrete cracking	$\Psi_{c,v}$ =	= 1.20		D.7.2.7
Member thickness	$\Psi_{h,v} = \max[(\text{sqrt}(1.5c_{a1} / h_a), 1.0]$	= 1.00		D.7.2.8
Conc shear breakout resistance	$V_{cbgr2} = \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_{br}$	= 53.7	[kN]	D.7.2.1 (D-23)
Min shear breakout resistance	$V_{cbgr} = \min(V_{cbgr1}, V_{cbgr2})$	= 43.8	[kN]	
Seismic design strength reduction	= x 1.0 not applicable	= 43.8	[kN]	D.4.3.5
	ratio = 0.41	> V_u	**OK**	

					Code Reference
Conc. Pryout Shear Resistance					*A23.3-04 (R2010)*
	k_{cp} = 2.0				D.7.3
Factored shear pryout resistance	V_{cpgr} = k_{cp} N_{cbgr}		= 171.0	[kN]	D.7.3 (D-32)
	$R_{v,c}$ = 1.00	pryout strength is always Condition B			D.5.4(c)
Seismic design strength reduction	= x 1.0	not applicable	= 171.0	[kN]	D.4.3.5
	ratio = 0.10		> V_u	**OK**	
					CSA S16-09
Anchor Rod on Conc Bearing	B_r = n_s x 1.4 x ϕ_c x min(8d_a, h_{ef}) x d_a x f_c'		= 221.9	[kN]	25.3.3.2
	ratio = 0.08		> V_u	**OK**	
Govern Shear Resistance	$\mathbf{V_r}$ = min (V_{sr}, V_{cbgr}, V_{cpgr} , B_r)		= **43.8**	[kN]	
					A23.3-04 (R2010)
Tension Shear Interaction					
Check if N_u >0.2 N_r and V_u >0.2 V_r	Yes				D.8.2 & D.8.3
	$N_u/N_r + V_u/V_r$		= 1.03		D.8.4 (D-35)
	ratio = 0.86		< 1.2	**OK**	
Ductility Tension					
	N_{sr} = 303.1 [kN]				
	> min (N_{cbgr}, N_{cpr}, N_{sbgr})		= 85.5	[kN]	
		Non-ductile			
Ductility Shear					
	V_{sr} = 136.4 [kN]				
	> min (V_{cbgr}, V_{cpgr}, B_r)		= 43.8	[kN]	
		Non-ductile			

Example 13: Anchor Bolt + No Anchor Reinft + Tension Shear & Moment + ACI 318-08 Code



M_u = 25 kip-ft N_u= 10 kips (Compression) V_u = 10 kips

Concrete f_c'= 5 ksi

Anchor bolt   F1554 Grade 36   1.25" dia   Heavy Hex Head h_{ef} = 16" h_a =20"

Oversized holes in base plate

Seismic design category < C

Supplementary reinforcement   Tension → Condition A

Shear → Condition A $\Psi_{c,V}$ = 1.2

Provide built-up grout pad

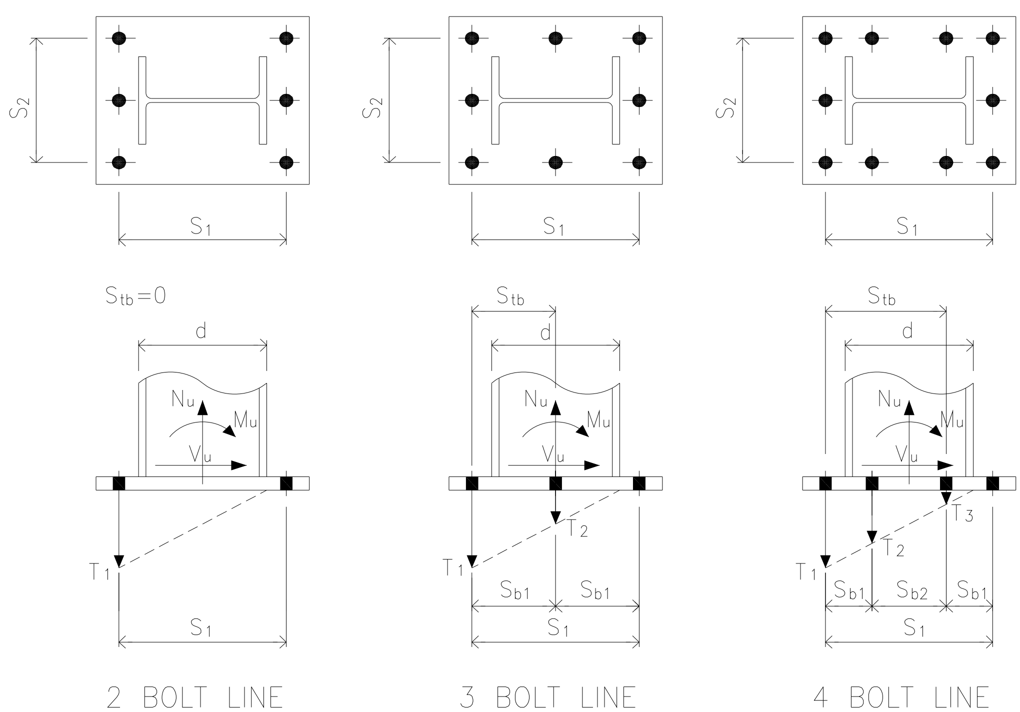
ANCHOR BOLT DESIGN **Combined Tension, Shear and Moment**

Anchor bolt design based on	**Code Abbreviation**
ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D	*ACI 318-08*
PIP STE05121 Anchor Bolt Design Guide-2006	*PIP STE05121*

Assumptions	**Code Reference**
	ACI 318-08
1. Concrete is cracked	
2. Condition A - supplementary reinforcement provided	D.4.4 (c)
3. Load combinations shall be per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2	D.4.4
4. Shear load acts through center of bolt group $\Psi_{ec,V}$ =1.0	D.6.2.5
5. For anchor group subject to moment, the anchor tensile load is designed using elastic analysis and there is no redistribution of the forces between highly stressed and less stressed anchors	D.3.1
6. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output	
7. Shear carried by only half of total anchor bolts due to oversized holes in column base plate	*AISC Design Guide 1* section 3.5.3

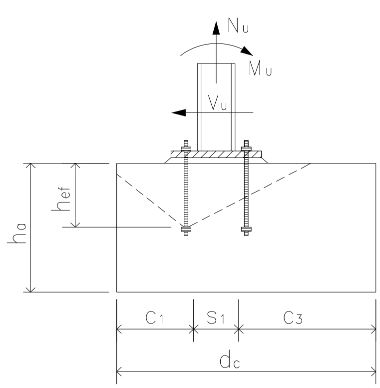
Anchor Bolt Data

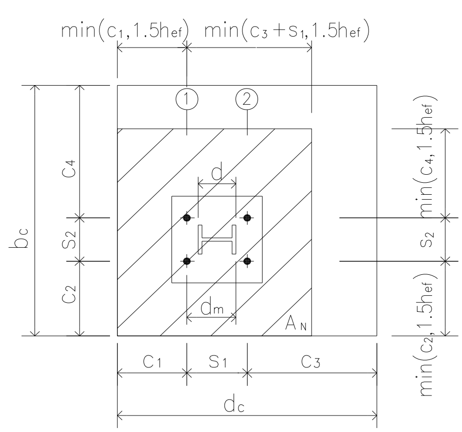
Factored moment	M_u =	25.0	[kip-ft]		= 33.9	[kNm]
Factored tension /compression	N_u =	-10.0	[kips]	in compression	= -44.5	[kN]
Factored shear	V_u =	10.0	[kips]		= 44.5	[kN]
Factored shear for bolt design	$\mathbf{V_u}$ =	**10.0**	[kips]	V_u = 0 if shear key is provided		



No of bolt line for resisting moment	=	3 Bolt Line
No of bolt along outermost bolt line	=	3

				min required				**Code Reference**
								PIP STE05121
Outermost bolt line spacing s_1	s_1 =	16.0	[in]	5.0	**OK**			Page A -1 Table 1
Outermost bolt line spacing s_2	s_2 =	16.0	[in]	5.0	**OK**			
Internal bolt line spacing s_{b1}	s_{b1} =	8.0	[in]	5.0	**OK**			
Internal bolt line spacing s_{b2}	s_{b2} =	0.0	[in]	5.0	**OK**			
Column depth	d =	12.7	[in]					
Concrete strength	f'_c =	5.0	[ksi]			= 34.5	[MPa]	
Anchor bolt material	=	F1554 Grade 36						
Anchor tensile strength	f_{uta} =	58	[ksi]			= 400	[MPa]	*ACI 318-08*
		Anchor is ductile steel element						D.1
Anchor bolt diameter	**d_a** =	**1.25**	[in]			= 31.8	[mm]	*PIP STE05121*
Bolt sleeve diameter	d_s =	3.0	[in]					Page A -1 Table 1
Bolt sleeve height	h_s =	10.0	[in]					
				min required				
Anchor bolt embedment depth	h_{ef} =	16.0	[in]	15.0	**OK**			Page A -1 Table 1
Concrete thickness	h_a =	20.0	[in]	19.0	**OK**			
Bolt edge distance c_1	c_1 =	6.0	[in]	5.0	**OK**			Page A -1 Table 1
Bolt edge distance c_2	c_2 =	6.0	[in]	5.0	**OK**			
Bolt edge distance c_3	c_3 =	100.0	[in]	5.0	**OK**			
Bolt edge distance c_4	c_4 =	100.0	[in]	5.0	**OK**			*ACI 318-08*
$c_i > 1.5h_{ef}$ for at least two edges to avoid reducing of h_{ef} when $N_u > 0$					**Yes**			D.5.2.3
Adjusted h_{ef} for design	h_{ef} =	**16.00**	[in]	15.0	**OK**			D.5.2.3





						Code Reference
Number of bolt at bolt line 1	n_1 =	3				
Number of bolt at bolt line 2	n_2 =	3				
Number of bolt carrying tension	n_t =	5				
Oversized holes in base plate ?	=	Yes	?			
Total no of anchor bolt	n =	8				
Number of bolt carrying shear	n_s =	4				
Anchor head type	=	Heavy Hex		?		
Anchor effective cross sect area	A_{se} =	0.969	[in^2]			
Bearing area of head	A_{brg} =	**2.237**	[in^2]			
	A_{brg}		[in^2]	not applicable		
Bolt 1/8" (3mm) corrosion allowance		No	?			
Provide shear key ?		No	?			*ACI 318-08*
Seismic design category >= C		No	?			D.3.3.3
Supplementary reinforcement						
For tension		Yes	Condition A			D.4.4 (c)
For shear	$\Psi_{c,V}$ =	1.2	Condition A	?		D.6.2.7
Provide built-up grout pad ?		Yes	?			D.6.1.3

Strength reduction factors						
Anchor reinforcement	ϕ_s =	0.75				D.5.2.9 & D.6.2.9
Anchor rod - ductile steel	$\phi_{t,s}$ =	0.75		$\phi_{v,s}$ = 0.65		D.4.4 (a)
Concrete	$\phi_{t,c}$ =	0.75	Cdn-A	$\phi_{v,c}$ = 0.75	Cdn-A	D.4.4 (c)

CONCLUSION

Abchor Rod Embedment, Spacing and Edge Distance		**OK**	
Overall	ratio = **0.81**	**OK**	
Tension			
Anchor Rod Tensile Resistance	ratio = 0.12	**OK**	
Conc. Tensile Breakout Resistance	ratio = 0.39	**OK**	
Anchor Pullout Resistance	ratio = 0.08	**OK**	
Side Blowout Resistance	ratio = 0.13	**OK**	
Shear			
Anchor Rod Shear Resistance	ratio = 0.14	**OK**	
Conc. Shear Breakout Resistance	ratio = 0.58	**OK**	
Conc. Pryout Shear Resistance	ratio = 0.11	**OK**	
Tension Shear Interaction			
Tension Shear Interaction	ratio = 0.81	**OK**	
Ductility			*ACI 318-08*
Tension **Ductile**	**Shear**	**Non-ductile**	
Seismic Design Requirement		**OK**	D.3.3.4

SDC< C, ACI318-08 D.3.3 ductility requirement is NOT required

CALCULATION					**Code Reference**
Anchor Tensile Force					*ACI 318-08*
Single bolt tensile force	T_1 = 4.86 [kips]	No of bolt for T_1 n_{T1} = 3			
	T_2 = 2.15 [kips]	No of bolt for T_2 n_{T2} = 2			
	T_3 = 0.00 [kips]	No of bolt for T_3 n_{T3} = 0			
Sum of bolt tensile force	$\mathbf{N_u} = \Sigma\, n_i\, T_i$		= **18.9**	[kips]	
Tensile bolts outer distance s_{tb}	s_{tb} = 8.0 [in]				
Eccentricity e'_N -- distance between resultant of tensile load and centroid of anchors loaded in tension	e'_N = 1.38 [in]				Fig. RD.5.2.4 (b)
Eccentricity modification factor	$\Psi_{ec,N} = \dfrac{1}{\left(1+\dfrac{2e'_N}{3h_{ef}}\right)}$		= 0.95		D.5.2.4 (D-9)
Anchor Rod Tensile Resistance	$\phi_{t,s} N_{sa} = \phi_{t,s} A_{se} f_{uta}$		= 42.2	[kips]	D.5.1.2 (D-3)
	ratio = 0.12		> T_1	**OK**	
Conc. Tensile Breakout Resistance					
	$N_b = 24\ \lambda \sqrt{f'_c}\ h_{ef}^{1.5}$ if $h_{ef} < 11"$ or $h_{ef} > 25"$		= 114.9	[kips]	D.5.2.2 (D-7)
	$16\ \lambda \sqrt{f'_c}\ h_{ef}^{5/3}$ if $11" \le h_{ef} \le 25"$				D.5.2.2 (D-8)
Projected conc failure area	$1.5h_{ef}$ =		= 24.00	[in]	
	$A_{Nc} = [s_{tb}+\min(c_1,1.5h_{ef})+\min(c_3,1.5h_{ef})]\text{x}$ $[s_2+\min(c_2,1.5h_{ef})+\min(c_4,1.5h_{ef})]$		= 1748.0	[in^2]	
	$A_{Nco} = 9\ h_{ef}^2$		= 2304.0	[in^2]	D.5.2.1 (D-6)
	$A_{Nc} = \min(A_{Nc},\ n_t A_{Nco})$		= 1748.0	[in^2]	D.5.2.1
Min edge distance	$c_{min} = \min(c_1, c_2, c_3, c_4)$		= 6.0	[in]	
Eccentricity effects	$\Psi_{ec,N}$ =		= 0.95		D.5.2.4 (D-9)
Edge effects	$\Psi_{ed,N} = \min[(0.7+0.3c_{min}/1.5h_{ef}),\ 1.0]$		= 0.78		D.5.2.5
Concrete cracking	$\Psi_{c,N}$ = 1.0 for cracked concrete				D.5.2.6
Concrete splitting	$\Psi_{cp,N}$ = 1.0 for cast-in anchor				D.5.2.7
Concrete breakout resistance	$\phi_{t,c} N_{cbg} = \phi_{t,c} \dfrac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$		= 47.9	[kips]	D.5.2.1 (D-5)
Seismic design strength reduction	= x 1.0 not applicable		= 47.9	[kips]	D.3.3.3
	ratio = 0.39		> N_u	**OK**	
Anchor Pullout Resistance					
Single bolt pullout resistance	$N_p = 8\ A_{brg}\ f_c'$		= 89.5	[kips]	D.5.3.4 (D-15)
	$\phi_{t,c} N_{pn} = \phi_{t,c} \Psi_{c,p} N_p$		= 62.6	[kips]	D.5.3.1 (D-14)
Seismic design strength reduction	= x 1.0 not applicable		= 62.6	[kips]	D.3.3.3
	ratio = 0.08		> T_1	**OK**	
	$\Psi_{c,p}$ = 1 for cracked conc				D.5.3.6
	$\phi_{t,c}$ = 0.70	pullout strength is always Condition B			D.4.4(c)

Side Blowout Resistance **Code Reference**

Failure Along Pedestal Width Edge *ACI 318-08*

Tensile load carried by anchors close to edge which may cause side-face blowout

along pedestal width edge $N_{buw} = n_{T1}\, T_1$ = 14.6 [kips] RD.5.4.2

$c = \min(c_1, c_3)$ = 6.0 [in]

Check if side blowout applicable $h_{ef} = 16.0$ [in]

$> 2.5c$ side bowout is applicable D.5.4.1

Check if edge anchors work as a $s_{22} = 8.0$ [in] $s = s_2 = 16.0$ [in]

a group or work individually $< 6c$ edge anchors work as a group D.5.4.2

Single anchor SB resistance $\phi_{t,c}\, N_{sb} = \phi_{t,c} \left(160\, c \sqrt{A_{brg}}\right) \lambda \sqrt{f'_c}$ = 76.1 [kips] D.5.4.1 (D-17)

Multiple anchors SB resistance $\phi_{t,c} N_{sbg,w} =$

work as a group - applicable $= (1+s/\,6c) \times \phi_{t,c}\, N_{sb}$ = 110.0 [kips] D.5.4.2 (D-18)

work individually - not applicable $= n_{bw} \times \phi_{t,c}\, N_{sb} \times [1+(c_2 \text{ or } c_4)\,/\,c]\,/\,4$ = 0.0 [kips] D.5.4.1

Seismic design strength reduction = x 1.0 not applicable = 110.0 [kips] D.3.3.3

ratio = 0.13 $> N_{buw}$ **OK**

Group side blowout resistance $\phi_{t,c}\, N_{sbg} = \phi_{t,c} \dfrac{N_{sbgr,w}}{n_{T1}} n_t$ = 183.3 [kips]

Govern Tensile Resistance $\mathbf{N_r} = \min[\,\phi_{t,s}\, n_t\, N_{sa},\ \phi_{t,c}\,(N_{cbg},\ n_t\, N_{pn},\ N_{sbg})\,]$ = **47.9** [kips]

Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear

Anchor Rod Shear Resistance $\phi_{v,s}\, V_{sa} = \phi_{v,s}\, n_s\, 0.6\, A_{se}\, f_{uta}$ = 87.7 [kips] D.6.1.2 (b) (D-20)

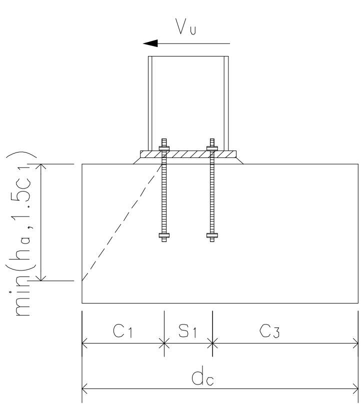
Reduction due to built-up grout pads = x 0.8 , applicable = 70.1 [kips] D.6.1.3

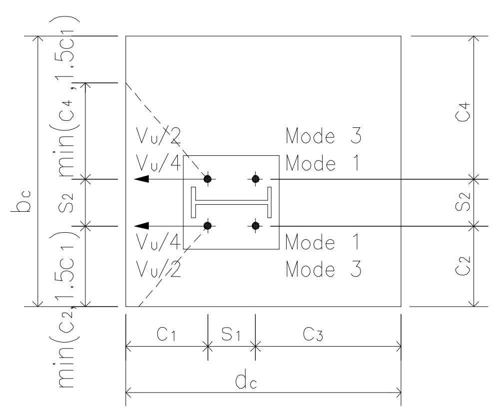
ratio = 0.14 $> V_u$ **OK**

Conc. Shear Breakout Resistance

Mode 1 Failure cone at front anchors, strength check against $0.5 \times V_u$

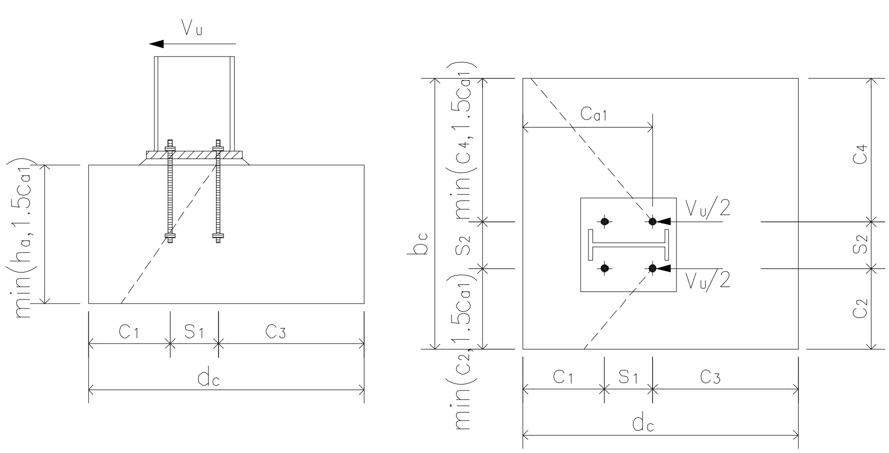
Mode 3 Failure cone at front anchors, strength check against $1.0 \times V_u$, applicable when oversized holes are used in base plate





					Code Reference
Bolt edge distance	c_{a1} =		= 6.0	[in]	*ACI 318-08*
Limiting c_{a1} when anchors are influenced by 3 or more edges			= No		D.6.2.4
Bolt edge distance - adjusted	$\mathbf{c_{a1}}$ =	ca1 needs NOT to be adjusted	= **6.0**	[in]	D.6.2.4
	c_2 =		= 6.0	[in]	
	$1.5c_{a1}$ =		= 9.0	[in]	
	A_{Vc} =	$[\min(c_2, 1.5c_{a1}) + s_2 + \min(c_4, 1.5c_{a1})]$ x $\min(1.5c_{a1}, h_a)$	= 279.0	[in^2]	D.6.2.1
	A_{Vco} =	$4.5c_{a1}^{\ 2}$	= 162.0	[in^2]	D.6.2.1 (D-23)
	A_{Vc} =	$\min(A_{Vc},\ n_1 A_{Vco})$	= 279.0	[in^2]	D.6.2.1
	l_e =	$\min(8d_a, h_{ef})$	= 10.0	[in]	D.6.2.2
	V_b =	$\left[7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right]\lambda\sqrt{f'_c}\ c_1^{1.5}$	= 12.3	[kips]	D.6.2.2 (D-24)
Eccentricity effects	$\Psi_{ec,v}$ =	1.0 shear acts through center of group			D.6.2.5
Edge effects	$\Psi_{ed,v}$ =	min[$(0.7+0.3c_2/1.5c_1)$, 1.0]	= 0.90		D.6.2.6
Concrete cracking	$\Psi_{c,v}$ =		= 1.20		D.6.2.7
Member thickness	$\Psi_{h,v}$ =	max[$(\mathrm{sqrt}(1.5c_1 / h_a)$, 1.0]	= 1.00		D.6.2.8
Conc shear breakout resistance	V_{cbg1} =	$\phi_{v,c}\frac{A_{Vc}}{A_{Vco}}\Psi_{ec,V}\ \Psi_{ed,V}\ \Psi_{c,V}\ \Psi_{h,V}\ V_b$	= 17.2	[kips]	D.6.2.1 (D-22) Fig. RD.6.2.1 (b)
Mode 3 is used for checking	$\mathbf{V_{cbg1}}$ =	V_{cbg1} x 1.0	= **17.2**	[kips]	note

Mode 2 Failure cone at back anchors



					ACI 318-08
Bolt edge distance	c_{a1} =		= 22.0	[in]	
Limiting c_{a1} when anchors are influenced by 3 or more edges			= No		D.6.2.4
Bolt edge distance - adjusted	$\mathbf{c_{a1}}$ =	ca1 needs NOT to be adjusted	= **22.0**	[in]	D.6.2.4
	c_2 =		6.0	[in]	
	$1.5c_{a1}$ =		33.0	[in]	

					Code Reference
					ACI 318-08
	A_{Vc} =	$[\min(c_2,1.5c_{a1}) + s_2 + \min(c_4,1.5c_{a1})]$ x $\min(1.5c_{a1}, h_a)$	= 1100.0	$[in^2]$	D.6.2.1
	A_{Vco} =	$4.5{c_{a1}}^2$	= 2178.0	$[in^2]$	D.6.2.1 (D-23)
	A_{Vc} =	$\min(A_{Vc},\ n_2 A_{Vco})$	= 1100.0	$[in^2]$	D.6.2.1
	l_e =	$\min(8d_a, h_{ef})$	= 10.0	[in]	D.6.2.2
	V_b =	$\left[7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right]\lambda\sqrt{f'_c}\ c_{a1}^{1.5}$	= 86.6	[kips]	D.6.2.2 (D-24)
Eccentricity effects	$\Psi_{ec,v}$ =	1.0 shear acts through center of group			D.6.2.5
Edge effects	$\Psi_{ed,v}$ =	$\min[(0.7+0.3c_2/1.5c_{a1}), 1.0]$	= 0.75		D.6.2.6
Concrete cracking	$\Psi_{c,v}$ =		= 1.20		D.6.2.7
Member thickness	$\Psi_{h,v}$ =	$\max[(\mathrm{sqrt}(1.5c_{a1}/h_a), 1.0]$	= 1.28		D.6.2.8
Conc shear breakout resistance	V_{cbg2} =	$\phi_{v,c}\frac{A_{Vc}}{A_{Vco}}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_b$	= 38.1	[kips]	D.6.2.1 (D-22)
Min shear breakout resistance	V_{cbg} =	$\min(V_{cbg1}, V_{cbg2})$	= 17.2	[kips]	
Seismic design strength reduction	=	x 1.0 not applicable	= 17.2	[kips]	D.3.3.3
	ratio =	0.58	> V_u	**OK**	

Conc. Pryout Shear Resistance

	k_{cp} =	2.0			D.6.3
Factored shear pryout resistance	$\phi_{v,c} V_{cpg}$ =	$\phi_{v,c} k_{cp} N_{cbg}$	= 89.5	[kips]	D.6.3 (D-31)
	$\phi_{v,c}$ =	0.70 pryout strength is always Condition B			D.4.4(c)
Seismic design strength reduction	=	x 1.0 not applicable	= 89.5	[kips]	D.3.3.3
	ratio =	0.11	> V_u	**OK**	

Govern Shear Resistance	$\mathbf{V_r}$ =	$\min[\phi_{v,s}V_{sa}, \phi_{v,c}(V_{cbg}, V_{cpg})]$	= **17.2**	[kips]	

Tension Shear Interaction

Check if $N_u > 0.2\phi N_n$ and $V_u > 0.2\phi V_n$		Yes			D.7.1 & D.7.2
		$N_u/\phi N_n + V_u/\phi V_n$	= 0.98		D.7.3 (D-32)
	ratio =	0.81	< 1.2	**OK**	

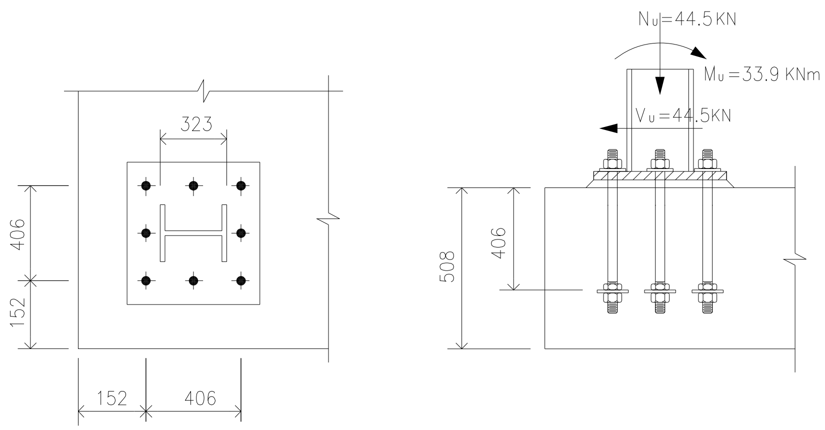
Ductility Tension

$\phi_{t,s} N_{sa}$ =	42.2 [kips]			
<	$\phi_{t,c} \min(N_{cbg}, N_{pn}, N_{sbg})$	= 47.9	[kips]	
	Ductile			

Ductility Shear

$\phi_{v,s} V_{sa}$ =	70.1 [kips]			
>	$\phi_{v,c} \min(V_{cbg}, V_{cpg})$	= 17.2	[kips]	
	Non-ductile			

Example 14: Anchor Bolt + No Anchor Reinft + Tension Shear & Moment + CSA A23.3-04 Code



M_u = 33.9 kNm N_u= 44.5 kN (Compression) V_u = 44.5 kN

Concrete f_c'= 34.5 MPa

Anchor bolt F1554 Grade 36 1.25" dia Heavy Hex Head h_{ef} = 406mm h_a =508mm

Oversized holes in base plate

Seismic design $I_E F_a S_a(0.2) < 0.35$

Supplementary reinforcement Tension → Condition A

Shear → Condition A $\Psi_{c,V}$ = 1.2

Provide built-up grout pad

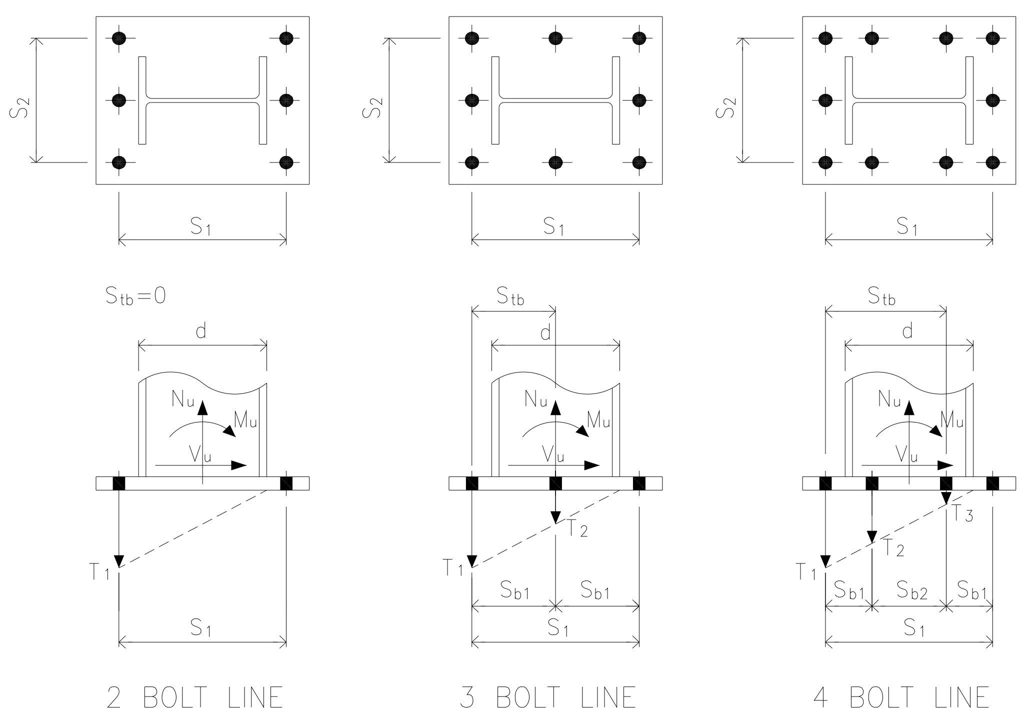
ANCHOR BOLT DESIGN **Combined Tension, Shear and Moment**

Anchor bolt design based on	**Code Abbreviation**
CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D	*A23.3-04 (R2010)*
ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary	*ACI318 M-08*
PIP STE05121 Anchor Bolt Design Guide-2006	*PIP STE05121*

Assumptions	**Code Reference**
	A23.3-04 (R2010)
1. Concrete is cracked	
2. Condition A for tension - supplementary reinforcement provided	D.5.4 (c)
3. Shear load acts through center of bolt group $\Psi_{ec,V}$ =1.0	D.7.2.5
4. For anchor group subject to moment, the anchor tensile load is designed using elastic analysis and there is no redistribution of the forces between highly stressed and less stressed anchors	D.4.1
5. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output	
6. Shear carried by only half of total anchor bolts due to oversized holes in column base plate	*AISC Design Guide 1* section 3.5.3

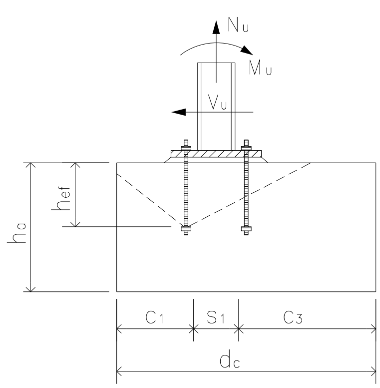
Anchor Bolt Data

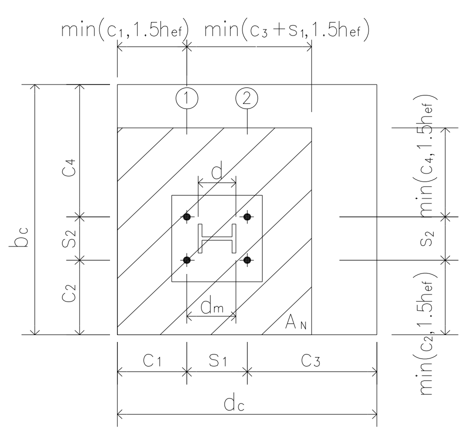
Factored moment	M_u =	33.9	[kNm]		= 25.0	[kip-ft]
Factored tension /compression	N_u =	-44.5	[kN]	in compression	= -10.0	[kips]
Factored shear	V_u =	44.5	[kN]		= 10.0	[kips]
Factored shear for bolt design	$\mathbf{V_u}$ =	**44.5**	[kN]	V_u = 0 if shear key is provided		



No of bolt line for resisting moment	=	3 Bolt Line
No of bolt along outermost bolt line	=	3

				min required		Code Reference
Outermost bolt line spacing s_1	s_1 = 406	[mm]		127	OK	*PIP STE05121*
Outermost bolt line spacing s_2	s_2 = 406	[mm]		127	OK	Page A -1 Table 1
Internal bolt line spacing s_{b1}	s_{b1} = 203	[mm]		127	OK	
Internal bolt line spacing s_{b2}	s_{b2} = 0	[mm]		127	OK	
Column depth	d = 323	[mm]				
Concrete strength	f'_c = 35	[MPa]	= 5.0 [ksi]			
Anchor bolt material	= F1554 Grade 36					
Anchor tensile strength	f_{uta} = 58	[ksi]	= 400 [MPa]			*A23.3-04 (R2010)*
	Anchor is ductile steel element					D.2
Anchor bolt diameter	d_a = **1.25**	[in]	= 31.8 [mm]			*PIP STE05121*
Bolt sleeve diameter	d_s = 76	[mm]				Page A -1 Table 1
Bolt sleeve height	h_s = 254	[mm]				
				min required		
Anchor bolt embedment depth	h_{ef} = 406	[mm]		381	OK	Page A -1 Table 1
Concrete thickness	h_a = 508	[mm]		482	OK	
Bolt edge distance c_1	c_1 = 152	[mm]		127	OK	Page A -1 Table 1
Bolt edge distance c_2	c_2 = 152	[mm]		127	OK	
Bolt edge distance c_3	c_3 = 2540	[mm]		127	OK	
Bolt edge distance c_4	c_4 = 2540	[mm]		127	OK	*A23.3-04 (R2010)*
$c_i > 1.5h_{ef}$ for at least two edges to avoid reducing of h_{ef} when $N_u > 0$					Yes	D.6.2.3
Adjusted h_{ef} for design	h_{ef} = **406**	[mm]		381	OK	D.6.2.3





						Code Reference
Number of bolt at bolt line 1	n_1 =	3				
Number of bolt at bolt line 2	n_2 =	3				
Total no of anchor bolt	n =	8				
Number of bolt carrying tension	n_t =	5				
Number of bolt carrying shear	n_s =	4				
Oversized holes in base plate ?	=	Yes	?			
Anchor head type	=	Heavy Hex		?		
	A_{se} =	625	[mm^2]			
Bearing area of head	A_{brg} =	**1443**	[mm^2]			
	A_{brg}		[mm^2]	not applicable		
Bolt 1/8" (3mm) corrosion allowance	=	No	?			
Provide shear key ?	=	No	?			*A23.3-04 (R2010)*
Seismic region where $I_EF_aS_a(0.2)$>=0.35	=	No	?			D.4.3.5
Supplementary reinforcement						
For tension	=	Yes	Condition A			D.5.4 (c)
For shear	$\Psi_{c,V}$ =	1.2	Condition A	?		D.7.2.7
Provide built-up grout pad ?	=	Yes	?			D.7.1.3

Strength reduction factors						
Anchor reinforcement factor	ϕ_{as} =	0.75				D.7.2.9
Steel anchor resistance factor	ϕ_s =	0.85				8.4.3 (a)
Concrete resistance factor	ϕ_c =	0.65				8.4.2
Resistance modification factors						
Anchor rod - ductile steel	$R_{t,s}$ =	0.80		$R_{v,s}$ =	0.75	D.5.4(a)
Concrete	$R_{t,c}$ =	1.15	Cdn-A	$R_{v,c}$ =	1.15 Cdn-A	D.5.4(c)

CONCLUSION

Abchor Rod Embedment, Spacing and Edge Distance		**OK**	
Overall	ratio = **0.81**	**OK**	
Tension			
Anchor Rod Tensile Resistance	ratio = 0.13	**OK**	
Conc. Tensile Breakout Resistance	ratio = 0.39	**OK**	
Anchor Pullout Resistance	ratio = 0.08	**OK**	
Side Blowout Resistance	ratio = 0.13	**OK**	
Shear			
Anchor Rod Shear Resistance	ratio = 0.15	**OK**	
Conc. Shear Breakout Resistance	ratio = 0.58	**OK**	
Conc. Pryout Shear Resistance	ratio = 0.12	**OK**	
Anchor Rod on Conc Bearing	ratio = 0.04	**OK**	
Tension Shear Interaction			
Tension Shear Interaction	ratio = 0.81	**OK**	
Ductility			*A23.3-04 (R2010)*
Tension **Ductile**	**Shear** **Non-ductile**		
Seismic Design Requirement		**OK**	D.4.3.6

IeFaSa(0.2)<0.35, A23.3-04 D.4.3.3 ductility requirement is NOT required

CALCULATION				**Code Reference**
Anchor Tensile Force				*A23.3-04 (R2010)*
Single bolt tensile force	T_1 = 21.6 [kN]	No of bolt for T_1 n_{T1} = 3		
	T_2 = 9.6 [kN]	No of bolt for T_2 n_{T2} = 2		
	T_3 = 0.0 [kN]	No of bolt for T_3 n_{T3} = 0		
Sum of bolt tensile force	$\mathbf{N_u} = \Sigma\, n_i\, T_i$	= **83.9**	[kN]	
Tensile bolts outer distance s_{tb}	s_{tb} = 203 [mm]			
Eccentricity e'_N -- distance between resultant of tensile load and centroid of anchors loaded in tension	e'_N = 35 [mm]			Figure D.8 (b)
Eccentricity modification factor	$\Psi_{ec,N} = \dfrac{1}{\left(1+\dfrac{2e'_N}{3h_{ef}}\right)}$	= 0.95		D.6.2.4 (D-9)
Anchor Rod Tensile Resistance	$N_{sr} = A_{se}\, \phi_s\, f_{uta}\, R_{t,s}$	= 170.0	[kN]	D.6.1.2 (D-3)
	ratio = 0.13	> T_1	**OK**	
Conc. Tensile Breakout Resistance				
	$N_{br} = 10\ \phi_c\ \sqrt{f'_c}\ h_{ef}^{1.5}\ R_{t,c}$ if $h_{ef} \le 275$ or $h_{ef} \ge 625$			D.6.2.2 (D-7)
	$3.9\ \phi_c\ \sqrt{f'_c}\ h_{ef}^{5/3}\ R_{t,c}$ if $275 < h_{ef} < 625$			D.6.2.2 (D-8)
		= 382.8	[kN]	
Projected conc failure area	$1.5h_{ef}$ =	= 609	[mm]	
	A_{Nc} = $[s_{tb}+\min(c_1,1.5h_{ef})+\min(c_3,1.5h_{ef})]$x $[s_2+\min(c_2,1.5h_{ef})+\min(c_4,1.5h_{ef})]$	= 1.1E+06	[mm^2]	
	$A_{Nco} = 9\ h_{ef}^2$	= 1.5E+06	[mm^2]	D.6.2.1 (D-6)
	$A_{Nc} = \min(\ A_{Nc},\ \ n_t\ A_{Nco}\ )$	= 1.1E+06	[mm^2]	D.6.2.1
Min edge distance	$c_{min} = \min(\ c_1,\ c_2,\ c_3,\ c_4\ )$	= 152	[mm]	
Eccentricity effects	$\Psi_{ec,N}$ =	= 0.95		D.6.2.4 (D-9)
Edge effects	$\Psi_{ed,N} = \min[\ (0.7+0.3c_{min}/1.5h_{ef}),\ 1.0\ ]$	= 0.78		D.6.2.5
Concrete cracking	$\Psi_{c,N}$ = 1.0 for cracked concrete			D.6.2.6
Concrete splitting	$\Psi_{cp,N}$ = 1.0 for cast-in anchor			D.6.2.7
Concrete breakout resistance	$N_{cbgr} = \dfrac{A_{Nc}}{A_{Nco}}\ \Psi_{ec,N}\ \Psi_{ed,N}\ \Psi_{c,N}\ \Psi_{cp,N}\ N_{br}$	= 213.0	[kN]	D.6.2.1 (D-5)
Seismic design strength reduction	= x 1.0 not applicable	= 213.0	[kN]	D.4.3.5
	ratio = 0.39	> N_u	**OK**	
Anchor Pullout Resistance				
Single bolt pullout resistance	$N_{pr} = 8\ A_{brg}\ \phi_c\ f'_c\ R_{t,c}$	= 261.2	[kN]	D.6.3.4 (D-16)
	$N_{cpr} = \Psi_{c,p}\, N_{pr}$	= 261.2	[kN]	D.6.3.1 (D-15)
Seismic design strength reduction	= x 1.0 not applicable	= 261.2	[kN]	D.4.3.5
	ratio = 0.08	> T_1	**OK**	
	$\Psi_{c,p}$ = 1 for cracked conc			D.6.3.6
	$R_{t,c}$ = 1.00 pullout strength is always Condition B			D.5.4(c)

Side Blowout Resistance				**Code Reference**
Failure Along Pedestal Width Edge				
Tensile load carried by anchors close to edge which may cause side-face blowout				*ACI318 M-08*
along pedestal width edge	$N_{buw} = n_{T1}\, T_1$	= 64.8	[kN]	RD.5.4.2
	$c = \min(c_1, c_3)$	= 152	[mm]	
Check if side blowout applicable	h_{ef} = 406 [mm]			*A23.3-04 (R2010)*
	> 2.5c side bowout is applicable			D.6.4.1
Check if edge anchors work as a	s_{22} = 203 [mm] $s = s_2$	= 406	[mm]	
a group or work individually	< 6c edge anchors work as a group			D.6.4.2
Single anchor SB resistance	$N_{sbr,w} = 13.3c\sqrt{A_{brg}}\ \phi_c\ \sqrt{f'_c}\ R_{t,c}$	= 339.6	[kN]	D.6.4.1 (D-18)
Multiple anchors SB resistance	$N_{sbgr,w}$ =			
work as a group - applicable	= $(1+s/\ 6c) \times N_{sbr,w}$	= 490.3	[kN]	D.6.4.2 (D-19)
work individually - not applicable	= $n_{bw} \times N_{sbr,w} \times [1+(c_2 \text{ or } c_4)/\ c]\ /\ 4$	= 0.0	[kN]	D.6.4.1
Seismic design strength reduction	= x 1.0 not applicable	= 490.3	[kN]	D.4.3.5
	ratio = 0.13	> N_{buw}	**OK**	
Group side blowout resistance	$N_{sbgr} = \dfrac{N_{sbgr,w}}{n_{T1}} n_t$	= 817.2	[kN]	
Govern Tensile Resistance	$\mathbf{N_r} = \min(n_t N_{sr},\ N_{rbr},\ n_t N_{cpr},\ N_{sbgr})$	= **213.0**	[kN]	

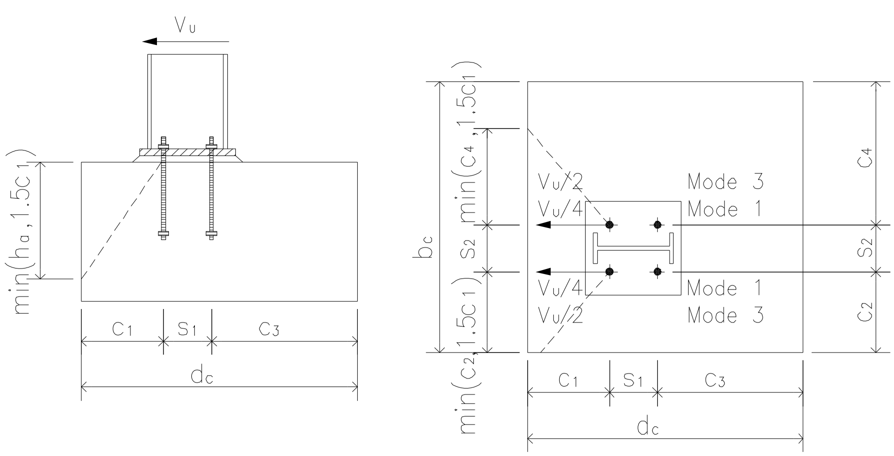
Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear

Anchor Rod Shear Resistance	$V_{sr} = n_s\, A_{se}\, \phi_s\, 0.6\, f_{uta}\, R_{v,s}$	= 382.5	[kN]	D.7.1.2 (b) (D-21)
Reduction due to built-up grout pads	= x 0.8 , applicable	= 306.0	[kN]	D.7.1.3
	ratio = 0.15	> V_u	**OK**	

Conc. Shear Breakout Resistance

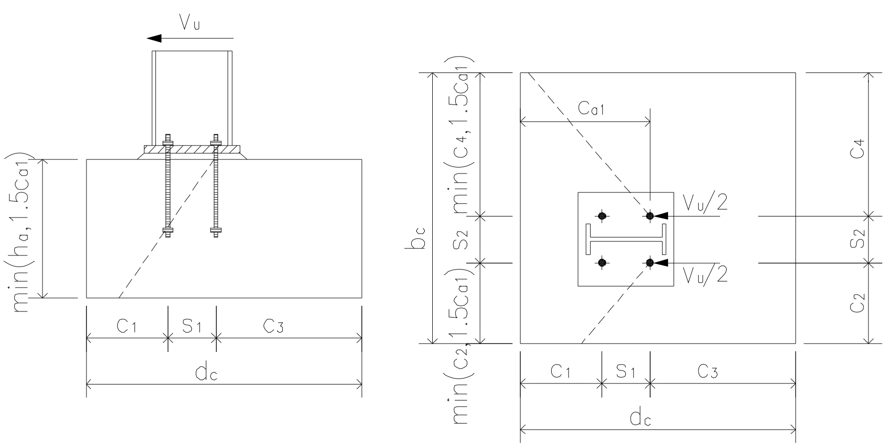
Mode 1 Failure cone at front anchors, strength check against 0.5 x V_u

Mode 3 Failure cone at front anchors, strength check against 1.0 x V_u , applicable when oversized holes are used in base plate



					Code Reference
Bolt edge distance	c_1 =		= 152	[mm]	*A23.3-04 (R2010)*
Limiting c_{a1} when anchors are influenced by 3 or more edges			= No		D.7.2.4
Bolt edge distance - adjusted	$\mathbf{c_1}$ =	ca1 needs NOT to be adjusted	= **152**	[mm]	D.7.2.4
	c_2 =		= 152	[mm]	
	$1.5c_1$ =		= 229	[mm]	
	A_{Vc} =	$[\min(c_2,1.5c_1) + s_2 + \min(c_4,1.5c_1)] \times \min(1.5c_1, h_a)$	= 1.8E+05	[mm^2]	D.7.2.1
	A_{Vco} =	$4.5c_1^2$	= 1.0E+05	[mm^2]	D.7.2.1 (D-24)
	A_{Vc} =	$\min(A_{Vc},\ n_1 A_{Vco})$	= 1.8E+05	[mm^2]	D.7.2.1
	l_e =	$\min(8d_a , h_{ef})$	= 254	[mm]	D.3
	V_{br} =	$0.58\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\ \phi_c\sqrt{f'_c}\ c_{a1}^{1.5}R_{v,c}$	= 41.1	[kN]	D.7.2.2 (D-25)
Eccentricity effects	$\Psi_{ec,v}$ =	1.0 shear acts through center of group			D.7.2.5
Edge effects	$\Psi_{ed,v}$ =	$\min[(0.7+0.3c_2/1.5c_1), 1.0]$	= 0.90		D.7.2.6
Concrete cracking	$\Psi_{c,v}$ =		= 1.20		D.7.2.7
Member thickness	$\Psi_{h,v}$ =	$\max[(\mathrm{sqrt}(1.5c_1 / h_a) , 1.0]$	= 1.00		D.7.2.8
Conc shear breakout resistance	V_{cbgr1} =	$\frac{A_{Vc}}{A_{Vco}}\Psi_{ec,V}\ \Psi_{ed,V}\ \Psi_{c,V}\ \Psi_{h,V}\ V_{br}$	= 76.4	[kN]	D.7.2.1 (D-23)
Mode 3 is used for checking	$\mathbf{V_{cbgr1}}$ =	V_{cbg1} x 1.0	= **76.4**	[kN]	

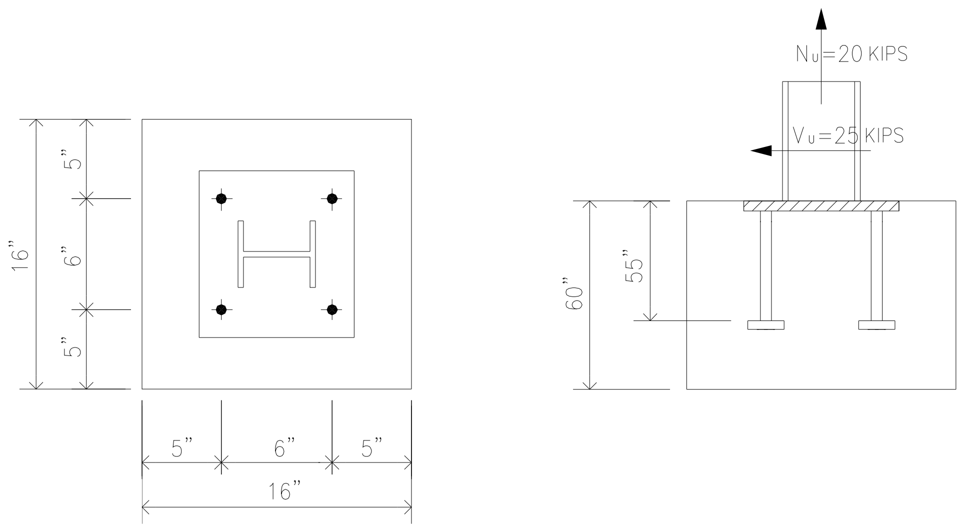
Mode 2 Failure cone at back anchors



					A23.3-04 (R2010)
Bolt edge distance	c_{a1} =	$c_1 + s_1$	= 558	[mm]	
Limiting c_{a1} when anchors are influenced by 3 or more edges			= No		D.7.2.4
Bolt edge distance - adjusted	$\mathbf{c_{a1}}$ =	ca1 needs NOT to be adjusted	= **558**	[mm]	D.7.2.4
	c_2 =		152	[mm]	
	$1.5c_{a1}$ =		838	[mm]	

				Code Reference
	$A_{Vc} = [\min(c_2, 1.5c_{a1}) + s_2 + \min(c_4, 1.5c_{a1})] \times \min(1.5c_{a1}, h_a)$	= 7.1E+05	[mm²]	*A23.3-04 (R2010)* D.7.2.1
	$A_{Vco} = 4.5c_{a1}^2$	= 1.4E+06	[mm²]	D.7.2.1 (D-24)
	$A_{Vc} = \min(A_{Vc},\ n_2 A_{Vco})$	= 7.1E+05	[mm²]	D.7.2.1
	$l_e = \min(8d_a, h_{ef})$	= 254	[mm]	D.3
	$V_{br} = 0.58\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\ \phi_c\sqrt{f_c'}\ c_{a1}^{1.5} R_{v,c}$	= 288.2	[kN]	D.7.2.2 (D-25)
Eccentricity effects	$\Psi_{ec,v}$ = 1.0 shear acts through center of group			D.7.2.5
Edge effects	$\Psi_{ed,v} = \min[(0.7+0.3c_2/1.5c_{a1}), 1.0]$	= 0.75		D.7.2.6
Concrete cracking	$\Psi_{c,v}$ =	= 1.20		D.7.2.7
Member thickness	$\Psi_{h,v} = \max[\text{sqrt}(1.5c_{a1} / h_a), 1.0]$	= 1.28		D.7.2.8
Conc shear breakout resistance	$V_{cbgr2} = \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V}\ \Psi_{ed,V}\ \Psi_{c,V}\ \Psi_{h,V}\ V_{br}$	= 169.4	[kN]	D.7.2.1 (D-23)
Min shear breakout resistance	$V_{cbgr} = \min(V_{cbgr1}, V_{cbgr2})$	= 76.4	[kN]	
Seismic design strength reduction	= x 1.0 not applicable	= 76.4	[kN]	D.4.3.5
	ratio = 0.58	> V_u	**OK**	
Conc. Pryout Shear Resistance				
	k_{cp} = 2.0			D.7.3
Factored shear pryout resistance	$V_{cpgr} = k_{cp} N_{cbgr}$	= 370.4	[kN]	D.7.3 (D-32)
	$R_{v,c}$ = 1.00 pryout strength is always Condition B			D.5.4(c)
Seismic design strength reduction	= x 1.0 not applicable	= 370.4	[kN]	D.4.3.5
	ratio = 0.12	> V_u	**OK**	
				CSA S16-09
Anchor Rod on Conc Bearing	$B_r = n_s \times 1.4 \times \phi_c \times \min(8d_a, h_{ef}) \times d_a \times f_c'$	= 1021.5	[kN]	25.3.3.2
	ratio = 0.04	> V_u	**OK**	
Govern Shear Resistance	$\mathbf{V_r} = \min(V_{sr}, V_{cbgr}, V_{cpgr}, B_r)$	= **76.4**	[kN]	
Tension Shear Interaction				*A23.3-04 (R2010)*
Check if $N_u > 0.2\ N_r$ and $V_u > 0.2\ V_r$	Yes			D.8.2 & D.8.3
	$N_u/N_r + V_u/V_r$	= 0.98		D.8.4 (D-35)
	ratio = 0.81	< 1.2	**OK**	
Ductility Tension				
	N_{sr} = 170.0 [kN]			
	< $\min(N_{cbgr}, N_{cpr}, N_{sbgr})$	= 213.0	[kN]	
	Ductile			
Ductility Shear				
	V_{sr} = 306.0 [kN]			
	> $\min(V_{cbgr}, V_{cpgr}, B_r)$	= 76.4	[kN]	
	Non-ductile			

Example 21: Welded Stud + Anchor Reinft + Tension & Shear + ACI 318-08 Code



N_u= 20 kips (Tension)		V_u = 25 kips		
Concrete	f_c'= 4 ksi	Rebar f_y = 60 ksi		
Pedestal size	16" x 16"			
Anchor stud	AWS D1.1 Grade B	1.0" dia	h_{ef} = 55"	h_a =60"
Seismic design category >= C				
Anchor reinforcement		Tension → 8-No 8 ver. bar		
		Shear → 2-layer, 4-leg No 4 hor. bar		

No built-up grout pad for embedded plate.

Note: The stud length used in this example may not be commercially available and it's for illustration purpose only.

Deep anchor stud embedment h_{ef} is required for anchor reinforcement to develop resistance on both sides of the failure plane.

STUD ANCHOR DESIGN **Combined Tension and Shear**

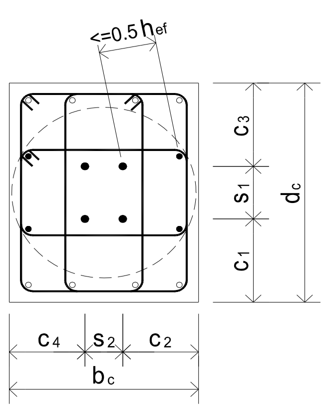
	Code Abbreviation
Anchor bolt design based on	
ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D	*ACI 318-08*
PIP STE05121 Anchor Bolt Design Guide-2006	*PIP STE05121*

	Code Reference
Assumptions	*ACI 318-08*
1. Concrete is cracked	
2. Condition A - supplementary reinforcement is provided	D.4.4 (c)
3. Load combinations shall be as per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2	D.4.4
4. Anchor reinft strength is used to replace concrete tension / shear breakout strength as per ACI318-08 Appendix D clause D.5.2.9 and D.6.2.9	D.5.2.9 & D.6.2.9
5. For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective	
6. Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft	

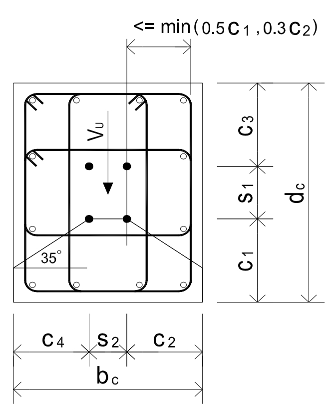
Input Data

set N_u = 0 if it's compression

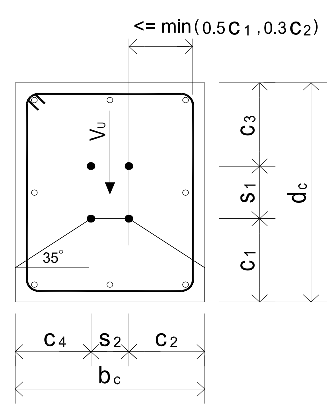
Factored tension	N_u =	**20.0**	[kips]		= 89.0	[kN]	
Factored shear	V_u =	**25.0**	[kips]		= 111.2	[kN]	
Concrete strength	f'_c =	4.0	[ksi]		= 27.6	[MPa]	
Stud material	=	AWS D1.1 Grade B					
Stud tensile strength	f_{uta} =	65	[ksi]		= 448	[MPa]	*ACI 318-08*
		Stud is ductile steel element					D.1
Stud diameter	d_a =	**1**	[in]		= 25.4	[mm]	
Stud shank area	A_{se} =	0.79	[in^2]		= 507	[mm^2]	
Stud head bearing area	A_{brg} =	1.29	[in^2]		= 831	[mm^2]	
				min required			*PIP STE05121*
Stud embedment depth	h_{ef} =	55.0	[in]	12.0	**OK**		Page A -1 Table 1
Pedestal height	h =	60.0	[in]	58.0	**OK**		
Pedestal width	b_c =	16.0	[in]				
Pedestal depth	d_c =	16.0	[in]				



Ver. Reinft For Tension



Hor. Ties For Shear - 4 Legs



Hor. Ties For Shear - 2 Legs

				min required		Code Reference
Stud edge distance c_1	c_1 =	5.0	[in]	4.5	OK	**Code Reference**
Stud edge distance c_2	c_2 =	5.0	[in]	4.5	OK	*PIP STE05121*
Stud edge distance c_3	c_3 =	5.0	[in]	4.5	OK	Page A -1 Table 1
Stud edge distance c_4	c_4 =	5.0	[in]	4.5	OK	
Outermost stud line spacing s_1	s_1 =	6.0	[in]	4.0	OK	Page A -1 Table 1
Outermost stud line spacing s_2	s_2 =	6.0	[in]	4.0	OK	

ACI 318-08

To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within $0.5h_{ef}$ from the outmost stud's centerline. In this design $0.5h_{ef}$ value is limited to 8 in. — RD.5.2.9

$0.5h_{ef}$ = **8.0** [in]

No of ver. rebar that are effective for resisting anchor tension n_v = 8

Ver. bar size No. 8 : 1.000 [in] dia single bar area A_s = 0.79 [in^2]

To be considered effective for resisting anchor shear, hor. reinft shall be located within min($0.5c_1$, $0.3c_2$) from the outmost stud's centerline — RD.6.2.9

min($0.5c_1$, $0.3c_2$) = **1.5** [in]

No of tie leg that are effective to resist anchor shear n_{leg} = 4 ?

No of tie layer that are effective to resist anchor shear n_{lay} = 2 ?

Hor. tie bar size No. 4 : 0.500 [in] dia single bar area A_s = 0.20 [in^2]

For anchor reinft shear breakout strength calc: 100% hor. tie bars develop full yield strength ?

Rebar yield strength f_y = 60 [ksi] suggest 60 = 414 [MPa]

Total no of welded stud n = 4

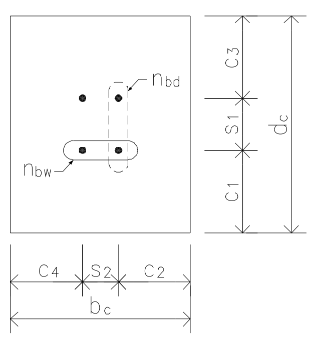
Number of stud carrying tension n_t = 4

Number of stud carrying shear n_s = 4

For side-face blowout check use

No of stud along width edge n_{bw} = 2

No of stud along depth edge n_{bd} = 2



Bolt No Input for Side-Face Blowout Check Use

ACI 318-08

Seismic design category >= C		= Yes ?				D.3.3.3
Provide built-up grout pad ?		= No ?				D.6.1.3
Strength reduction factors						
Anchor reinforcement	ϕ_s =	0.75				D.5.2.9 & D.6.2.9
Anchor rod - ductile steel	$\phi_{t,s}$ =	0.75	$\phi_{v,s}$ =	0.65		D.4.4(a)
Concrete - condition A	$\phi_{t,c}$ =	0.75	$\phi_{v,c}$ =	0.75		D.4.4(c)

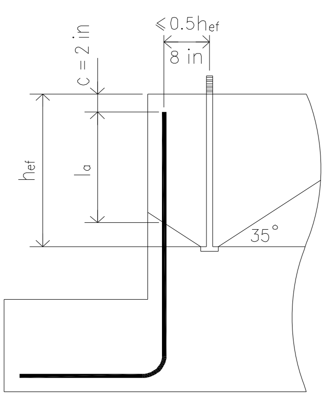
CONCLUSION			**Code Reference**
Abchor Rod Embedment, Spacing and Edge Distance		**OK**	*ACI 318-08*
Min Rquired Anchor Reinft. Development Length	ratio = 0.25	**OK**	12.2.1
Overall	ratio = **0.60**	**OK**	
Tension			
Stud Tensile Resistance	ratio = 0.13	**OK**	
Anchor Reinft Tensile Breakout Resistance	ratio = 0.09	**OK**	
Stud Pullout Resistance	ratio = 0.23	**OK**	
Side Blowout Resistance	ratio = 0.26	**OK**	
Shear			
Stud Shear Resistance	ratio = 0.19	**OK**	
Anchor Reinft Shear Breakout Resistance			
Strut Bearing Strength	ratio = 0.59	**OK**	
Tie Reinforcement	ratio = 0.46	**OK**	
Conc. Pryout Not Govern When h_{ef} >= $12d_a$		**OK**	
Tension Shear Interaction			
Tension Shear Interaction	ratio = 0.60	**OK**	
Ductility			
Tension **Non-ductile**	**Shear** **Non-ductile**		*ACI 318-08*
Seismic Design Requirement		**NG**	D.3.3.4

SDC>= C, ACI318-08 D.3.3.5 or D.3.3.6 must be satisfied for non-ductile design

CACULATION					**Code Reference**
					ACI 318-08
Stud Tensile Resistance	$\phi_{t,s} N_{sa} = \phi_{t,s}\, n_t\, A_{se}\, f_{uta}$	= 153.2	[kips]		D.5.1.2 (D-3)
	ratio = 0.13	> N_u		**OK**	

Anchor Reinft Tensile Breakout Resistance

Min tension development length for ver. #8 bar	l_d =	= 47.4	[in]		12.2.1, 12.2.2, 12.2.4
Actual development lenngth	$l_a = h_{ef}$ - c (2 in) - 8 in x tan35	= 47.4	[in]		
		> 12.0		**OK**	12.2.1



					ACI 318-08
	$N_{rb} = \phi_s \times f_y \times n_v \times A_s \times (l_a / l_d$, if $l_a < l_d)$	= 284.2	[kips]		12.2.5
Seismic design strength reduction	= x 0.75 applicable	= 213.1	[kips]		D.3.3.3
	ratio = 0.09	> N_u		**OK**	

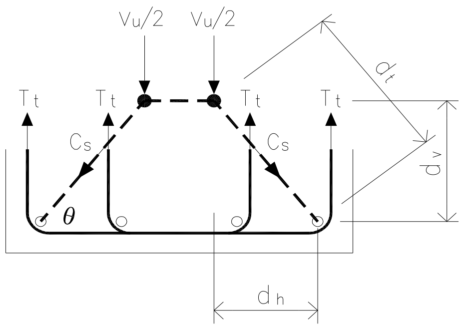
						Code Reference
Stud Pullout Resistance						*ACI 318-08*
Single bolt pullout resistance	N_p	$= 8\,A_{brg}\,f_c'$		= 41.2	[kips]	D.5.3.4 (D-15)
	$N_{cpr} = \phi_{t,c}\,N_{pn}$	$= \phi_{t,c}\,n_t\,\Psi_{c,p}\,N_p$		= 115.5	[kips]	D.5.3.1 (D-14)
Seismic design strength reduction		= x 0.75 applicable		= 86.6	[kips]	D.3.3.3
	ratio	= 0.23		> N_u	**OK**	
	$\Psi_{c,p}$	= 1 for cracked conc				D.5.3.6
	$\phi_{t,c}$	= 0.70	pullout strength is always Condition B			D.4.4(c)
Side Blowout Resistance						
Failure Along Pedestal Width Edge						
Tensile load carried by anchors close to edge which may cause side-face blowout						
along pedestal width edge	N_{buw}	$= N_u \times n_{bw} / n_t$		= 10.0	[kips]	RD.5.4.2
	c	$= \min(c_1, c_3)$		= 5.0	[in]	
Check if side blowout applicable	h_{ef}	= 55.0 [in]				
		> 2.5c	side bowout is applicable			D.5.4.1
Check if edge anchors work as a	s_{22}	= 6.0 [in]	$s = s_2$	= 6.0	[in]	
a group or work individually		< 6c	edge anchors work as a group			D.5.4.2
Single anchor SB resistance	$\phi_{t,c}\,N_{sb}$	$= \phi_{t,c}\left(160\,c\sqrt{A_{brg}}\right)\lambda\sqrt{f'_c}$		= 43.1	[kips]	D.5.4.1 (D-17)
Multiple anchors SB resistance	$\phi_{t,c}N_{sbg,w}$	=				
work as a group - applicable		$= (1+s/\,6c) \times \phi_{t,c}\,N_{sb}$		= 51.7	[kips]	D.5.4.2 (D-18)
work individually - not applicable		$= n_{bw} \times \phi_{t,c}\,N_{sb} \times [1+(c_2 \text{ or } c_4)/\,c] / 4$		= 0.0	[kips]	D.5.4.1
Seismic design strength reduction		= x 0.75 applicable		= 38.8	[kips]	D.3.3.3
	ratio	= 0.26		> N_{buw}	**OK**	
Failure Along Pedestal Depth Edge						
Tensile load carried by anchors close to edge which may cause side-face blowout						
along pedestal depth edge	N_{bud}	$= N_u \times n_{bd} / n_t$		= 10.0	[kips]	RD.5.4.2
	c	$= \min(c_2, c_4)$		= 5.0	[in]	
Check if side blowout applicable	h_{ef}	= 55.0 [in]				
		> 2.5c	side bowout is applicable			D.5.4.1
Check if edge anchors work as a	s_{11}	= 6.0 [in]	$s = s_1$	= 6.0	[in]	
a group or work individually		< 6c	edge anchors work as a group			D.5.4.2
Single anchor SB resistance	$\phi_{t,c}\,N_{sb}$	$= \phi_{t,c}\left(160\,c\sqrt{A_{brg}}\right)\lambda\sqrt{f'_c}$		= 43.1	[kips]	D.5.4.1 (D-17)
Multiple anchors SB resistance	$\phi_{t,c}N_{sbg,d}$	=				
work as a group - applicable		$= (1+s/\,6c) \times \phi_{t,c}\,N_{sb}$		= 51.7	[kips]	D.5.4.2 (D-18)
work individually - not applicable		$= n_{bd} \times \phi_{t,c}\,N_{sb} \times [1+(c_1 \text{ or } c_3)/\,c] / 4$		= 0.0	[kips]	D.5.4.1
Seismic design strength reduction		= x 0.75 applicable		= 38.8	[kips]	D.3.3.3
	ratio	= 0.26		> N_{bud}	**OK**	
Group side blowout resistance	$\phi_{t,c}\,N_{sbg}$	$= \phi_{t,c}\,\min\left(\frac{N_{sbg,w}}{n_{bw}}n_t\,,\ \frac{N_{sbg,d}}{n_{bd}}n_t\right)$		= 77.5	[kips]	
Govern Tensile Resistance	$\mathbf{N_r}$	$= \phi_{t,c}\min(N_s, N_{rb}, N_{cp}, N_{sbg})$		= **77.5**	[kips]	

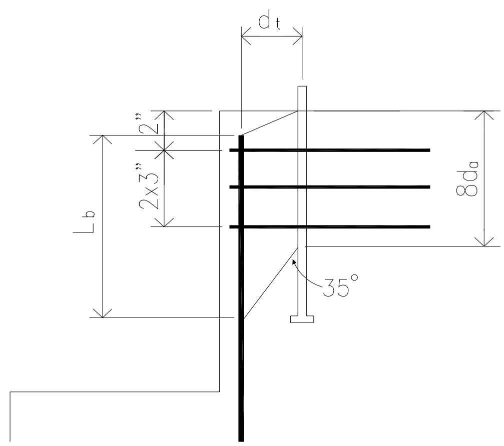
				Code Reference
				ACI 318-08
Stud Shear Resistance	$\phi_{v,s} V_{sa} = \phi_{v,s} n_s A_{se} f_{uta}$	= 132.7	[kips]	D.6.1.2 (a) (D-19)
Reduction due to built-up grout pads	= x 1.0 , not applicable	= 132.7	[kips]	D.6.1.3
	ratio = 0.19	> V_u	**OK**	

Anchor Reinft Shear Breakout Resistance

Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft

STM strength reduction factor	ϕ_{st} = 0.75			9.3.2.6





Strut-and-Tie model geometry	d_v = 2.250	[in]	d_h = 2.250	[in]	
	θ = 45		d_t = 3.182	[in]	
Strut compression force	C_s = 0.5 V_u / sinθ		= 17.7	[kips]	
					ACI 318-08
Strut Bearing Strength					
Strut compressive strength	f_{ce} = 0.85 f'_c		= 3.4	[ksi]	A.3.2 (A-3)
* Bearing of anchor bolt					
Anchor bearing length	l_e = min($8d_a$, h_{ef})		= 8.0	[in]	D.6.2.2
Anchor bearing area	A_{brg} = l_e x d_a		= 8.0	[in^2]	
Anchor bearing resistance	C_r = n_s x ϕ_{st} x f_{ce} x A_{brg}		= 81.6	[kips]	
			> V_u	**OK**	
* Bearing of ver reinft bar					
Ver bar bearing area	A_{brg} = (l_e +1.5 x d_t - d_a/2 -d_b/2) x d_b		= 11.8	[in^2]	
Ver bar bearing resistance	C_r = ϕ_{st} x f_{ce} x A_{brg}		= 30.0	[kips]	
	ratio = 0.59		> C_s	**OK**	

Tie Reinforcement

Code Reference

* For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
* For enclosed tie, at hook location the tie cannot develop full yield strength f_y . Use the pullout resistance in tension of a single hooked bolt as per ACI318-08 Eq. (D-16) as the max force can be developed at hook T_h
* Assume 100% of hor. tie bars can develop full yield strength.

Total number of hor tie bar	$n = n_{leg}$ (leg) x n_{lay} (layer)	= 8		
				ACI 318-08
Pull out resistance at hook	$T_h = \phi_{t,c}\ 0.9\ f_c'\ e_h\ d_a$	= 3.0	[kips]	D.5.3.5 (D-16)
	$e_h = 4.5\ d_b$	= 2.250	[in]	
Single tie bar tension resistance	$T_r = \phi_s \times f_y \times A_s$	= 9.0	[kips]	
Total tie bar tension resistance	V_{rb} = 1.0 x n x Tr	= 72.0	[kips]	
Seismic design strength reduction	= x 0.75 applicable	= 54.0	[kips]	D.3.3.3
	ratio = 0.46	> V_u	**OK**	

Conc. Pryout Shear Resistance

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} > = $12d_a$, the pryout failure will not govern

$12d_a$ = 12.0	[in]	h_{ef} = 55.0	[in]	
		> $12d_a$	**OK**	

Govern Shear Resistance	$\mathbf{V_r}$ = min ($\phi_{v,s}V_{sa}$, V_{rb})	= **54.0**	[kips]

Tension Shear Interaction

Check if $N_u > 0.2\phi\ N_n$ and $V_u > 0.2\phi\ V_n$	Yes			D.7.1 & D.7.2
	$N_u/N_r + V_u/V_r$	= 0.72		D.7.3 (D-32)
	ratio = 0.60	< 1.2	**OK**	

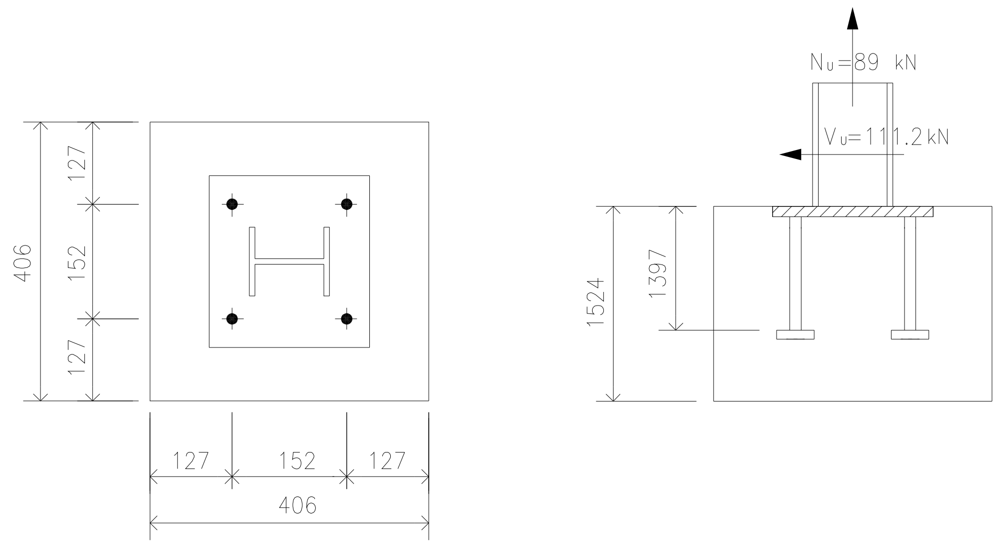
Ductility Tension

$\phi_{t,s}\ N_{sa}$ = 153.2	[kips]		
> min [N_{rb} , $\phi_{t,c}$ (N_{pn}, N_{sbg})]		= 77.5	[kips]
	Non-ductile		

Ductility Shear

$\phi_{t,s}\ N_{sa}$ = 132.7	[kips]		
> V_{rb}		= 54.0	[kips]
	Non-ductile		

Example 22: Welded Stud + Anchor Reinft + Tension & Shear + CSA A23.3-04 Code



N_u= 89 kN (Tension) V_u = 111.2 kN

Concrete f_c'= 27.6 MPa Rebar f_y = 414 MPa

Pedestal size 406mm x 406mm

Anchor stud AWS D1.1 Grade B 1.0" dia h_{ef} = 1397mm h_a =1524mm

Seismic design $I_E F_a S_a(0.2)$ >= 0.35

Anchor reinforcement Tension → 8-25M ver. bar

Shear → 2-layer, 4-leg 15M hor. bar

No built-up grout pad for embedded plate.

Note: The stud length used in this example may not be commercially available and it's for illustration purpose only.

Deep anchor stud embedment h_{ef} is required for anchor reinforcement to develop resistance on both sides of the failure plane.

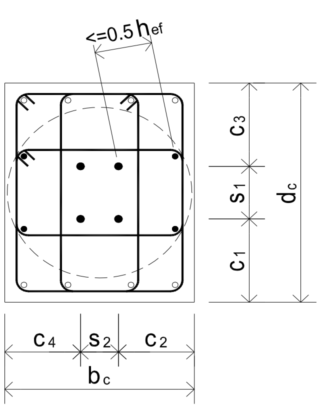
STUD ANCHOR DESIGN **Combined Tension and Shear**

Anchor bolt design based on	**Code Abbreviation**
CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D	*A23.3-04 (R2010)*
ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary	*ACI318 M-08*
PIP STE05121 Anchor Bolt Design Guide-2006	*PIP STE05121*

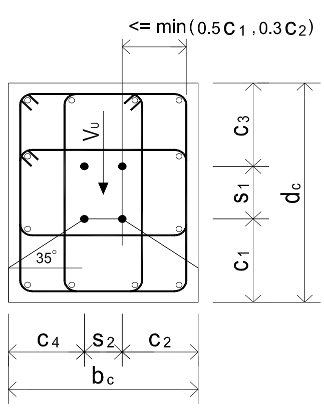
Assumptions	**Code Reference**
1. Concrete is cracked	*A23.3-04 (R2010)*
2. Condition A - supplementary reinforcement is provided	D.5.4 (c)
3. Anchor reinft strength is used to replace concrete tension / shear breakout strength as per	*ACI318 M-08*
ACI318 M-08 Appendix D clause D.5.2.9 and D.6.2.9	D.5.2.9 & D.6.2.9
4. For tie reinft, only the top most 2 or 3 layers of ties (50mm from TOC and 2x75mm after) are effective	
5. Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft	

Input Data

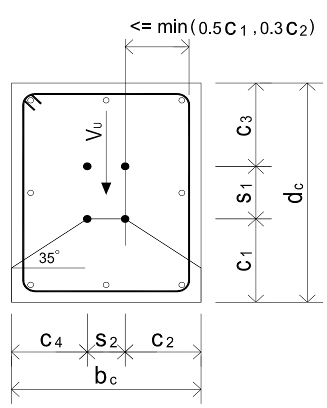
		set N_u = 0 if it's compression					
Factored tension	$\mathbf{N_u}$ =	**89.0**	[kN]		= 20.0	[kips]	
Factored shear	$\mathbf{V_u}$ =	**111.2**	[kN]		= 25.0	[kips]	
Concrete strength	f'_c =	28	[MPa]		= 4.0	[ksi]	
Stud material	=	AWS D1.1 Grade B					
Stud tensile strength	f_{uta} =	65	[ksi]		= 448	[MPa]	*A23.3-04 (R2010)*
		Stud is ductile steel element					D.2
Stud diameter	$\mathbf{d_a}$ =	**1**	[in]		= 25.4	[mm]	
Stud shank area	A_{se} =	0.79	[in^2]		= 507	[mm^2]	
Stud head bearing area	A_{brg} =	1.29	[in^2]		= 831	[mm^2]	
				min required			*PIP STE05121*
Anchor bolt embedment depth	h_{ef} =	1397	[mm]	305	**OK**		Page A -1 Table 1
Pedestal height	h =	1524	[mm]	1473	**OK**		
Pedestal width	b_c =	406	[mm]				
Pedestal depth	d_c =	406	[mm]				



Ver. Reinft For Tension



Hor. Ties For Shear - 4 Legs



Hor. Ties For Shear - 2 Legs

				min required		
Stud edge distance c_1	c_1 =	127	[mm]	115	**OK**	**Code Reference**
Stud edge distance c_2	c_2 =	127	[mm]	115	**OK**	*PIP STE05121*
Stud edge distance c_3	c_3 =	127	[mm]	115	**OK**	Page A -1 Table 1
Stud edge distance c_4	c_4 =	127	[mm]	115	**OK**	
Outermost stud line spacing s_1	s_1 =	152	[mm]	102	**OK**	Page A -1 Table 1
Outermost stud line spacing s_2	s_2 =	152	[mm]	102	**OK**	

ACI318 M-08

To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within $0.5h_{ef}$ from the outmost anchor's centerline. In this design $0.5h_{ef}$ value is limited to 200mm. RD.5.2.9

$0.5h_{ef}$ = **200** [mm]

No of ver. rebar that are effective for resisting anchor tension n_v = 8

Ver. bar size d_b = 25 single bar area A_s = 500 [mm^2]

To be considered effective for resisting anchor shear, hor. reinft shall be located within min($0.5c_1$, $0.3c_2$) from the outmost anchor's centerline min($0.5c_1$, $0.3c_2$) = **38** [mm] RD.6.2.9

No of tie leg that are effective to resist anchor shear n_{leg} = 4 ?

No of tie layer that are effective to resist anchor shear n_{lay} = 2 ?

Hor. bar size d_b = 15 single bar area A_s = 200 [mm^2]

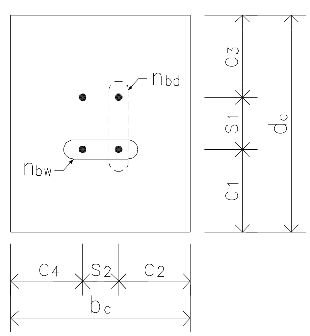
For anchor reinft shear breakout strength calc: 100% hor. tie bars develop full yield strength ?

				suggest		
Rebar yield strength	f_y =	414	[MPa]	400	= 60.0	[ksi]

Total no of welded stud	n =	4
No of stud carrying tension	n_t =	4
No of stud carrying shear	n_s =	4

For side-face blowout check use

No of stud along width edge	n_{bw} =	2
No of stud along depth edge	n_{bd} =	2



Bolt No Input for Side-Face Blowout Check Use

A23.3-04 (R2010)

Seismic region where $I_EF_aS_a(0.2)$>=0.35	=	Yes ?	D.4.3.5
Provide built-up grout pad ?	=	No ?	D.7.1.3

Strength reduction factors

Anchor reinforcement factor	ϕ_{as} =	0.75	D.7.2.9
Steel anchor resistance factor	ϕ_s =	0.85	8.4.3 (a)
Concrete resistance factor	ϕ_c =	0.65	8.4.2

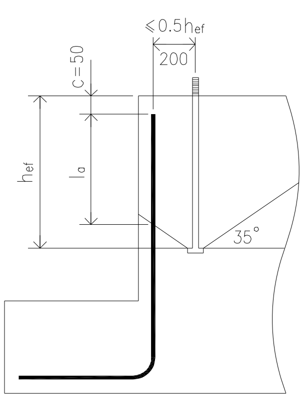
Resistance modification factors

Anchor rod - ductile steel	$R_{t,s}$ =	0.80	$R_{v,s}$ =	0.75	D.5.4(a)
Concrete - condition A	$R_{t,c}$ =	1.15	$R_{v,c}$ =	1.15	D.5.4(c)

CONCLUSION			**Code Reference**
Abchor Rod Embedment, Spacing and Edge Distance		**OK**	*A23.3-04 (R2010)*
Min Rquired Anchor Reinft. Development Length	ratio = 0.25	**OK**	12.2.1
Overall	ratio = **0.60**	**OK**	
Tension			
Stud Tensile Resistance	ratio = 0.14	**OK**	
Anchor Reinft Tensile Breakout Resistance	ratio = 0.10	**OK**	
Stud Pullout Resistance	ratio = 0.25	**OK**	
Side Blowout Resistance	ratio = 0.26	**OK**	
Shear			
Stud Shear Resistance	ratio = 0.19	**OK**	
Anchor Reinft Shear Breakout Resistance			
Strut Bearing Strength	ratio = 0.60	**OK**	
Tie Reinforcement	ratio = 0.30	**OK**	
Conc. Pryout Not Govern When h_{ef} >= $12d_a$		**OK**	
Stud on Conc Bearing	ratio = 0.21	**OK**	
Tension Shear Interaction			
Tension Shear Interaction	ratio = 0.46	**OK**	
Ductility	**Tension** **Non-ductile**	**Shear** **Non-ductile**	*A23.3-04 (R2010)*
Seismic Design Requirement		**NG**	D.4.3.6

IeFaSa(0.2)>=0.35, A23.3-04 D.4.3.7 or D.4.3.8 must be satisfied for non-ductile design

CACULATION				**Code Reference**
				A23.3-04 (R2010)
Stud Tensile Resistance	$N_{sr} = n_t\ A_{se}\ \phi_s\ f_{uta}\ R_{t,s}$	= 617.7	[kN]	D.6.1.2 (D-3)
	ratio = 0.14	> N_u	**OK**	
Anchor Reinft Tensile Breakout Resistance				
Min tension development length for ver. 25M bar	l_d =	= 887	[mm]	12.2.3
Actual development lenngth	$l_a = h_{ef}$ - c (50mm) - 200mm x tan35	= 1207	[mm]	
		> 300	**OK**	12.2.1



	$N_{rbr} = \phi_{as}$ x f_y x n_v x A_s x (l_a / l_{d} , if $l_a < l_d$)	= 1242.0	[kN]	12.2.5
Seismic design strength reduction	= x 0.75 applicable	= 931.5	[kN]	D.4.3.5
	ratio = 0.10	> N_u	**OK**	

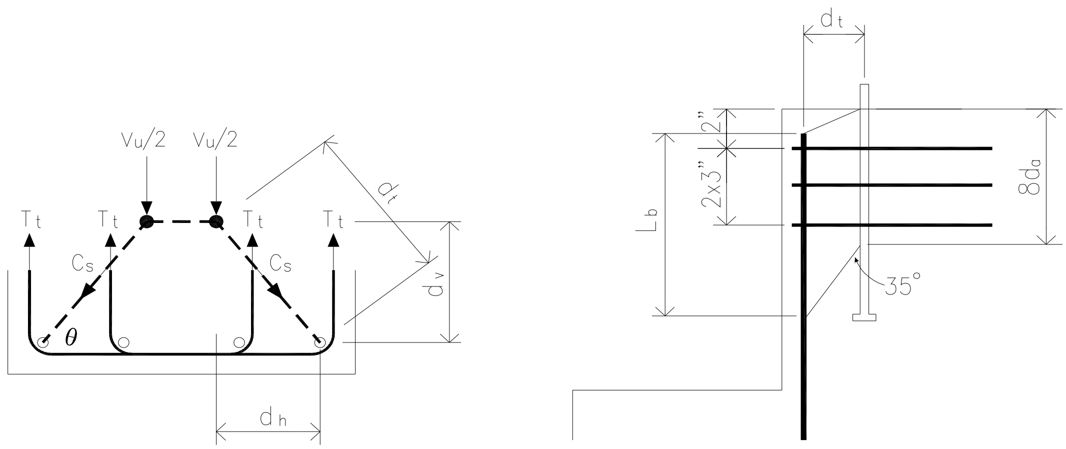
					Code Reference
Stud Pullout Resistance					*A23.3-04 (R2010)*
Single bolt pullout resistance	N_{pr} =	$8\ A_{brg}\ \phi_c\ f_c'\ R_{t,c}$	= 119.3	[kN]	D.6.3.4 (D-16)
	N_{cpr} =	$n_t\ \Psi_{c,p}\ N_{pr}$	= 477.2	[kN]	D.6.3.1 (D-15)
Seismic design strength reduction	=	x 0.75 applicable	= 357.9	[kN]	D.4.3.5
	ratio =	0.25	> N_u	OK	
	$\Psi_{c,p}$ =	1 for cracked conc			D.6.3.6
	$R_{t,c}$ =	1.00 pullout strength is always Condition B			D.5.4(c)
Side Blowout Resistance					
<u>Failure Along Pedestal Width Edge</u>					
Tensile load carried by anchors close to edge which may cause side-face blowout					*ACI318 M-08*
along pedestal width edge	N_{buw} =	N_u x n_{bw} / n_t	= 44.5	[kN]	RD.5.4.2
	c =	min (c_1, c_3)	= 127	[mm]	
Check if side blowout applicable	h_{ef} =	1397 [mm]			*A23.3-04 (R2010)*
	>	2.5c side bowout is applicable			D.6.4.1
Check if edge anchors work as a	s_{22} =	152 [mm]	s = s_2 = 152	[mm]	
a group or work individually	<	6c edge anchors work as a group			D.6.4.2
Single anchor SB resistance	$N_{sbr,w}$ =	$13.3c\sqrt{A_{brg}}\ \phi_c\ \sqrt{f'_c}\ R_{t,c}$	= 191.3	[kN]	D.6.4.1 (D-18)
Multiple anchors SB resistance	$N_{sbgr,w}$ =				
work as a group - applicable	=	(1+s/ 6c) x $N_{sbr,w}$	= 229.4	[kN]	D.6.4.2 (D-19)
work individually - not applicable	=	n_{bw} x $N_{sbr,w}$ x [1+(c_2 or c_4) / c] / 4	= 0.0	[kN]	D.6.4.1
Seismic design strength reduction	=	x 0.75 applicable	= 172.1	[kN]	D.4.3.5
	ratio =	0.26	> N_{buw}	OK	
<u>Failure Along Pedestal Depth Edge</u>					
Tensile load carried by anchors close to edge which may cause side-face blowout					*ACI318 M-08*
along pedestal depth edge	N_{bud} =	N_u x n_{bd} / n_t	= 44.5	[kN]	RD.5.4.2
	c =	min (c_2, c_4)	= 127	[mm]	
Check if side blowout applicable	h_{ef} =	1397 [mm]			*A23.3-04 (R2010)*
	>	2.5c side bowout is applicable			D.6.4.1
Check if edge anchors work as a	s_{11} =	152 [mm]	s = s_1 = 152	[mm]	
a group or work individually	<	6c edge anchors work as a group			D.6.4.2
Single anchor SB resistance	$N_{sbr,d}$ =	$13.3c\sqrt{A_{brg}}\ \phi_c\ \sqrt{f'_c}\ R_{t,c}$	= 191.3	[kN]	D.6.4.1 (D-18)
Multiple anchors SB resistance	$N_{sbgr,d}$ =				
work as a group - applicable	=	(1+s/ 6c) x $\phi_{t,c}$ $N_{sbr,d}$	= 229.4	[kN]	D.6.4.2 (D-19)
work individually - not applicable	=	n_{bd} x $N_{sbr,d}$ x [1+(c_1 or c_3) / c] / 4	= 0.0	[kN]	D.6.4.1
Seismic design strength reduction	=	x 0.75 applicable	= 172.1	[kN]	D.4.3.5
	ratio =	0.26	> N_{bud}	OK	
Group side blowout resistance	N_{sbgr} =	$\min\left(\frac{N_{sbgr,w}}{n_{bw}} n_t,\ \frac{N_{sbgr,d}}{n_{bd}} n_t\right)$	= 344.1	[kN]	
Govern Tensile Resistance	$\mathbf{N_r}$ =	min (N_{sr}, N_{rbr}, N_{cpr}, N_{sbgr})	= **344.1**	[kN]	

				Code Reference
				A23.3-04 (R2010)
Stud Shear Resistance	$V_{sr} = n_s A_{se} \phi_s f_{uta} R_{v,s}$	= 579.1	[kN]	D.7.1.2 (a) (D-20)
Reduction due to built-up grout pads	= x 1.0 , not applicable	= 579.1	[kN]	D.7.1.3
	ratio = 0.19	> V_u	**OK**	

Anchor Reinft Shear Breakout Resistance *ACI318 M-08*

Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft

STM strength reduction factor ϕ_{st} = 0.75 9.3.2.6



Strut-and-Tie model geometry	d_v = 57 [mm]	d_h = 57	[mm]	
	θ = 45	d_t = 81	[mm]	
Strut compression force	C_s = 0.5 V_u / sinθ	= 78.6	[kN]	
				ACI318 M-08
Strut Bearing Strength				
Strut compressive strength	$f_{ce} = 0.85 f'_c$	= 23.5	[MPa]	A.3.2 (A-3)
* Bearing of anchor bolt				
Anchor bearing length	l_e = min($8d_a$, h_{ef})	= 203	[mm]	D.6.2.2
Anchor bearing area	$A_{brg} = l_e \times d_a$	= 5161	[mm^2]	
Anchor bearing resistance	$C_r = n_s \times \phi_{st} \times f_{ce} \times A_{brg}$	= 363.3	[kN]	
		> V_u	**OK**	
* Bearing of ver reinft bar				
Ver bar bearing area	$A_{brg} = (l_e + 1.5 \times d_t - d_a/2 - d_b/2) \times d_b$	= 7473	[mm^2]	
Ver bar bearing resistance	$C_r = \phi_{st} \times f_{ce} \times A_{brg}$	= 131.5	[kN]	
	ratio = 0.60	> C_s	**OK**	

Tie Reinforcement				**Code Reference**

* For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
* For enclosed tie, at hook location the tie cannot develop full yield strength f_y . Use the pullout resistance in tension of a single J-bolt as per A23.3-04 Annex D Eq. (D-17) as the max force can be developed at hook T_h
* Assume 100% of hor. tie bars can develop full yield strength.

Total number of hor tie bar	$n = n_{leg}$ (leg) x n_{lay} (layer)	= 8		
				A23.3-04 (R2010)
Pull out resistance at hook	$T_h = 0.9\ \phi_c\ f_c'\ e_h\ d_b\ R_{t,c}$	= 16.3	[kN]	D.6.3.5 (D-17)
	$e_h = 4.5\ d_b$	= 68	[mm]	
Single tie bar tension resistance	$T_r = \phi_{as}$ x f_y x A_s	= 62.1	[kN]	
Total tie bar tension resistance	V_{rbr} = 1.0 x n x Tr	= 496.8	[kN]	
Seismic design strength reduction	= x 0.75 applicable	= 372.6	[kN]	D.4.3.5
	ratio = 0.30	> V_u	**OK**	

Conc. Pryout Shear Resistance

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} > = $12d_a$, the pryout failure will not govern

	$12d_a$ = 305 [mm]	h_{ef} = 1397	[mm]	
		> $12d_a$	**OK**	
				CSA S16-09
Stud on Conc Bearing	$B_r = n_s$ x 1.4 x ϕ_c x min($8d_a$, h_{ef}) x d_a x f_c'	= 518.5	[kN]	25.3.3.2
	ratio = 0.21	> V_u	**OK**	
Govern Shear Resistance	$\mathbf{V_r}$ = min (V_{sr} , V_{rbr} , B_r)	= **372.6**	[kN]	
				A23.3-04 (R2010)
Tension Shear Interaction				
Check if N_u >0.2 N_r and V_u >0.2 V_r	Yes			D.8.2 & D.8.3
	$N_u/N_r + V_u/V_r$	= 0.56		D.8.4 (D-35)
	ratio = 0.46	< 1.2	**OK**	

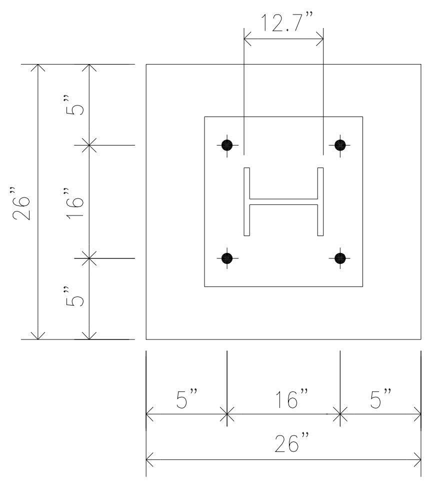
Ductility Tension

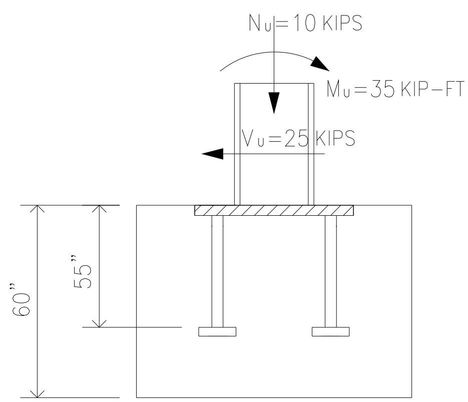
N_{sr} = 617.7 [kN]			
> min (N_{rbr} , N_{cpr}, N_{sbgr})		= 344.1	[kN]
	Non-ductile		

Ductility Shear

V_{sr} = 579.1 [kN]			
> min (V_{rbr}, B_r)		= 372.6	[kN]
	Non-ductile		

Example 23: Welded Stud + Anchor Reinft + Tension Shear & Moment + ACI 318-08 Code





M_u = 35 kip-ft N_u= 10 kips (Compression) V_u = 25 kips

Concrete f_c'= 4 ksi Rebar f_y = 60 ksi

Pedestal size 26” x 26”

Anchor stud AWS D1.1 Grade B 1.0” dia h_{ef} = 55” h_a =60”

Seismic design category < C

Anchor reinforcement Tension → 2-No 8 ver. bar

Shear → 2-layer, 2-leg No 4 hor. bar

No built-up grout pad for embedded plate.

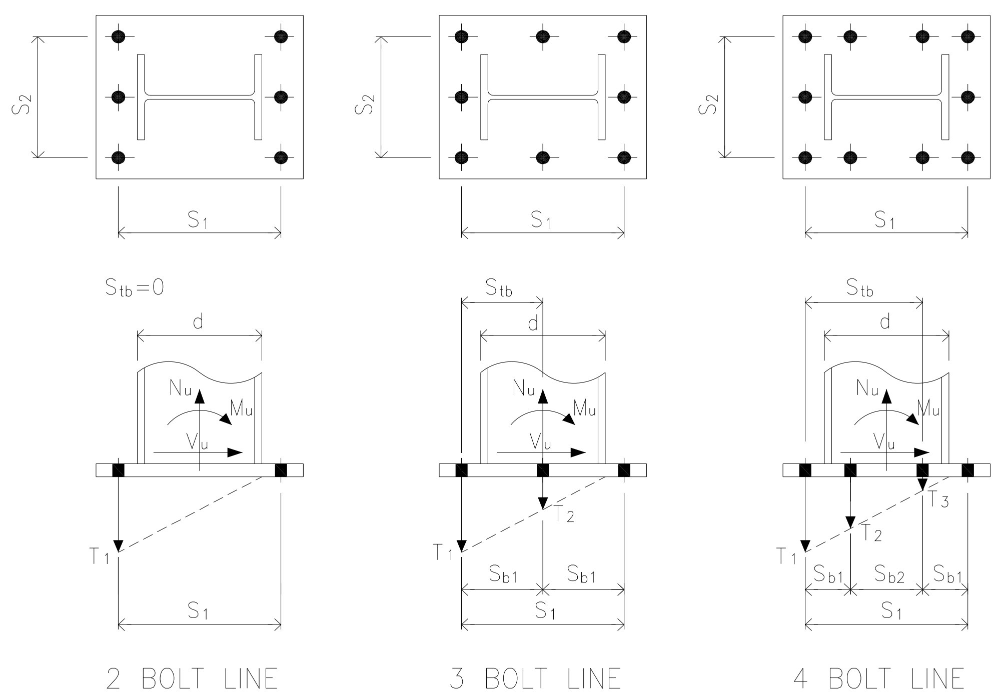
Note: The stud length used in this example may not be commercially available and it's for illustration purpose only. Deep anchor stud embedment h_{ef} is required for anchor reinforcement to develop resistance on both sides of the failure plane.

STUD ANCHOR DESIGN **Combined Tension, Shear and Moment**

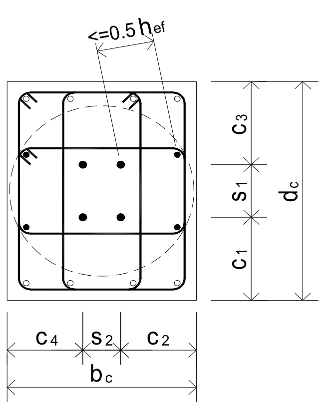
Anchor bolt design based on	**Code Abbreviation**
ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D	*ACI 318-08*
PIP STE05121 Anchor Bolt Design Guide-2006	*PIP STE05121*
	Code Reference
Assumptions	*ACI 318-08*
1. Concrete is cracked	
2. Condition A - supplementary reinforcement is provided	D.4.4 (c)
3. Load combinations shall be as per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2	D.4.4
4. Anchor reinft strength is used to replace concrete tension / shear breakout strength as per ACI318-08 Appendix D clause D.5.2.9 and D.6.2.9	D.5.2.9 & D.6.2.9
5. For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective	
6. Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft	
7. For anchor group subject to moment, the anchor tensile load is designed using elastic analysis and there is no redistribution of the forces between highly stressed and less stressed anchors	D.3.1
8. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output	

Anchor Stud Data

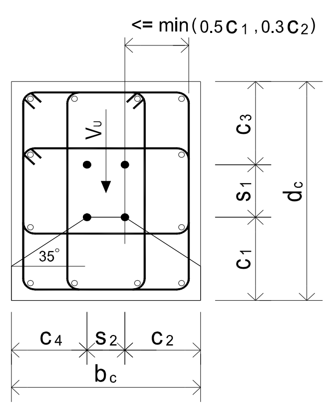
Factored moment	M_u =	35.0	[kip-ft]		= 47.5	[kNm]
Factored tension /compression	N_u =	-10.0	[kips]	in compression	= -44.5	[kN]
Factored shear	$\mathbf{V_u}$ =	**25.0**	[kips]		= 111.2	[kN]



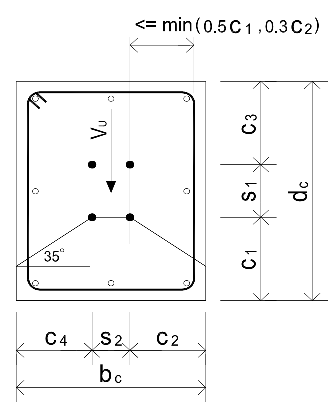
No of bolt line for resisting moment	=	2 Bolt Line					**Code Reference**
No of bolt along outermost bolt line	=	2					
				min required			*PIP STE05121*
Outermost stud line spacing s_1	s_1 =	16.0	[in]	4.0	**OK**		Page A -1 Table 1
Outermost stud line spacing s_2	s_2 =	16.0	[in]	4.0	**OK**		
Internal stud line spacing s_{b1}	s_{b1} =	10.5	[in]	4.0	**OK**		
Internal stud line spacing s_{b2}	s_{b2} =	0.0	[in]	4.0	**OK**		
Column depth	d =	12.7	[in]				
Concrete strength	f'_c =	4.0	[ksi]		= 27.6	[MPa]	
Stud material	=	AWS D1.1 Grade B					
Stud tensile strength	f_{uta} =	65	[ksi]		= 448	[MPa]	*ACI 318-08*
		Stud is ductile steel element					D.1
Stud diameter	$\mathbf{d_a}$ =	**1**	[in]		= 25.4	[mm]	
Stud shank area	A_{se} =	0.79	[in^2]		= 507	[mm^2]	
Stud head bearing area	A_{brg} =	1.29	[in^2]		= 831	[mm^2]	
				min required			*PIP STE05121*
Stud embedment depth	h_{ef} =	55.0	[in]	12.0	**OK**		Page A -1 Table 1
Pedestal height	h =	60.0	[in]	58.0	**OK**		
Pedestal width	b_c =	26.0	[in]				
Pedestal depth	d_c =	26.0	[in]				
Stud edge distance c_1	c_1 =	5.0	[in]	4.5	**OK**		Page A -1 Table 1
Stud edge distance c_2	c_2 =	5.0	[in]	4.5	**OK**		
Stud edge distance c_3	c_3 =	5.0	[in]	4.5	**OK**		
Stud edge distance c_4	c_4 =	5.0	[in]	4.5	**OK**		



Ver. Reinft For Tension

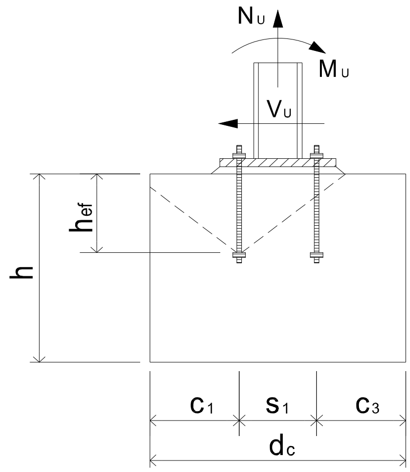


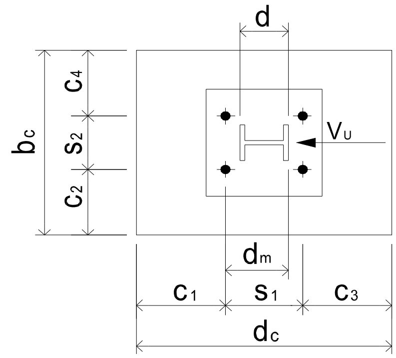
Hor. Ties For Shear - 4 Legs



Hor. Ties For Shear - 2 Legs

Code Reference





ACI 318-08

To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within $0.5h_{ef}$ from the outmost anchor's centerline. In this design $0.5h_{ef}$ value is limited to 8 in. — RD.5.2.9

$0.5h_{ef}$ = **8.0** [in]

No of ver. rebar that are effective for resisting anchor tension n_v = 2

Ver. bar size No. 8 1.000 [in] dia single bar area A_s = 0.79 [in^2]

To be considered effective for resisting anchor shear, hor. reinft shall be located within min($0.5c_1$, $0.3c_2$) from the outmost anchor's centerline — RD.6.2.9

min($0.5c_1$, $0.3c_2$) = **1.5** [in]

No of tie leg that are effective to resist anchor shear n_{leg} = 2 ?

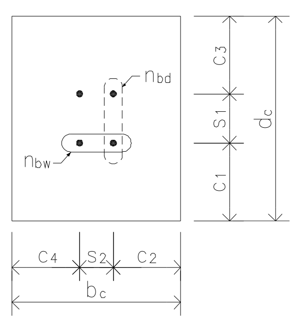
No of tie layer that are effective to resist anchor shear n_{lay} = 2 ?

Hor. tie bar size No. 4 0.500 [in] dia single bar area A_s = 0.20 [in^2]

For anchor reinft shear breakout strength calc 100% hor. tie bars develop full yield strength ?

suggest

Rebar yield strength f_y = 60 [ksi] 60 = 414 [MPa]



Bolt No Input for Side-Face Blowout Check Use

Total no of welded stud n = 4

Number of stud carrying tension n_t = 2

Number of stud carrying shear n_s = 2

For side-face blowout check use

No of stud along width edge n_{bw} = 2

ACI 318-08

Seismic design category >= C = No ? — D.3.3.3

Provide built-up grout pad ? = No ? — D.6.1.3

				Code Reference
Strength reduction factors				*ACI 318-08*
Anchor reinforcement	ϕ_s = 0.75			D.5.2.9 & D.6.2.9
Anchor rod - ductile steel	$\phi_{t,s}$ = 0.75	$\phi_{v,s}$ = 0.65		D.4.4(a)
Concrete - condition A	$\phi_{t,c}$ = 0.75	$\phi_{v,c}$ = 0.75		D.4.4(c)

CONCLUSION

Abchor Rod Embedment, Spacing and Edge Distance		**OK**	
Min Rquired Anchor Reinft. Development Length	ratio = 0.25	**OK**	12.2.1
Overall	ratio = **0.94**	**OK**	
Tension			
Stud Tensile Resistance	ratio = 0.32	**OK**	
Anchor Reinft Tensile Breakout Resistance	ratio = 0.35	**OK**	
Stud Pullout Resistance	ratio = 0.43	**OK**	
Side Blowout Resistance	ratio = 0.38	**OK**	
Shear			
Stud Shear Resistance	ratio = 0.38	**OK**	
Anchor Reinft Shear Breakout Resistance			
Strut Bearing Strength	ratio = 0.59	**OK**	
Tie Reinforcement	ratio = 0.69	**OK**	
Conc. Pryout Not Govern When h_{ef} >= $12d_a$		**OK**	
Tension Shear Interaction			
Tension Shear Interaction	ratio = 0.94	**OK**	
Ductility			
Tension **Non-ductile**	**Shear** **Non-ductile**		
Seismic Design Requirement		**OK**	D.3.3.4

SDC< C, ACI318-08 D.3.3 ductility requirement is NOT required

CACULATION

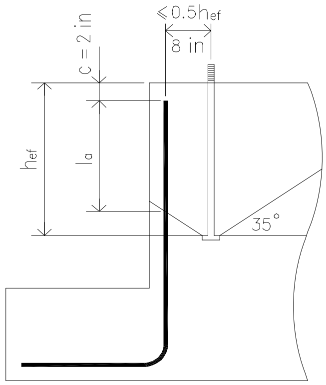
Stud Tensile Force *ACI 318-08*

Single stud tensile force	T_1 = 12.42	[kips]	No of stud for T_1 n_{T1} = 2		
	T_2 = 0.00	[kips]	No of stud for T_2 n_{T2} = 0		
	T_3 = 0.00	[kips]	No of stud for T_3 n_{T3} = 0		
Sum of bolt tensile force	$\mathbf{N_u} = \Sigma\, n_i\, T_i$		= **24.8**	[kips]	
Stud Tensile Resistance	$\phi_{t,s} N_{sa} = \phi_{t,s} A_{se} f_{uta}$		= 38.3	[kips]	D.5.1.2 (D-3)
	ratio = 0.32		> T_1	**OK**	

Anchor Reinft Tensile Breakout Resistance

Min tension development length	l_d =	= 47.4	[in]	12.2.1, 12.2.2, 12.2.4
for ver. #8 bar				
Actual development lenngth	$l_a = h_{ef}$ - c (2 in) - 8 in x tan35	= 47.4	[in]	
		> 12.0	**OK**	12.2.1

Code Reference



ACI 318-08

	N_{rbr} = ϕ_s x f_y x n_v x A_s x (l_a / l_d , if l_a < l_d)	= 71.0	[kips]	12.2.5
Seismic design strength reduction	= x 1.0 not applicable	= 71.0	[kips]	D.3.3.3
	ratio = 0.35	> N_u	**OK**	

Stud Pullout Resistance

Single bolt pullout resistance	$N_p = 8\ A_{brg}\ f_c'$	= 41.2	[kips]	D.5.3.4 (D-15)
	$N_{cpr} = \phi_{t,c}\ N_{pn} = \phi_{t,c}\ \Psi_{c,p}\ N_p$	= 28.9	[kips]	D.5.3.1 (D-14)
Seismic design strength reduction	= x 1.0 not applicable	= 28.9	[kips]	D.3.3.3
	ratio = 0.43	> T_1	**OK**	
	$\Psi_{c,p}$ = 1 for cracked conc			D.5.3.6
	$\phi_{t,c}$ = 0.70 pullout strength is always Condition B			D.4.4(c)

Side Blowout Resistance

<u>Failure Along Pedestal Width Edge</u>

Tensile load carried by anchors close to edge which may cause side-face blowout

along pedestal width edge	$N_{buw} = n_{T1}\ T_1$	= 24.8	[kips]	RD.5.4.2
	c = min (c_1, c_3)	= 5.0	[in]	
Check if side blowout applicable	h_{ef} = 55.0 [in]			
	> 2.5c side bowout is applicable			D.5.4.1
Check if edge anchors work as a	s_{22} = 16.0 [in]	s = s_2 = 16.0	[in]	
a group or work individually	< 6c edge anchors work as a group			D.5.4.2
Single anchor SB resistance	$\phi_{t,c}\ N_{sb} = \phi_{t,c}\left(160\,c\sqrt{A_{brg}}\right)\lambda\sqrt{f'_c}$	= 43.1	[kips]	D.5.4.1 (D-17)
Multiple anchors SB resistance	$\phi_{t,c}N_{sbg,w}$ =			
work as a group - applicable	= (1+s/ 6c) x $\phi_{t,c}$ N_{sb}	= 66.0	[kips]	D.5.4.2 (D-18)
work individually - not applicable	= n_{bw} x $\phi_{t,c}$ N_{sb} x [1+(c_2 or c_4) / c] / 4	= 0.0	[kips]	D.5.4.1
Seismic design strength reduction	= x 1.0 not applicable	= 66.0	[kips]	D.3.3.3
	ratio = 0.38	> N_{buw}	**OK**	
Group side blowout resistance	$\phi_{t,c}\ N_{sbg} = \phi_{t,c}\ \dfrac{N_{sbgr,w}}{n_{T1}}\ n_t$	= 66.0	[kips]	
Govern Tensile Resistance	$\mathbf{N_r} = \phi_{t,c}\ \min(n_t\ N_s,\ N_{rb},\ n_t\ N_{cp},\ N_{sbg})$	= **57.7**	[kips]	

Code Reference

ACI 318-08

Stud Shear Resistance $\phi_{v,s} V_{sa} = \phi_{v,s} n_s A_{se} f_{uta}$ = 66.4 [kips] D.6.1.2 (a) (D-19)

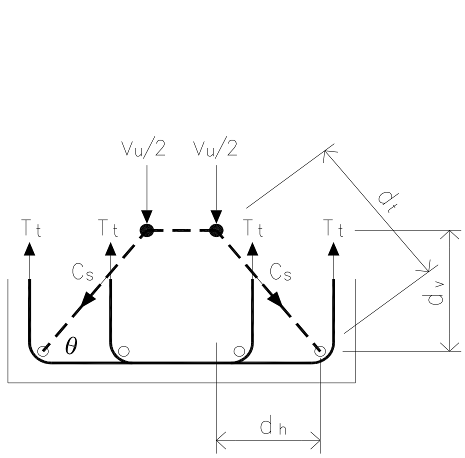
Reduction due to built-up grout pads = x 1.0 , not applicable = 66.4 [kips] D.6.1.3

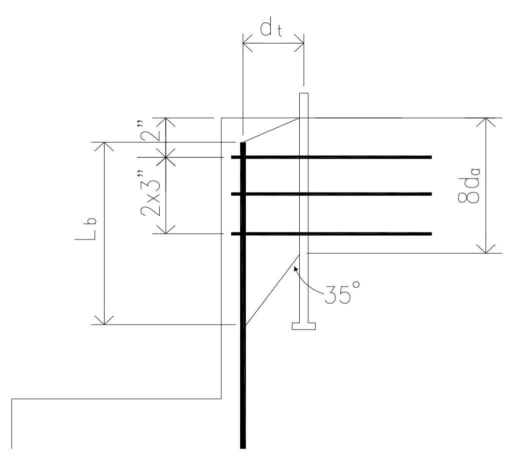
ratio = 0.38 > V_u **OK**

Anchor Reinft Shear Breakout Resistance

Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft

STM strength reduction factor ϕ_{st} = 0.75 9.3.2.6





Strut-and-Tie model geometry d_v = 2.250 [in] d_h = 2.250 [in]

θ = 45 d_t = 3.182 [in]

Strut compression force $C_s = 0.5\, V_u / \sin\theta$ = 17.7 [kips]

ACI 318-08

Strut Bearing Strength

Strut compressive strength $f_{ce} = 0.85\, f'_c$ = 3.4 [ksi] A.3.2 (A-3)

* Bearing of anchor bolt

Anchor bearing length $l_e = \min(8d_a, h_{ef})$ = 8.0 [in] D.6.2.2

Anchor bearing area $A_{brg} = l_e \times d_a$ = 8.0 [in^2]

Anchor bearing resistance $C_r = n_s \times \phi_{st} \times f_{ce} \times A_{brg}$ = 40.8 [kips]

> V_u **OK**

* Bearing of ver reinft bar

Ver bar bearing area $A_{brg} = (l_e + 1.5 \times d_t - d_a/2 - d_b/2) \times d_b$ = 11.8 [in^2]

Ver bar bearing resistance $C_r = \phi_{st} \times f_{ce} \times A_{brg}$ = 30.0 [kips]

ratio = 0.59 > C_s **OK**

Code Reference

Tie Reinforcement *ACI 318-08*

* For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
* For enclosed tie, at hook location the tie cannot develop full yield strength f_y . Use the pullout resistance in tension of a single hooked bolt as per ACI318-08 Eq. (D-16) as the max force can be developed at hook T_h
* Assume 100% of hor. tie bars can develop full yield strength.

Total number of hor tie bar	n = n_{leg} (leg) x n_{lay} (layer)	= 4			
Pull out resistance at hook	T_h = $\phi_{t,c}$ 0.9 f_c' e_h d_a	= 3.0	[kips]		D.5.3.5 (D-16)
	e_h = 4.5 d_b	= 2.250	[in]		
Single tie bar tension resistance	T_r = ϕ_s x f_y x A_s	= 9.0	[kips]		
Total tie bar tension resistance	V_{rb} = 1.0 x n x Tr	= 36.0	[kips]		
Seismic design strength reduction	= x 1.0 not applicable	= 36.0	[kips]		D.3.3.3
	ratio = 0.69	> V_u		**OK**	

Conc. Pryout Shear Resistance

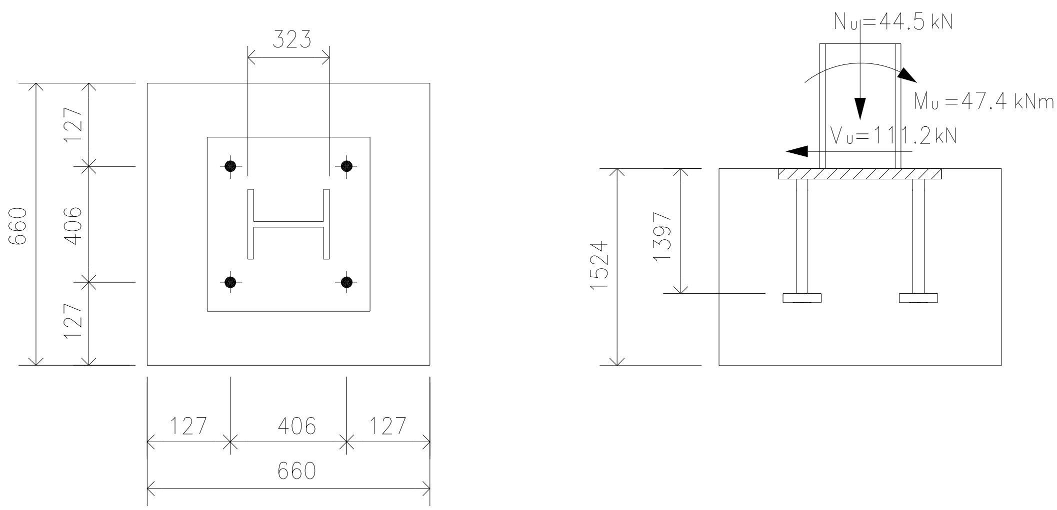
The pryout failure is only critical for short and stiff anchors. It is reasonable to assume that for general cast-in place headed anchors with h_{ef} > = 12d_a , the pryout failure will not govern

	12d_a = 12.0 [in]	h_{ef} = 55.0	[in]		
		> 12d_a		**OK**	
Govern Shear Resistance	$\mathbf{V_r}$ = min ($\phi_{v,s}V_{sa}$, V_{rb})	= **36.0**	[kips]		

Tension Shear Interaction

Check if N_u >0.2ϕ N_n and V_u >0.2ϕ V_n	Yes				D.7.1 & D.7.2
	N_u/N_r + V_u/V_r	= 1.12			D.7.3 (D-32)
	ratio = 0.94	< 1.2		**OK**	
Ductility Tension	$\phi_{t,s}$ N_{sa} = 38.3 [kips]				
	> $\phi_{t,c}$ min (N_{rb}, N_{pn}, N_{sbg})	= 28.9	[kips]		
	Non-ductile				
Ductility Shear	$\phi_{t,s}$ N_{sa} = 66.4 [kips]				
	> V_{rb}	= 36.0	[kips]		
	Non-ductile				

Example 24: Welded Stud + Anchor Reinft + Tension Shear & Moment + CSA A23.3-04 Code



M_u = 47.4 kNm N_u= 44.5 kN (Compression) V_u = 111.2 kN

Concrete f_c'= 27.6 MPa Rebar f_y = 414 MPa

Pedestal size 660mm x 660mm

Anchor stud AWS D1.1 Grade B 1.0" dia h_{ef} = 1397mm h_a =1524mm

Seismic design $I_E F_a S_a(0.2) < 0.35$

Anchor reinforcement Tension → 2-25M ver. bar

Shear → 2-layer, 2-leg 15M hor. bar

No built-up grout pad for embedded plate.

Note: The stud length used in this example may not be commercially available and it's for illustration purpose only.

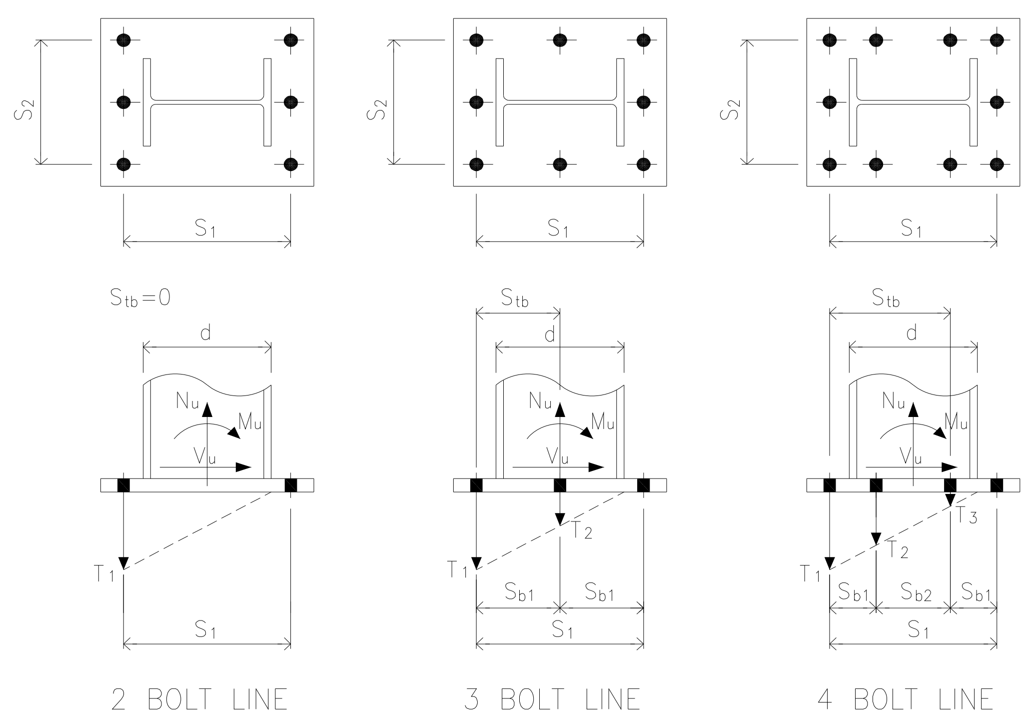
Deep anchor stud embedment h_{ef} is required for anchor reinforcement to develop resistance on both sides of the failure plane.

STUD ANCHOR DESIGN **Combined Tension, Shear and Moment**

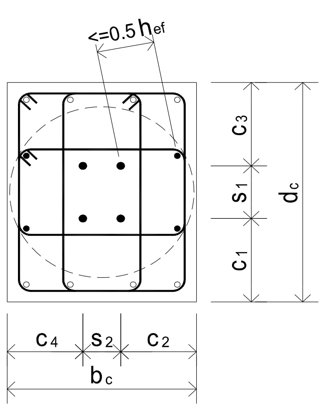
Anchor bolt design based on	**Code Abbreviation**
CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D	*A23.3-04 (R2010)*
ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary	*ACI318 M-08*
PIP STE05121 Anchor Bolt Design Guide-2006	*PIP STE05121*
	Code Reference
Assumptions	*A23.3-04 (R2010)*
1. Concrete is cracked	
2. Condition A - supplementary reinforcement is provided	D.5.4 (c)
3. Anchor reinft strength is used to replace concrete tension / shear breakout strength as per ACI318 M-08 Appendix D clause D.5.2.9 and D.6.2.9	*ACI318 M-08* D.5.2.9 & D.6.2.9
4. For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective	
5. Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft	*A23.3-04 (R2010)*
6. For anchor group subject to moment, the anchor tensile load is designed using <u>elastic analysis</u> and there is no redistribution of the forces between highly stressed and less stressed anchors	D.4.1
7. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output	

Anchor Stud Data

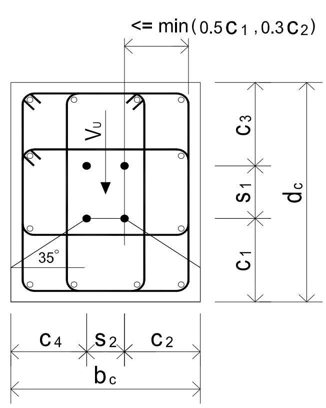
Factored moment	M_u =	47.4	[kNm]		=	35.0	[kip-ft]
Factored tension /compression	N_u =	-44.5	[kN]	in compression	=	-10.0	[kips]
Factored shear	$\mathbf{V_u}$ =	**111.2**	[kN]		=	25.0	[kips]



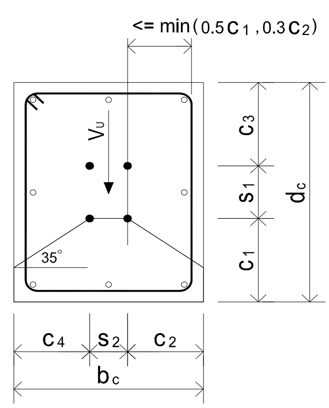
								Code Reference
No of bolt line for resisting moment	=	2 Bolt Line						
No of bolt along outermost bolt line	=	2						
				min required				
Outermost stud line spacing s_1	s_1 =	406	[mm]	102	**OK**			*PIP STE05121*
Outermost stud line spacing s_2	s_2 =	406	[mm]	102	**OK**			Page A -1 Table 1
Internal stud line spacing s_{b1}	s_{b1} =	267	[mm]	102	**OK**			
Internal stud line spacing s_{b2}	s_{b2} =	0	[mm]	102	**OK**			
Column depth	d =	323	[mm]					
Concrete strength	f'_c =	28	[MPa]			= 4.0	[ksi]	
Anchor bolt material	=	AWS D1.1 Grade B						
Anchor tensile strength	f_{uta} =	65	[ksi]			= 448	[MPa]	*A23.3-04 (R2010)*
		Stud is ductile steel element						D.2
Stud diameter	**d_a** =	**1**	[in]			= 25.4	[mm]	
Stud shank area	A_{se} =	0.79	[in^2]			= 507	[mm^2]	
Stud head bearing area	A_{brg} =	1.29	[in^2]			= 831	[mm^2]	
				min required				*PIP STE05121*
Anchor bolt embedment depth	h_{ef} =	1397	[mm]	305	**OK**			Page A -1 Table 1
Pedestal height	h =	1524	[mm]	1473	**OK**			
Pedestal width	b_c =	660	[mm]					
Pedestal depth	d_c =	660	[mm]					
Stud edge distance c_1	c_1 =	127	[mm]	115	**OK**			Page A -1 Table 1
Stud edge distance c_2	c_2 =	127	[mm]	115	**OK**			
Stud edge distance c_3	c_3 =	127	[mm]	115	**OK**			
Stud edge distance c_4	c_4 =	127	[mm]	115	**OK**			



Ver. Reinft For Tension

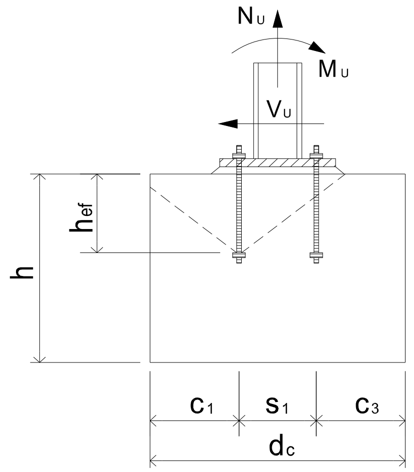


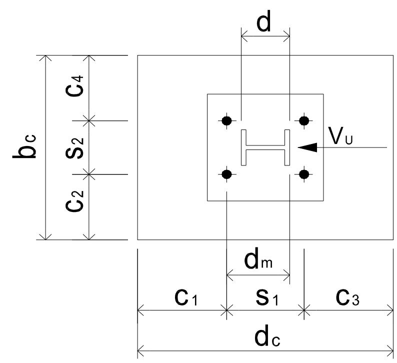
Hor. Ties For Shear - 4 Legs



Hor. Ties For Shear - 2 Legs

Code Reference





ACI318 M-08

To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within $0.5h_{ef}$ from the outmost anchor's centerline. In this design $0.5h_{ef}$ value is limited to 200mm. — RD.5.2.9

$0.5h_{ef}$ = **200** [mm]

No of ver. rebar that are effective for resisting anchor tension n_v = 2

Ver. bar size d_b = 25 single bar area A_s = 500 [mm^2]

To be considered effective for resisting anchor shear, hor. reinft shall be located within min($0.5c_1$, $0.3c_2$) from the outmost anchor's centerline — RD.6.2.9

min($0.5c_1$, $0.3c_2$) = **38** [mm]

No of tie leg that are effective to resist anchor shear n_{leg} = 2 ?

No of tie layer that are effective to resist anchor shear n_{lay} = 2 ?

Hor. bar size d_b = 15 single bar area A_s = 200 [mm^2]

For anchor reinft shear breakout strength calc: 100% hor. tie bars develop full yield strength ?

suggest

Rebar yield strength f_y = 414 [MPa] 400 = 60.0 [ksi]

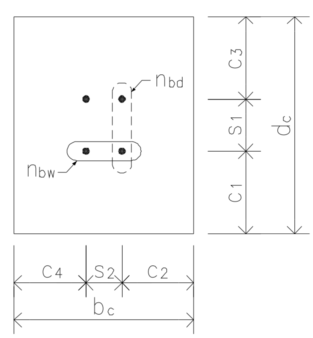
Total no of welded stud n = 4

No of stud carrying tension n_t = 2

No of stud carrying shear n_s = 2

For side-face blowout check use

No of stud along width edge n_{bw} = 2



Bolt No Input for Side-Face Blowout Check Use

A23.3-04 (R2010)

Seismic region where $I_E F_a S_a(0.2)$>=0.35 = No ? — D.4.3.5

Provide built-up grout pad ? = No ? — D.7.1.3

				Code Reference
Strength reduction factors				*A23.3-04 (R2010)*
Anchor reinforcement factor	ϕ_{as} = 0.75			D.7.2.9
Steel anchor resistance factor	ϕ_s = 0.85			8.4.3 (a)
Concrete resistance factor	ϕ_c = 0.65			8.4.2
Resistance modification factors				
Anchor rod - ductile steel	$R_{t,s}$ = 0.80	$R_{v,s}$ = 0.75		D.5.4(a)
Concrete - condition A	$R_{t,c}$ = 1.15	$R_{v,c}$ = 1.15		D.5.4(c)

CONCLUSION

Abchor Rod Embedment, Spacing and Edge Distance		**OK**	
Min Rquired Anchor Reinft. Development Length	ratio = 0.25	**OK**	12.2.1
Overall	ratio = **0.76**	**OK**	
Tension			
Stud Tensile Resistance	ratio = 0.36	**OK**	
Anchor Reinft Tensile Breakout Resistance	ratio = 0.36	**OK**	
Stud Pullout Resistance	ratio = 0.46	**OK**	
Side Blowout Resistance	ratio = 0.38	**OK**	
Shear			
Stud Shear Resistance	ratio = 0.38	**OK**	
Anchor Reinft Shear Breakout Resistance			
Strut Bearing Strength	ratio = 0.60	**OK**	
Tie Reinforcement	ratio = 0.45	**OK**	
Conc. Pryout Not Govern When h_{ef} >= $12d_a$		**OK**	
Stud on Conc Bearing	ratio = 0.43	**OK**	
Tension Shear Interaction			
Tension Shear Interaction	ratio = 0.76	**OK**	
Ductility			
Tension **Non-ductile**	**Shear** **Non-ductile**		
Seismic Design Requirement		**OK**	D.4.3.6

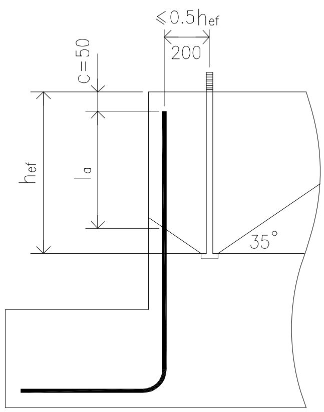
IeFaSa(0.2)<0.35, A23.3-04 D.4.3.3 ductility requirement is NOT required

CACULATION

Anchor Tensile Force

Single stud tensile force	T_1 = 55.2	[kN]	No of stud for T_1 n_{T1} = 2		
	T_2 = 0.0	[kN]	No of stud for T_2 n_{T2} = 0		
	T_3 = 0.0	[kN]	No of stud for T_3 n_{T3} = 0		
Sum of stud tensile force	$\mathbf{N_u} = \Sigma\, n_i\, T_i$		= **110.3**	[kN]	
Stud Tensile Resistance	$N_{sr} = A_{se}\, \phi_s\, f_{uta}\, R_{t,s}$		= 154.4	[kN]	D.6.1.2 (D-3)
	ratio = 0.36		> T_1	**OK**	

					Code Reference
Anchor Reinft Tensile Breakout Resistance					*A23.3-04 (R2010)*
Min tension development length	l_d =		= 887	[mm]	12.2.3
for ver. 25M bar					
Actual development lenngth	l_a =	h_{ef} - c (50mm) - 200mm x tan35	= 1207	[mm]	
			> 300	**OK**	12.2.1



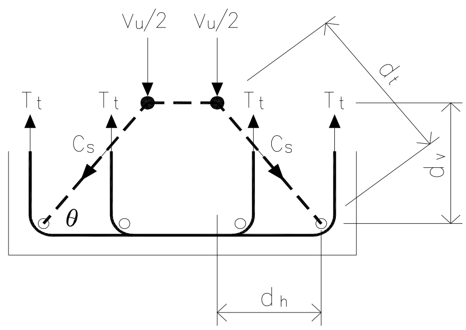
	N_{rbr} =	ϕ_{as} x f_y x n_v x A_s x (l_a / l_d , if l_a < l_d)	= 310.5	[kN]	12.2.5
Seismic design strength reduction	=	x 1.0 not applicable	= 310.5	[kN]	D.4.3.5
	ratio =	0.36	> N_u	**OK**	
Stud Pullout Resistance					
Single bolt pullout resistance	N_{pr} =	8 A_{brg} ϕ_c f_c' $R_{t,c}$	= 119.3	[kN]	D.6.3.4 (D-16)
	N_{cpr} =	$\Psi_{c,p}$ N_{pr}	= 119.3	[kN]	D.6.3.1 (D-15)
Seismic design strength reduction	=	x 1.0 not applicable	= 119.3	[kN]	D.4.3.5
	ratio =	0.46	> T_1	**OK**	
	$\Psi_{c,p}$ =	1 for cracked conc			D.6.3.6
	$R_{t,c}$ =	1.00 pullout strength is always Condition B			D.5.4(c)
Side Blowout Resistance					
<u>Failure Along Pedestal Width Edge</u>					*ACI318 M-08*
Tensile load carried by anchors close to edge which may cause side-face blowout					
along pedestal width edge	N_{buw} =	n_{T1} T_1	= 110.3	[kN]	RD.5.4.2
	c =	min (c_1, c_3)	= 127	[mm]	
Check if side blowout applicable	h_{ef} =	1397 [mm]			*A23.3-04 (R2010)*
	>	2.5c side bowout is applicable			D.6.4.1
Check if edge anchors work as a	s_{22} =	406 [mm] s = s_2	= 406	[mm]	
a group or work individually	<	6c edge anchors work as a group			D.6.4.2
Single anchor SB resistance	$N_{sbr,w}$ =	$13.3c\sqrt{A_{brg}}\ \phi_c\ \sqrt{f'_c}\ R_{t,c}$	= 191.3	[kN]	D.6.4.1 (D-18)
Multiple anchors SB resistance	$N_{sbgr,w}$ =				
work as a group - applicable	=	(1+s/ 6c) x $N_{sbr,w}$	= 293.2	[kN]	D.6.4.2 (D-19)
work individually - not applicable	=	n_{bw} x $N_{sbr,w}$ x [1+(c_2 or c_4) / c] / 4	= 0.0	[kN]	D.6.4.1
Seismic design strength reduction	=	x 1.0 not applicable	= 293.2	[kN]	D.4.3.5
	ratio =	0.38	> N_{buw}	**OK**	
Group side blowout resistance	N_{sbgr} =	$\frac{N_{sbgr,w}}{n_{bw}} n_t$	= 293.2	[kN]	
Govern Tensile Resistance	$\mathbf{N_r}$ =	min (n_t N_{sr}, N_{rbr}, n_t N_{cpr}, N_{sbgr})	= **238.6**	[kN]	

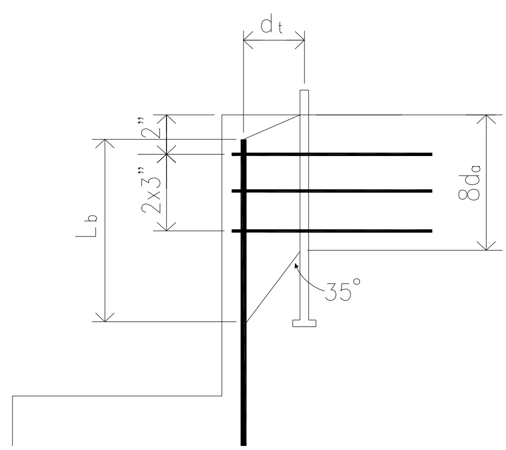
				Code Reference
				A23.3-04 (R2010)
Stud Shear Resistance	$V_{sr} = n_s A_{se} \phi_s f_{uta} R_{v,s}$	= 289.5	[kN]	D.7.1.2 (a) (D-20)
Reduction due to built-up grout pads	= x 1.0 , not applicable	= 289.5	[kN]	D.7.1.3
	ratio = 0.38	> V_u	**OK**	

Anchor Reinft Shear Breakout Resistance *ACI318 M-08*

Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft

STM strength reduction factor ϕ_{st} = 0.75 9.3.2.6





Strut-and-Tie model geometry	d_v = 57 [mm]	d_h = 57	[mm]	
	θ = 45	d_t = 81	[mm]	
Strut compression force	C_s = 0.5 V_u / sinθ	= 78.6	[kN]	
				ACI318 M-08
Strut Bearing Strength				
Strut compressive strength	f_{ce} = 0.85 f'_c	= 23.5	[MPa]	A.3.2 (A-3)
* Bearing of anchor bolt				
Anchor bearing length	l_e = min($8d_a$, h_{ef})	= 203	[mm]	D.6.2.2
Anchor bearing area	A_{brg} = l_e x d_a	= 5161	[mm^2]	
Anchor bearing resistance	C_r = n_s x ϕ_{st} x f_{ce} x A_{brg}	= 181.6	[kN]	
		> V_u	**OK**	
* Bearing of ver reinft bar				
Ver bar bearing area	A_{brg} = (l_e +1.5 x d_t - d_a/2 -d_b/2) x d_b	= 7473	[mm^2]	
Ver bar bearing resistance	C_r = ϕ_{st} x f_{ce} x A_{brg}	= 131.5	[kN]	
	ratio = 0.60	> C_s	**OK**	

Code Reference

Tie Reinforcement

* For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
* For enclosed tie, at hook location the tie cannot develop full yield strength f_y . Use the pullout resistance in tension of a single J-bolt as per A23.3-04 Annex D Eq. (D-17) as the max force can be developed at hook T_h
* Assume 100% of hor. tie bars can develop full yield strength.

Total number of hor tie bar	$n = n_{leg}$ (leg) x n_{lay} (layer)	= 4			*A23.3-04 (R2010)*
Pull out resistance at hook	$T_h = 0.9\ \phi_c\ f_c'\ e_h\ d_b\ R_{t,c}$	= 16.3	[kN]		D.6.3.5 (D-17)
	$e_h = 4.5\ d_b$	= 68	[mm]		
Single tie bar tension resistance	$T_r = \phi_{as}$ x f_y x A_s	= 62.1	[kN]		
Total tie bar tension resistance	$V_{rbr} = 1.0$ x n x Tr	= 248.4	[kN]		
Seismic design strength reduction	= x 1.0 not applicable	= 248.4	[kN]		D.4.3.5
	ratio = 0.45	> V_u		**OK**	

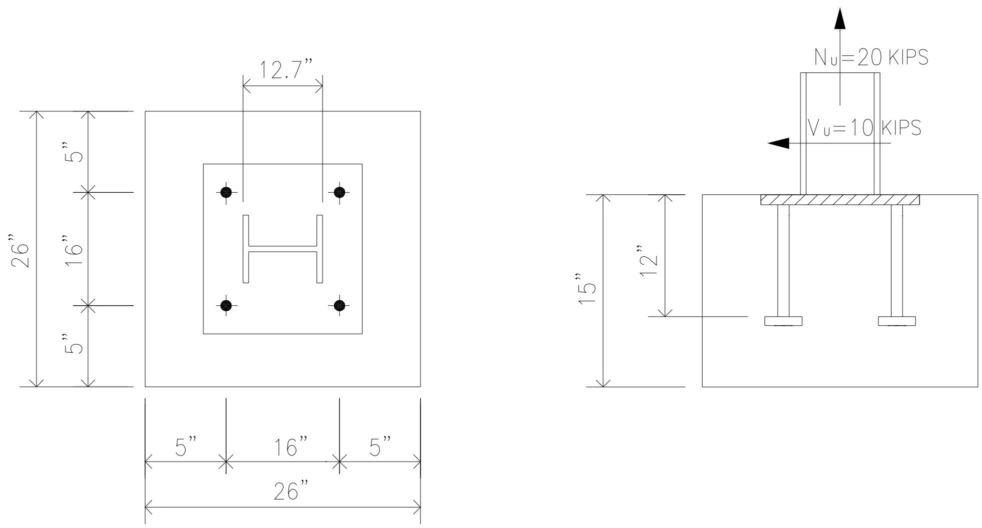
Conc. Pryout Shear Resistance

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} > = $12d_a$, the pryout failure will not govern

	$12d_a = 305$ [mm]	h_{ef} = 1397	[mm]		
		> $12d_a$		**OK**	*CSA S16-09*
Stud on Conc Bearing	$B_r = n_s$ x 1.4 x ϕ_c x min($8d_a$, h_{ef}) x d_a x f_c'	= 259.3	[kN]		25.3.3.2
	ratio = 0.43	< V_u		**OK**	
Govern Shear Resistance	$\mathbf{V_r}$ = min (V_{sr} , V_{rbr} , B_r)	= **248.4**	[kN]		

Tension Shear Interaction					*A23.3-04 (R2010)*
Check if N_u >0.2 N_r and V_u >0.2 V_r	Yes				D.8.2 & D.8.3
	$N_u/N_r + V_u/V_r$	= 0.91			D.8.4 (D-35)
	ratio = 0.76	< 1.2		**OK**	
Ductility Tension	N_{sr} = 154.4 [kN]				
	> min (N_{rbr} , N_{cpr}, N_{sbgr})	= 119.3	[kN]		
	Non-ductile				
Ductility Shear	V_{sr} = 289.5 [kN]				
	> min (V_{rbr}, B_r)	= 248.4	[kN]		
	Non-ductile				

Example 31: Welded Stud + No Anchor Reinft + Tension & Shear + ACI 318-08 Code



N_u= 20 kips (Tension) V_u = 10 kips

Concrete f_c'= 4.5 ksi

Anchor stud AWS D1.1 Grade B 1.0" dia h_{ef} = 12" h_a =15"

Seismic design category < C

Supplementary reinforcement Tension → Condition A

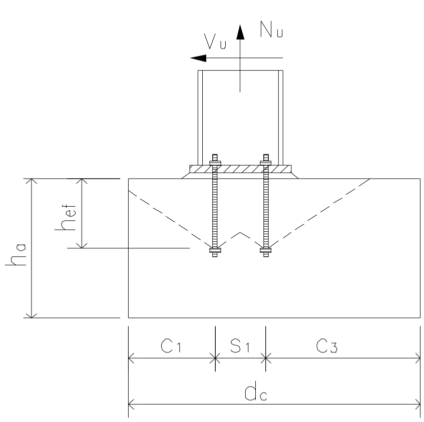
Shear → Condition A $\Psi_{c,V}$ = 1.2

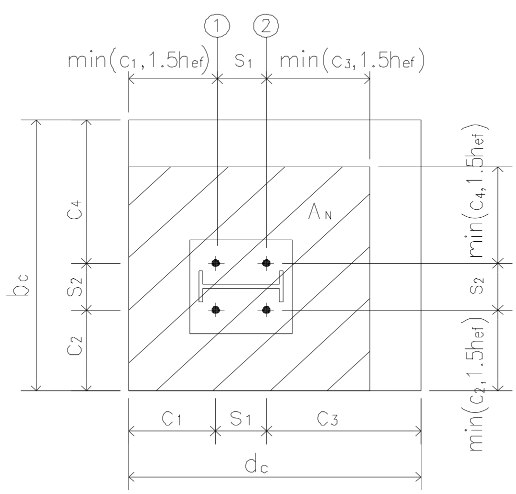
No built-up grout pad for embedded plate.

Note: The stud length used in this example may not be commercially available and it's for illustration purpose only.

STUD ANCHOR DESIGN **Combined Tension and Shear**

Anchor bolt design based on						**Code Abbreviation**
ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D						*ACI 318-08*
PIP STE05121 Anchor Bolt Design Guide-2006						*PIP STE05121*
Input Data		set N_u = 0 if it's compression				**Code Reference**
Factored tension	N_u =	**20.0**	[kips]	= 89.0	[kN]	
Factored shear	V_u =	**10.0**	[kips]	= 44.5	[kN]	
Concrete strength	f'_c =	4.5	[ksi]	= 31.0	[MPa]	
Stud material	=	AWS D1.1 Grade B				
Stud tensile strength	f_{uta} =	65	[ksi]	= 448	[MPa]	*ACI 318-08*
		Stud is ductile steel element				D.1
Stud diameter	d_a =	**1**	[in]	= 25.4	[mm]	
Stud shank area	A_{se} =	0.79	[in^2]	= 507	[mm^2]	
Stud head bearing area	A_{brg} =	1.29	[in^2]	= 831	[mm^2]	
				min required		*PIP STE05121*
Stud embedment depth	h_{ef} =	12.0	[in]	12.0	**OK**	Page A -1 Table 1
Concrete thickness	h_a =	15.0	[in]	15.0	**OK**	
Stud edge distance c_1	c_1 =	5.0	[in]	4.5	**OK**	Page A -1 Table 1
Stud edge distance c_2	c_2 =	5.0	[in]	4.5	**OK**	
Stud edge distance c_3	c_3 =	5.0	[in]	4.5	**OK**	
Stud edge distance c_4	c_4 =	5.0	[in]	4.5	**OK**	*ACI 318-08*
$c_i > 1.5h_{ef}$ for at least two edges to avoid reducing of h_{ef} when $N_u > 0$					**No**	D.5.2.3
Adjusted h_{ef} for design	h_{ef} =	**5.33**	[in]	12.0	**Warn**	D.5.2.3
Outermost stud line spacing s_1	s_1 =	16.0	[in]	4.0	**OK**	*PIP STE05121*
Outermost stud line spacing s_2	s_2 =	16.0	[in]	4.0	**OK**	Page A -1 Table 1



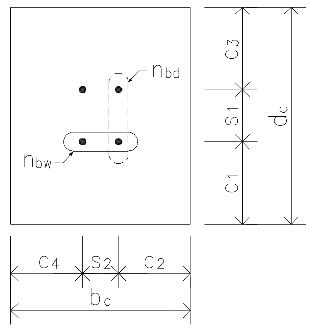


Code Reference

Number of stud at bolt line 1	n_1	= 2		
Number of stud at bolt line 2	n_2	= 2		
Total no of welded stud	n	= 4		
Number of stud carrying tension	n_t	= 4		
Number of stud carrying shear	n_s	= 2		

For side-face blowout check use

No of stud along width edge	n_{bw}	= 2
No of stud along depth edge	n_{bd}	= 2



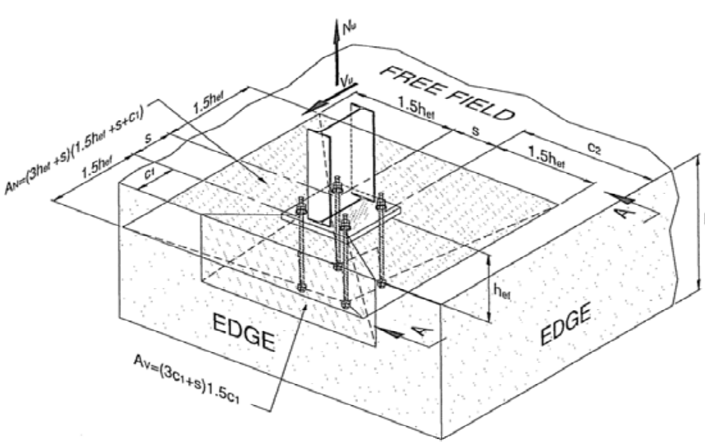
Bolt No Input for Side-Face Blowout Check Use

ACI 318-08

				Code Reference
Seismic design category >= C		= No ?		D.3.3.3
Supplementary reinforcement				
For tension		= Yes	Condition A	D.4.4 (c)
For shear	$\Psi_{c,V}$	= 1.2	Condition A ?	D.6.2.7
Provide built-up grout pad ?		= No ?		D.6.1.3
Strength reduction factors				
Anchor reinforcement	ϕ_s	= 0.75		D.5.2.9 & D.6.2.9
Anchor rod - ductile steel	$\phi_{t,s}$	= 0.75	$\phi_{v,s}$ = 0.65	D.4.4 (a)
Concrete	$\phi_{t,c}$	= 0.75 Cdn-A	$\phi_{v,c}$ = 0.75 Cdn-A	D.4.4 (c)

Assumptions

1. Concrete is cracked
2. Condition A - supplementary reinforcement provided — D.4.4 (c)
3. Load combinations shall be per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2 — D.4.4
4. Tensile load acts through center of bolt group $\Psi_{ec,N}$ =1.0 — D.5.2.4
5. Shear load acts through center of bolt group $\Psi_{ec,V}$ =1.0 — D.6.2.5



CONCLUSION

Abchor Rod Embedment, Spacing and Edge Distance		**Warn**	
Overall	ratio = **1.00**	**OK**	
Tension			
Stud Tensile Resistance	ratio = 0.13	**OK**	
Conc. Tensile Breakout Resistance	ratio = 0.57	**OK**	
Stud Pullout Resistance	ratio = 0.15	**OK**	
Side Blowout Resistance	ratio = 0.00	**OK**	
Shear			
Stud Shear Resistance	ratio = 0.15	**OK**	
Conc. Shear Breakout Resistance	ratio = 0.62	**OK**	
Conc. Pryout Shear Resistance	ratio = 0.15	**OK**	
Tension Shear Interaction			
Tension Shear Interaction	ratio = 1.00	**OK**	
Ductility			
Tension **Non-ductile**	**Shear** **Non-ductile**		
Seismic Design Requirement		**OK**	D.3.3.4

SDC< C, ACI318-08 D.3.3 ductility requirement is NOT required

CALCULATION | **Code Reference** *ACI 318-08*

Stud Tensile Resistance $\phi_{t,s} N_{sa} = \phi_{t,s}\, n_t\, A_{se}\, f_{uta}$ = 153.2 [kips] D.5.1.2 (D-3)

ratio = 0.13 > N_u **OK**

Conc. Tensile Breakout Resistance

$N_b = 24\ \lambda \sqrt{f_c'}\ h_{ef}^{1.5}$ if $h_{ef} < 11"$ or $h_{ef} > 25"$ = 19.8 [kips] D.5.2.2 (D-7)

$16\ \lambda \sqrt{f_c'}\ h_{ef}^{5/3}$ if $11" \le h_{ef} \le 25"$ D.5.2.2 (D-8)

Projected conc failure area	$1.5h_{ef}$ =	= 8.00	[in]	
	A_{Nc} = $[s_1+\min(c_1,1.5h_{ef})+\min(c_3,1.5h_{ef})]\times$ $[s_2+\min(c_2,1.5h_{ef})+\min(c_4,1.5h_{ef})]$	= 676.0	[in^2]	
	$A_{Nco} = 9\ h_{ef}^2$	= 256.0	[in^2]	D.5.2.1 (D-6)
	$A_{Nc} = \min(A_{Nc},\ n_t A_{Nco})$	= 676.0	[in^2]	D.5.2.1
Min edge distance	$c_{min} = \min(c_1, c_2, c_3, c_4)$	= 5.0	[in]	
Eccentricity effects	$\Psi_{ec,N}$ = 1.0 for no eccentric load			D.5.2.4
Edge effects	$\Psi_{ed,N} = \min[(0.7+0.3c_{min}/1.5h_{ef}),\ 1.0]$	= 0.89		D.5.2.5
Concrete cracking	$\Psi_{c,N}$ = 1.0 for cracked concrete			D.5.2.6
Concrete splitting	$\Psi_{cp,N}$ = 1.0 for cast-in anchor			D.5.2.7

					Code Reference
					ACI 318-08
Concrete breakout resistance	$\phi_{t,c} N_{cbg}$	$= \phi_{t,c} \frac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$	= 34.9	[kips]	D.5.2.1 (D-5)
Seismic design strength reduction		= x 1.0 not applicable	= 34.9	[kips]	D.3.3.3
	ratio	= 0.57	> N_u	**OK**	

Stud Pullout Resistance

Single bolt pullout resistance	N_p	$= 8 A_{brg} f_c'$	= 46.4	[kips]	D.5.3.4 (D-15)
	$\phi_{t,c} N_{pn}$	$= \phi_{t,c} n_t \Psi_{c,p} N_p$	= 129.9	[kips]	D.5.3.1 (D-14)
Seismic design strength reduction		= x 1.0 not applicable	= 129.9	[kips]	D.3.3.3
	ratio	= 0.15	> N_u	**OK**	
	$\Psi_{c,p}$	= 1 for cracked conc			D.5.3.6
	$\phi_{t,c}$	= 0.70 pullout strength is always Condition B			D.4.4(c)

Side Blowout Resistance

Failure Along Pedestal Width Edge

Tensile load carried by anchors close to edge which may cause side-face blowout

along pedestal width edge	N_{buw}	$= N_u \times n_{bw} / n_t$	= 10.0	[kips]	RD.5.4.2
	c	$= \min(c_1, c_3)$	= 5.0	[in]	
Check if side blowout applicable	h_{ef}	= 12.0 [in]			
		< 2.5c side bowout is NOT applicable			D.5.4.1
Check if edge anchors work as a	s_{22}	= 0.0 [in] $s = s_2$	= 0.0	[in]	
a group or work individually		< 6c side bowout is NOT applicable			D.5.4.2
Single anchor SB resistance	$\phi_{t,c} N_{sb}$	$= \phi_{t,c} \left(160\, c \sqrt{A_{brg}}\right) \lambda \sqrt{f'_c}$	= 0.0	[kips]	D.5.4.1 (D-17)
Multiple anchors SB resistance	$\phi_{t,c} N_{sbg,w}$	=			
work as a group - not applicable		$= (1+s/6c) \times \phi_{t,c} N_{sb}$	= 0.0	[kips]	D.5.4.2 (D-18)
work individually - not applicable		$= n_{bw} \times \phi_{t,c} N_{sb} \times [1+(c_2 \text{ or } c_4)/c] / 4$	= 0.0	[kips]	D.5.4.1
Seismic design strength reduction		= x 1.0 not applicable	= 0.0	[kips]	D.3.3.3
	ratio	= 0.00	< N_{buw}	**OK**	

Failure Along Pedestal Depth Edge

Tensile load carried by anchors close to edge which may cause side-face blowout

along pedestal depth edge	N_{bud}	$= N_u \times n_{bd} / n_t$	= 10.0	[kips]	RD.5.4.2
	c	$= \min(c_2, c_4)$	= 5.0	[in]	
Check if side blowout applicable	h_{ef}	= 12.0 [in]			
		< 2.5c side bowout is NOT applicable			D.5.4.1
Check if edge anchors work as a	s_{11}	= 0.0 [in] $s = s_1$	= 0.0	[in]	
a group or work individually		< 6c side bowout is NOT applicable			D.5.4.2
Single anchor SB resistance	$\phi_{t,c} N_{sb}$	$= \phi_{t,c} \left(160\, c \sqrt{A_{brg}}\right) \lambda \sqrt{f'_c}$	= 0.0	[kips]	D.5.4.1 (D-17)
Multiple anchors SB resistance	$\phi_{t,c} N_{sbg,d}$	=			
work as a group - not applicable		$= (1+s/6c) \times \phi_{t,c} N_{sb}$	= 0.0	[kips]	D.5.4.2 (D-18)
work individually - not applicable		$= n_{bd} \times \phi_{t,c} N_{sb} \times [1+(c_1 \text{ or } c_3)/c] / 4$	= 0.0	[kips]	D.5.4.1
Seismic design strength reduction		= x 1.0 not applicable	= 0.0	[kips]	D.3.3.3
	ratio	= 0.00	< N_{bud}	**OK**	

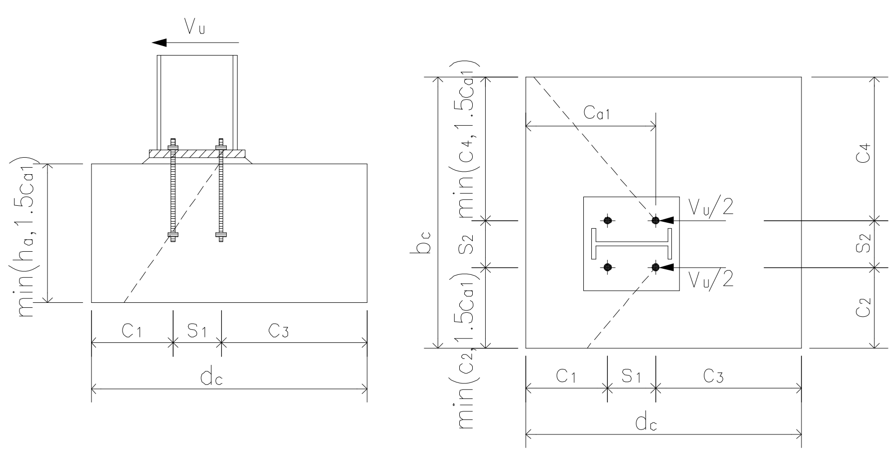
				Code Reference
Group side blowout resistance	$\phi_{t,c} N_{sbg} = \phi_{t,c} \min\left(\frac{N_{sbg,w}}{n_{bw}} n_t , \frac{N_{sbg,d}}{n_{bd}} n_t\right)$	= 0.0	[kips]	*ACI 318-08*
Govern Tensile Resistance	$\mathbf{N_r} = \min [\, \phi_{t,s} N_{sa}, \phi_{t,c} (N_{cbg}, N_{pn}, N_{sbg})\,]$	= **34.9**	[kips]	
Stud Shear Resistance	$\phi_{v,s} V_{sa} = \phi_{v,s} n_s A_{se} f_{uta}$	= 66.4	[kips]	D.6.1.2 (a) (D-19)
Reduction due to built-up grout pads	= x 1.0 , not applicable	= 66.4	[kips]	D.6.1.3
	ratio = 0.15	> V_u	**OK**	

Conc. Shear Breakout Resistance

Only Case 2 needs to be considered when anchors are rigidly connected to the attachment — Fig. RD.6.2.1(b) notes

This applies to welded stud case so only Mode 2 is considered for shear checking — in Case 2

Mode 2 Failure cone at back anchors



ACI 318-08

Bolt edge distance	$c_{a1} = c_1 + s_1$	= 21.0	[in]	
Limiting c_{a1} when anchors are influenced by 3 or more edges		= Yes		D.6.2.4
Bolt edge distance - adjusted	$\mathbf{c_{a1}}$ = ca1 needs to be adjusted	= **10.0**	[in]	D.6.2.4
	c_2 =	5.0	[in]	
	$1.5c_{a1}$ =	15.0	[in]	
	$A_{Vc} = [\min(c_2,1.5c_{a1}) + s_2 + \min(c_4,1.5c_{a1})] \times \min(1.5c_{a1}, h_a)$	= 390.0	[in^2]	D.6.2.1
	$A_{Vco} = 4.5c_{a1}^2$	= 450.0	[in^2]	D.6.2.1 (D-23)
	$A_{Vc} = \min(A_{Vc}, n_2 A_{Vco})$	= 390.0	[in^2]	D.6.2.1
	$l_e = \min(8d_a , h_{ef})$	= 8.0	[in]	D.6.2.2
	$V_b = \left[8\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right] \lambda\sqrt{f'_c}\ c_{a1}^{1.5}$	= 25.7	[kips]	D.6.2.3 (D-25)

					Code Reference
					ACI 318-08
Eccentricity effects	$\Psi_{ec,v}$ = 1.0 shear acts through center of group				D.6.2.5
Edge effects	$\Psi_{ed,v}$ = min[$(0.7+0.3c_2/1.5c_{a1})$, 1.0]	= 0.80			D.6.2.6
Concrete cracking	$\Psi_{c,v}$ =	= 1.20			D.6.2.7
Member thickness	$\Psi_{h,v}$ = max[(sqrt($1.5c_{a1}$ / h_a) , 1.0]	= 1.00			D.6.2.8
Conc shear breakout resistance	$V_{cbg2} = \phi_{v,c} \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V}\ \Psi_{ed,V}\ \Psi_{c,V}\ \Psi_{h,V}\ V_b$	= 16.1	[kips]		D.6.2.1 (D-22)
Seismic design strength reduction	= x 1.0 not applicable	= 16.1	[kips]		D.3.3.3
	ratio = 0.62	> V_u		OK	

Conc. Pryout Shear Resistance

	k_{cp} = 2.0				D.6.3
Factored shear pryout resistance	$\phi_{v,c} V_{cpg} = \phi_{v,c}\ k_{cp}\ N_{cbg}$	= 65.1	[kips]		D.6.3 (D-31)
	$\phi_{v,c}$ = 0.70 pryout strength is always Condition B				D.4.4(c)
Seismic design strength reduction	= x 1.0 not applicable	= 65.1	[kips]		D.3.3.3
	ratio = 0.15	> V_u		OK	

Govern Shear Resistance	$\mathbf{V_r}$ = min [$\phi_{v,s}V_{sa}$, $\phi_{v,c}$ (V_{cbg}, V_{cpg})]	= **16.1**	[kips]	

Tension Shear Interaction

Check if $N_u > 0.2\phi\ N_n$ and $V_u > 0.2\phi\ V_n$	Yes			D.7.1 & D.7.2
	$N_u/N_r + V_u/V_r$	= 1.20		D.7.3 (D-32)
	ratio = 1.00	< 1.2	OK	

Ductility Tension

$\phi_{t,s} N_{sa}$ = 153.2 [kips]

> $\phi_{t,c}$ min (N_{cbg}, N_{pn}, N_{sbg}) = 34.9 [kips]

Non-ductile

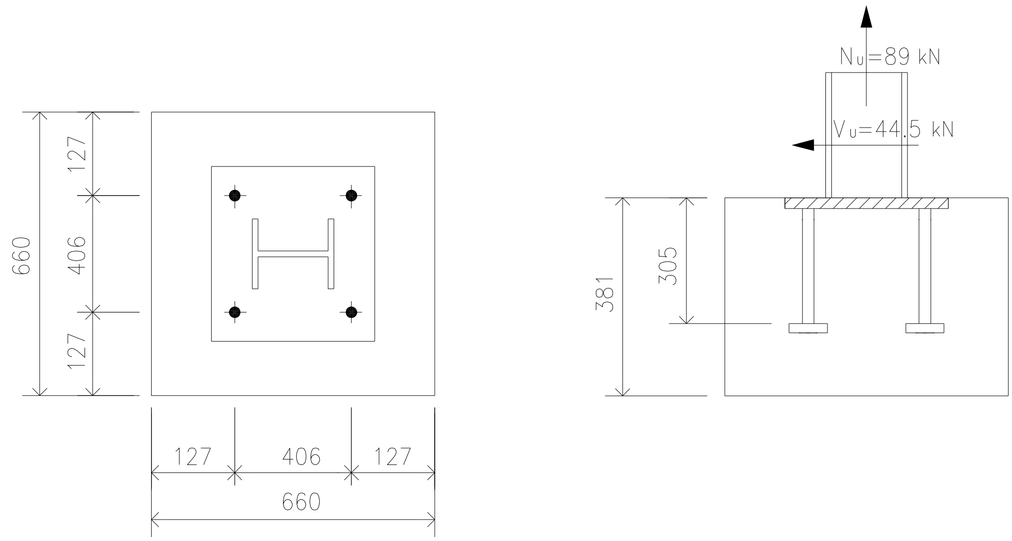
Ductility Shear

$\phi_{v,s} V_{sa}$ = 66.4 [kips]

> $\phi_{v,c}$ min (V_{cbg}, V_{cpg}) = 16.1 [kips]

Non-ductile

Example 32: Welded Stud + No Anchor Reinft + Tension & Shear + CSA A23.3-04 Code



N_u= 89 kN (Tension) V_u = 44.5 kN

Concrete f_c'= 31 MPa

Anchor stud AWS D1.1 Grade B 1.0" dia h_{ef} = 305mm h_a =381mm

Seismic design $I_E F_a S_a(0.2) < 0.35$

Supplementary reinforcement Tension → Condition A

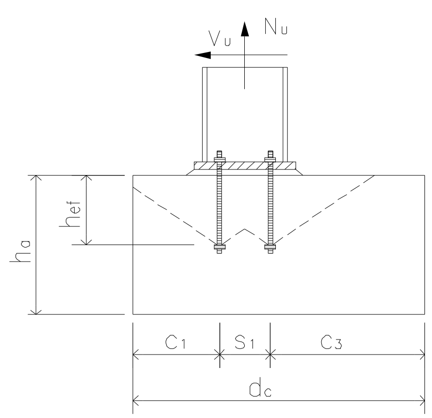
Shear → Condition A $\Psi_{c,V}$ = 1.2

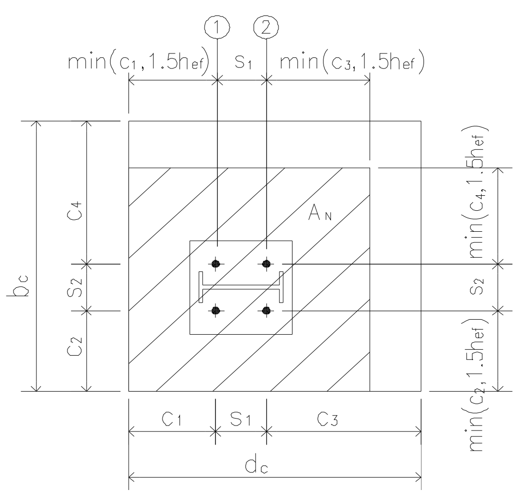
No built-up grout pad for embedded plate.

Note: The stud length used in this example may not be commercially available and it's for illustration purpose only.

STUD ANCHOR DESIGN **Combined Tension and Shear**

Anchor bolt design based on								**Code Abbreviation**
CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D								*A23.3-04 (R2010)*
ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary								*ACI318 M-08*
PIP STE05121 Anchor Bolt Design Guide-2006								*PIP STE05121*
Input Data		set N_u = 0 if it's compression						**Code Reference**
Factored tension	N_u =	**89.0**	[kN]			= 20.0	[kips]	
Factored shear	V_u =	**44.5**	[kN]			= 10.0	[kips]	
Concrete strength	f'_c =	31	[MPa]			= 4.5	[ksi]	
Anchor bolt material	=	AWS D1.1 Grade B						
Anchor tensile strength	f_{uta} =	65	[ksi]			= 448	[MPa]	*A23.3-04 (R2010)*
		Stud is ductile steel element						D.2
Stud diameter	d_a =	**1**	[in]			= 25.4	[mm]	
Stud shank area	A_{se} =	0.79	[in^2]			= 507	[mm^2]	
Stud head bearing area	A_{brg} =	1.29	[in^2]			= 831	[mm^2]	
				min required				*PIP STE05121*
Anchor bolt embedment depth	h_{ef} =	305	[mm]	305	**OK**			Page A -1 Table 1
Concrete thickness	h_a =	381	[mm]	381	**OK**			
Stud edge distance c_1	c_1 =	127	[mm]	115	**OK**			Page A -1 Table 1
Stud edge distance c_2	c_2 =	127	[mm]	115	**OK**			
Stud edge distance c_3	c_3 =	127	[mm]	115	**OK**			
Stud edge distance c_4	c_4 =	127	[mm]	115	**OK**			*A23.3-04 (R2010)*
$c_i > 1.5h_{ef}$ for at least two edges to avoid reducing of h_{ef} when $N_u > 0$					**No**			D.6.2.3
Adjusted h_{ef} for design	h_{ef} =	**135**	[mm]	305	**Warn**			D.6.2.3
Outermost stud line spacing s_1	s_1 =	406	[mm]	102	**OK**			*PIP STE05121*
Outermost stud line spacing s_2	s_2 =	406	[mm]	102	**OK**			Page A -1 Table 1

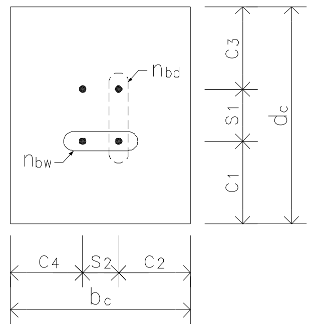




No of stud at bolt line 1	n_1	=	2
No of stud at bolt line 2	n_2	=	2
Total no of welded stud	n	=	4
No of stud carrying tension	n_t	=	4
No of stud carrying shear	n_s	=	2

For side-face blowout check use

No of stud along width edge	n_{bw}	=	2
No of stud along depth edge	n_{bd}	=	2



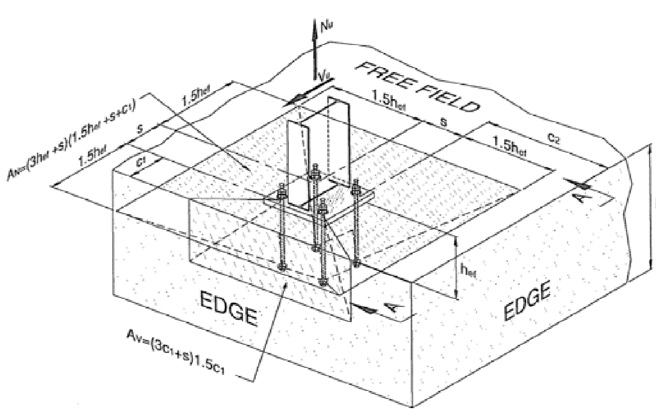
Bolt No Input for Side-Face Blowout Check Use

						Code Reference
						A23.3-04 (R2010)
Seismic region where $I_EF_aS_a(0.2)$>=0.35		=	No	?		D.4.3.5
Supplementary reinforcement						
For tension		=	Yes	Condition A		D.5.4 (c)
For shear	$\Psi_{c,V}$	=	1.2	Condition A	?	D.7.2.7
Provide built-up grout pad ?		=	No	?		D.7.1.3

Strength reduction factors									
Anchor reinforcement factor	ϕ_{as}	=	0.75						D.7.2.9
Steel anchor resistance factor	ϕ_s	=	0.85						8.4.3 (a)
Concrete resistance factor	ϕ_c	=	0.65						8.4.2
Resistance modification factors									
Anchor rod - ductile steel	$R_{t,s}$	=	0.80		$R_{v,s}$	=	0.75		D.5.4(a)
Concrete	$R_{t,c}$	=	1.15	Cdn-A	$R_{v,c}$	=	1.15	Cdn-A	D.5.4(c)

Assumptions

1. Concrete is cracked
2. Condition A for tension - supplementary reinforcement provided — D.5.4 (c)
3. Tensile load acts through center of bolt group $\Psi_{ec,N}$ =1.0 — D.6.2.4
4. Shear load acts through center of bolt group $\Psi_{ec,V}$ =1.0 — D.7.2.5



CONCLUSION

Abchor Rod Embedment, Spacing and Edge Distance			**Warn**
Overall		ratio = **1.01**	**NG**
Tension			
Stud Tensile Resistance		ratio = 0.14	**OK**
Conc. Tensile Breakout Resistance		ratio = 0.58	**OK**
Stud Pullout Resistance		ratio = 0.17	**OK**
Side Blowout Resistance		ratio = 0.00	**OK**
Shear			
Stud Shear Resistance		ratio = 0.15	**OK**
Conc. Shear Breakout Resistance		ratio = 0.63	**OK**
Conc. Pryout Shear Resistance		ratio = 0.17	**OK**
Stud on Conc Bearing		ratio = 0.15	**OK**
Tension Shear Interaction			
Tension Shear Interaction		ratio = 1.01	**NG**
Ductility			
	Tension **Non-ductile**	**Shear**	**Non-ductile**
Seismic Design Requirement			**OK** D.4.3.6

IeFaSa(0.2)<0.35, A23.3-04 D.4.3.3 ductility requirement is NOT required

CALCULATION **Code Reference**

A23.3-04 (R2010)

Stud Tensile Resistance $N_{sr} = n_t A_{se} \phi_s f_{uta} R_{t,s}$ = 617.7 [kN] D.6.1.2 (D-3)

ratio = 0.14 > N_u **OK**

Conc. Tensile Breakout Resistance

$N_{br} = 10\ \phi_c \sqrt{f'_c}\ h_{ef}^{1.5} R_{t,c}$ if $h_{ef} \le 275$ or $h_{ef} \ge 625$ D.6.2.2 (D-7)

$3.9\ \phi_c \sqrt{f'_c}\ h_{ef}^{5/3} R_{t,c}$ if $275 < h_{ef} < 625$ D.6.2.2 (D-8)

= 65.5 [kN]

Projected conc failure area	$1.5h_{ef}$ =	= 203	[mm]	
	A_{Nc} = $[s_1+\min(c_1,1.5h_{ef})+\min(c_3,1.5h_{ef})]\times$ $[s_2+\min(c_2,1.5h_{ef})+\min(c_4,1.5h_{ef})]$	= 4.4E+05	[mm^2]	
	$A_{Nco} = 9\ h_{ef}^2$	= 1.6E+05	[mm^2]	D.6.2.1 (D-6)
	$A_{Nc} = \min(A_{Nc},\ n_t A_{Nco})$	= 4.4E+05	[mm^2]	D.6.2.1
Min edge distance	$c_{min} = \min(c_1, c_2, c_3, c_4)$	= 127	[mm]	
Eccentricity effects	$\Psi_{ec,N}$ = 1.0 for no eccentric load			D.6.2.4
Edge effects	$\Psi_{ed,N} = \min[(0.7+0.3c_{min}/1.5h_{ef}), 1.0]$	= 0.89		D.6.2.5
Concrete cracking	$\Psi_{c,N}$ = 1.0 for cracked concrete			D.6.2.6
Concrete splitting	$\Psi_{cp,N}$ = 1.0 for cast-in anchor			D.6.2.7

					Code Reference
					A23.3-04 (R2010)
Concrete breakout resistance	N_{cbgr}	$= \frac{A_{Nc}}{A_{Nco}} \Psi_{ec,N}\ \Psi_{ed,N}\ \Psi_{c,N}\ \Psi_{cp,N}\ N_{br}$	= 153.7	[kN]	D.6.2.1 (D-5)
Seismic design strength reduction		= x 1.0 not applicable	= 153.7	[kN]	D.4.3.5
	ratio	= 0.58	> N_u	**OK**	
Stud Pullout Resistance					
Single bolt pullout resistance	N_{pr}	$= 8\ A_{brg}\ \phi_c\ f_c'\ R_{t,c}$	= 134.0	[kN]	D.6.3.4 (D-16)
	N_{cpr}	$= n_t\ \Psi_{c,p}\ N_{pr}$	= 536.0	[kN]	D.6.3.1 (D-15)
Seismic design strength reduction		= x 1.0 not applicable	= 536.0	[kN]	D.4.3.5
	ratio	= 0.17	> N_u	**OK**	
	$\Psi_{c,p}$	= 1 for cracked conc			D.6.3.6
	$R_{t,c}$	= 1.00 pullout strength is always Condition B			D.5.4(c)
Side Blowout Resistance					
Failure Along Pedestal Width Edge					
Tensile load carried by anchors close to edge which may cause side-face blowout					*ACI318 M-08*
along pedestal width edge	N_{buw}	$= N_u \times n_{bw} / n_t$	= 44.5	[kN]	RD.5.4.2
	c	$= \min(c_1, c_3)$	= 127	[mm]	
Check if side blowout applicable	h_{ef}	= 305 [mm]			*A23.3-04 (R2010)*
		< 2.5c side bowout is NOT applicable			D.6.4.1
Check if edge anchors work as a	s_{22}	= 0 [mm]	$s = s_2$ = 0	[mm]	
a group or work individually		< 6c side bowout is NOT applicable			D.6.4.2
Single anchor SB resistance	$N_{sbr,w}$	$= 13.3c\sqrt{A_{brg}}\ \phi_c\ \sqrt{f'_c}\ R_{t,c}$	= 0.0	[kN]	D.6.4.1 (D-18)
Multiple anchors SB resistance	$N_{sbgr,w}$	=			
work as a group - not applicable		$= (1+s/\ 6c) \times N_{sbr,w}$	= 0.0	[kN]	D.6.4.2 (D-19)
work individually - not applicable		$= n_{bw} \times N_{sbr,w} \times [1+(c_2 \text{ or } c_4) / c] / 4$	= 0.0	[kN]	D.6.4.1
Seismic design strength reduction		= x 1.0 not applicable	= 0.0	[kN]	D.4.3.5
	ratio	= 0.00	< N_{buw}	**OK**	
Failure Along Pedestal Depth Edge					
Tensile load carried by anchors close to edge which may cause side-face blowout					*ACI318 M-08*
along pedestal depth edge	N_{bud}	$= N_u \times n_{bd} / n_t$	= 44.5	[kN]	RD.5.4.2
	c	$= \min(c_2, c_4)$	= 127	[mm]	
Check if side blowout applicable	h_{ef}	= 305 [mm]			*A23.3-04 (R2010)*
		< 2.5c side bowout is NOT applicable			D.6.4.1
Check if edge anchors work as a	s_{11}	= 0 [mm]	$s = s_1$ = 0	[mm]	
a group or work individually		< 6c side bowout is NOT applicable			D.6.4.2
Single anchor SB resistance	$N_{sbr,d}$	$= 13.3c\sqrt{A_{brg}}\ \phi_c\ \sqrt{f'_c}\ R_{t,c}$	= 0.0	[kN]	D.6.4.1 (D-18)
Multiple anchors SB resistance	$N_{sbgr,d}$	=			
work as a group - not applicable		$= (1+s/\ 6c) \times \phi_{t,c}\ N_{sbr,d}$	= 0.0	[kN]	D.6.4.2 (D-19)
work individually - not applicable		$= n_{bd} \times N_{sbr,d} \times [1+(c_1 \text{ or } c_3) / c] / 4$	= 0.0	[kN]	D.6.4.1
Seismic design strength reduction		= x 1.0 not applicable	= 0.0	[kN]	D.4.3.5
	ratio	= 0.00	< N_{bud}	**OK**	

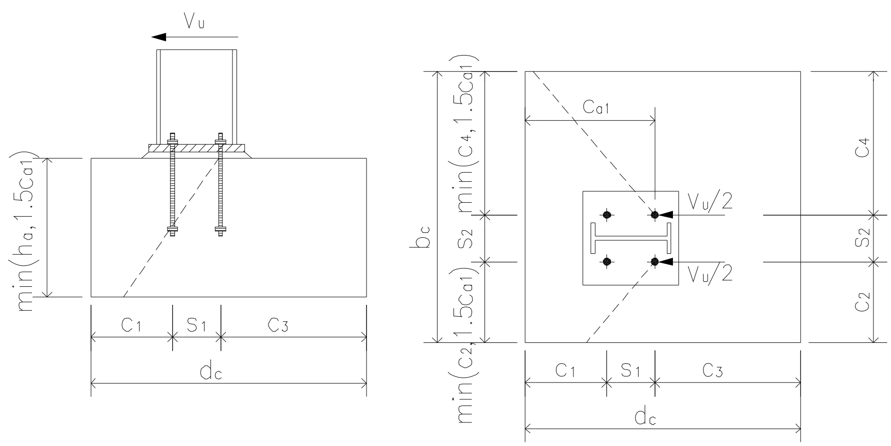
				Code Reference
Group side blowout resistance	$N_{sbgr} = \min\left(\frac{N_{sbgr,w}}{n_{bw}} n_t \ , \ \frac{N_{sbgr,d}}{n_{bd}} n_t\right)$	= 0.0	[kN]	*A23.3-04 (R2010)*
Govern Tensile Resistance	$\mathbf{N_r} = \min (N_{sr}, N_{rbr}, N_{cpr}, N_{sbgr})$	= **153.7**	[kN]	
Stud Shear Resistance	$V_{sr} = n_s A_{se} \phi_s f_{uta} R_{v,s}$	= 289.5	[kN]	D.7.1.2 (a) (D-20)
Reduction due to built-up grout pads	= x 1.0 , not applicable	= 289.5	[kN]	D.7.1.3
	ratio = 0.15	> V_u	**OK**	

Conc. Shear Breakout Resistance *ACI318 M-08*

Only Case 2 needs to be considered when anchors are rigidly connected to the attachment — Fig. RD.6.2.1(b) notes

This applies to welded stud case so only Mode 2 is considered for shear checking — in Case 2

Mode 2 Failure cone at back anchors



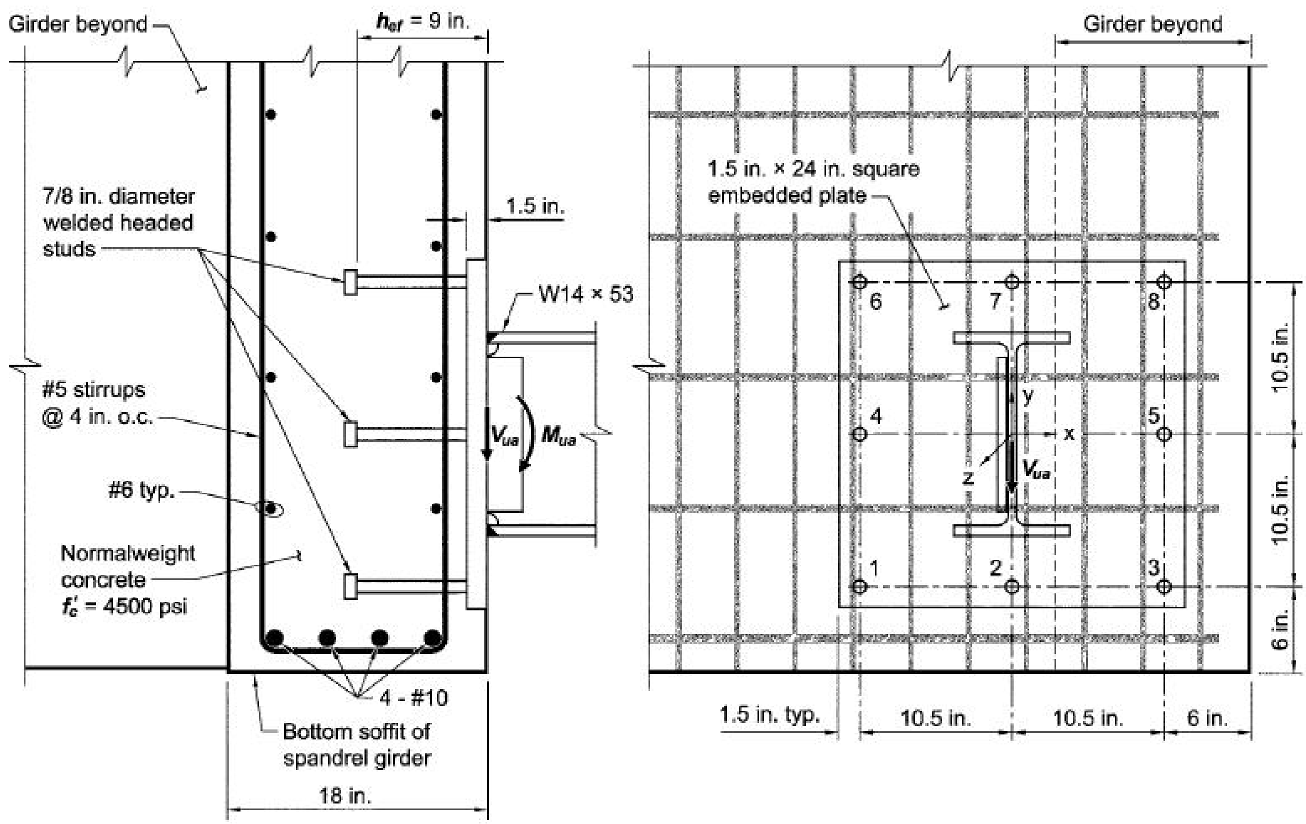
A23.3-04 (R2010)

Bolt edge distance	$c_{a1} = c_1 + s_1$	= 533	[mm]	
Limiting c_{a1} when anchors are influenced by 3 or more edges		= Yes		D.7.2.4
Bolt edge distance - adjusted	$\mathbf{c_{a1}}$ = ca1 needs to be adjusted	= **254**	[mm]	D.7.2.4
	c_2 =	127	[mm]	
	$1.5c_{a1}$ =	381	[mm]	
	$A_{Vc} = [\min(c_2,1.5c_{a1}) + s_2 + \min(c_4,1.5c_{a1})] \times \min(1.5c_{a1}, h_a)$	= 2.5E+05	[mm^2]	D.7.2.1
	$A_{Vco} = 4.5c_{a1}^{\ 2}$	= 2.9E+05	[mm^2]	D.7.2.1 (D-24)
	$A_{Vc} = \min (A_{Vc}, \ n_2 A_{Vco})$	= 2.5E+05	[mm^2]	D.7.2.1
	$l_e = \min(8d_a , h_{ef})$	= 203	[mm]	D.3
	$V_{br} = 0.66\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a} \ \phi_c \sqrt{f'_c} \ c_{a1}^{1.5} R_{v,c}$	= 84.9	[kN]	D.7.2.3 (D-26)

				Code Reference
				A23.3-04 (R2010)
Eccentricity effects	$\Psi_{ec,v}$ = 1.0 shear acts through center of group			D.7.2.5
Edge effects	$\Psi_{ed,v}$ = min[(0.7+0.3c$_2$/1.5c$_{a1}$), 1.0]	= 0.80		D.7.2.6
Concrete cracking	$\Psi_{c,v}$ =	= 1.20		D.7.2.7
Member thickness	$\Psi_{h,v}$ = max[(sqrt(1.5c$_{a1}$ / h$_a$) , 1.0]	= 1.00		D.7.2.8
Conc shear breakout resistance	$V_{cbgr} = \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V}\ \Psi_{ed,V}\ \Psi_{c,V}\ \Psi_{h,V}\ V_{br}$	= 70.6	[kN]	D.7.2.1 (D-23)
Seismic design strength reduction	= x 1.0 not applicable	= 70.6	[kN]	D.4.3.5
	ratio = 0.63	> V_u	**OK**	
Conc. Pryout Shear Resistance				
	k_{cp} = 2.0			D.7.3
Factored shear pryout resistance	$V_{cpgr} = k_{cp} N_{cbgr}$	= 267.3	[kN]	D.7.3 (D-32)
	$R_{v,c}$ = 1.00 pryout strength is always Condition B			D.5.4(c)
Seismic design strength reduction	= x 1.0 not applicable	= 267.3	[kN]	D.4.3.5
	ratio = 0.17	> V_u	**OK**	
				CSA S16-09
Stud on Conc Bearing	$B_r = n_s \times 1.4 \times \phi_c \times \min(8d_a, h_{ef}) \times d_a \times f_c'$	= 291.2	[kN]	25.3.3.2
	ratio = 0.15	> V_u	**OK**	
Govern Shear Resistance	$\mathbf{V_r}$ = min (V_{sr}, V_{cbgr}, V_{cpgr} , B_r)	= **70.6**	[kN]	
				A23.3-04 (R2010)
Tension Shear Interaction				
Check if N_u >0.2 N_r and V_u >0.2 V_r	Yes			D.8.2 & D.8.3
	$N_u/N_r + V_u/V_r$	= 1.21		D.8.4 (D-35)
	ratio = 1.01	> 1.2	**NG**	
Ductility Tension				
	N_{sr} = 617.7 [kN]			
	> min (N_{cbgr}, N_{cpr}, N_{sbgr})	= 153.7	[kN]	
	Non-ductile			
Ductility Shear				
	V_{sr} = 289.5 [kN]			
	> min (V_{cbgr}, V_{cpgr}, B_r)	= 70.6	[kN]	
	Non-ductile			

Example 33: Welded Stud + No Anchor Reinft + Tension Shear & Moment + ACI 318-08 Code

This example taken from Example 10 on page 82 of *ACI 355.3R-11 Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D*



M_u = 30 kip-ft N_u = 0 kips, V_u=20 kips, f_c' = 4.5 ksi

Anchor stud d_a=7/8 in h_{ef} =9 in h_a=18 in

Supplementary reinforcement Tension → Condition B Shear → Condition A $\Psi_{c,V}$ =1.2

Provide built-up grout pad Seismic is not a consideration

Field welded plate washers to base plate at each anchor

Notes:

There are two locations in this calculation which are different from calculation in ACI 355.3R-11 Example 10

1. Concrete tension breakout A_{Nc} = 1215 in^2, different from A_{Nc} = 1519 in^2 , value in ACI 355.3R-11 page 86.
   We assume the moment may apply in both directions. When moment causes tensile anchors being close to the edge side, the A_{Nc} value is consequently reduced.
2. Concrete shear breakout c_{a1} reduction from 27" to 12" in ACI 355.3R-11 page 90 is not correct. It doesn't comply with both edge distances $c_{a2,1}$<1.5c_{a1} and $c_{a2,2}$<1.5c_{a1} . Refer to ACI 318-11 Fig. RD.6.2.4 for more details.

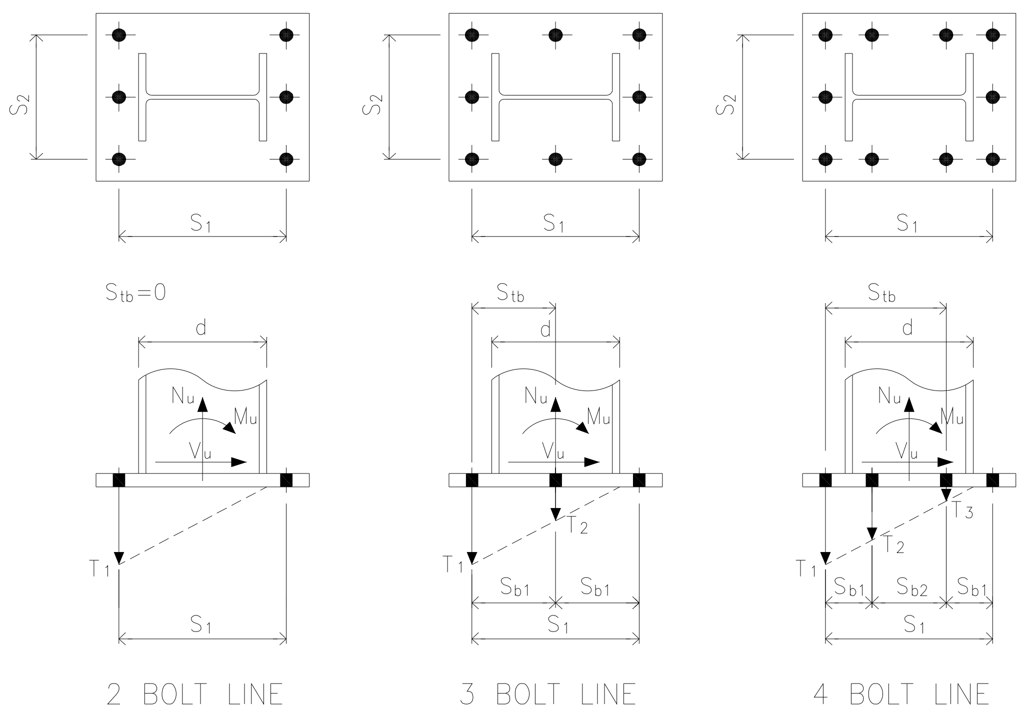
STUD ANCHOR DESIGN **Combined Tension, Shear and Moment**

Anchor bolt design based on	**Code Abbreviation**
ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D	*ACI 318-08*
PIP STE05121 Anchor Bolt Design Guide-2006	*PIP STE05121*

Assumptions	**Code Reference**
1. Concrete is cracked	*ACI 318-08*
2. Condition B - no supplementary reinforcement provided	D.4.4 (c)
3. Load combinations shall be per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2	D.4.4
4. Shear load acts through center of bolt group $\Psi_{ec,V}$ =1.0	D.6.2.5
5. For anchor group subject to moment, the anchor tensile load is designed using elastic analysis and there is no redistribution of the forces between highly stressed and less stressed anchors	D.3.1
6. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output	

Anchor Stud Data

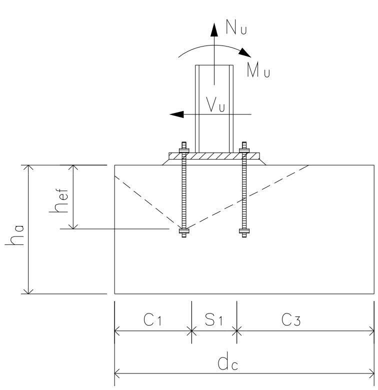
Factored moment	M_u =	30.0	[kip-ft]	= 40.7	[kNm]
Factored tension /compression	N_u =	0.0	[kips]	= 0.0	[kN]
Factored shear	$\mathbf{V_u}$ =	**20.0**	[kips]	= 89.0	[kN]

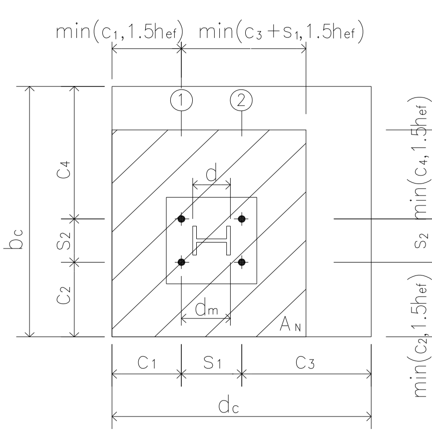


No of bolt line for resisting moment = 3 Bolt Line

No of bolt along outermost bolt line = 3

								Code Reference
				min required				*PIP STE05121*
Outermost stud line spacing s_1	s_1 =	21.0	[in]	3.5	**OK**			Page A -1 Table 1
Outermoststud line spacing s_2	s_2 =	21.0	[in]	3.5	**OK**			
Internal stud line spacing s_{b1}	s_{b1} =	10.5	[in]	3.5	**OK**			
Internal stud line spacing s_{b2}	s_{b2} =	0.0	[in]	3.5	**OK**			
Column depth	d =	13.9	[in]					
Concrete strength	f'_c =	4.5	[ksi]			= 31.0	[MPa]	
Stud material	=	AWS D1.1 Grade B						
Stud tensile strength	f_{uta} =	65	[ksi]			= 448	[MPa]	*ACI 318-08*
		Stud is ductile steel element						D.1
Stud diameter	d_a =	**0.875**	[in]			= 22.2	[mm]	
Stud shank area	A_{se} =	0.60	[in^2]			= 388	[mm^2]	
Stud head bearing area	A_{brg} =	0.88	[in^2]			= 570	[mm^2]	
				min required				*PIP STE05121*
Stud embedment depth	h_{ef} =	9.0	[in]	10.5	**Warn**			Page A -1 Table 1
Concrete thickness	h_a =	18.0	[in]	12.0	**OK**			
Stud edge distance c_1	c_1 =	6.0	[in]	4.5	**OK**			Page A -1 Table 1
Stud edge distance c_2	c_2 =	6.0	[in]	4.5	**OK**			
Stud edge distance c_3	c_3 =	100.0	[in]	4.5	**OK**			
Stud edge distance c_4	c_4 =	100.0	[in]	4.5	**OK**			*ACI 318-08*
$c_i > 1.5h_{ef}$ for at least two edges to avoid reducing of h_{ef} when $N_u > 0$					**Yes**			D.5.2.3
Adjusted h_{ef} for design	h_{ef} =	**9.00**	[in]	10.5	**Warn**			D.5.2.3





						Code Reference
Number of stud at bolt line 1	n_1 =	3				
Number of stud at bolt line 2	n_2 =	3				
Total no of welded stud	n =	8				
Number of stud carrying tension	n_t =	5				
Number of stud carrying shear	n_s =	3				
						ACI 318-08
Seismic design category >= C	=	No	?			D.3.3.3
Supplementary reinforcement						
For tension	=	No	Condition B			D.4.4 (c)
For shear	$\Psi_{c,V}$ =	1.2	Condition A	?		D.6.2.7
Provide built-up grout pad ?	=	No	?			D.6.1.3
Strength reduction factors						
Anchor reinforcement	ϕ_s =	0.75				D.5.2.9 & D.6.2.9
Anchor rod - ductile steel	$\phi_{t,s}$ =	0.75		$\phi_{v,s}$ = 0.65		D.4.4 (a)
Concrete	$\phi_{t,c}$ =	0.70	Cdn-B	$\phi_{v,c}$ = 0.75	Cdn-A	D.4.4 (c)

CONCLUSION

Abchor Rod Embedment, Spacing and Edge Distance			**Warn**
Overall		ratio = **0.95**	**OK**
Tension			
Stud Tensile Resistance		ratio = 0.21	**OK**
Conc. Tensile Breakout Resistance		ratio = 0.64	**OK**
Stud Pullout Resistance		ratio = 0.28	**OK**
Side Blowout Resistance		ratio = 0.00	**OK**
Shear			
Stud Shear Resistance		ratio = 0.26	**OK**
Conc. Shear Breakout Resistance		ratio = 0.50	**OK**
Conc. Pryout Shear Resistance		ratio = 0.27	**OK**
Tension Shear Interaction			
Tension Shear Interaction		ratio = 0.95	**OK**
Ductility			*ACI 318-08*
	Tension **Non-ductile**	**Shear** **Non-ductile**	
Seismic Design Requirement			**OK** D.3.3.4

SDC< C, ACI318-08 D.3.3 ductility requirement is NOT required

CALCULATION					**Code Reference**
Anchor Stud Tensile Force					
Single bolt tensile force	T_1 = 6.22 [kips]	No of bolt for T_1 n_{T1} = 3			
	T_2 = 2.48 [kips]	No of bolt for T_2 n_{T2} = 2			
	T_3 = 0.00 [kips]	No of bolt for T_3 n_{T3} = 0			
Sum of bolt tensile force	$\mathbf{N_u} = \Sigma\, n_{Ti}\, T_i$		= **23.6**	[kips]	
Tensile bolts outer distance s_{tb}	s_{tb} = 10.5 [in]				
Eccentricity e'_N -- distance between resultant of tensile load and centroid of anchors					*ACI 318-08*
loaded in tension	e'_N = 2.00 [in]				Fig. RD.5.2.4 (b)
Eccentricity modification factor	$\Psi_{ec,N} = \dfrac{1}{\left(1+\dfrac{2e'_N}{3h_{ef}}\right)}$		= 0.87		D.5.2.4 (D-9)
Stud Tensile Resistance	$\phi_{t,s}\, N_{sa} = \phi_{t,s}\, A_{se}\, f_{uta}$		= 29.3	[kips]	D.5.1.2 (D-3)
	ratio = 0.21		> T_1	**OK**	
Conc. Tensile Breakout Resistance					
	$N_b = 24\ \lambda\sqrt{f'_c}\ h_{ef}^{1.5}$ if $h_{ef} < 11"$ or $h_{ef} > 25"$		= 43.5	[kips]	D.5.2.2 (D-7)
	$16\ \lambda\sqrt{f'_c}\ h_{ef}^{5/3}$ if $11" \le h_{ef} \le 25"$				D.5.2.2 (D-8)
Projected conc failure area	$1.5h_{ef}$ =		= 13.50	[in]	
	$A_{Nc} = [s_{tb}+\min(c_1,1.5h_{ef})+\min(c_3,1.5h_{ef})]\text{x}$ $[s_2+\min(c_2,1.5h_{ef})+\min(c_4,1.5h_{ef})]$		= 1215.0	[in^2]	
	$A_{Nco} = 9\ h_{ef}^2$		= 729.0	[in^2]	D.5.2.1 (D-6)
	$A_{Nc} = \min(A_{Nc},\ n_t\, A_{Nco})$		= 1215.0	[in^2]	D.5.2.1
Min edge distance	$c_{min} = \min(c_1, c_2, c_3, c_4)$		= 6.0	[in]	
Eccentricity effects	$\Psi_{ec,N}$ =		= 0.87		D.5.2.4 (D-9)
Edge effects	$\Psi_{ed,N} = \min[\,(0.7+0.3c_{min}/1.5h_{ef}),\ 1.0\,]$		= 0.83		D.5.2.5
Concrete cracking	$\Psi_{c,N}$ = 1.0 for cracked concrete				D.5.2.6
Concrete splitting	$\Psi_{cp,N}$ = 1.0 for cast-in anchor				D.5.2.7
Concrete breakout resistance	$\phi_{t,c}\, N_{cbg} = \phi_{t,c}\dfrac{A_{Nc}}{A_{Nco}}\Psi_{ec,N}\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N}\ N_b$		= 36.8	[kips]	D.5.2.1 (D-5)
Seismic design strength reduction	= x 1.0 not applicable		= 36.8	[kips]	D.3.3.3
	ratio = 0.64		> N_u	**OK**	
Stud Pullout Resistance					
Single bolt pullout resistance	$N_p = 8\ A_{brg}\ f_c'$		= 31.8	[kips]	D.5.3.4 (D-15)
	$\phi_{t,c}\, N_{pn} = \phi_{t,c}\ \Psi_{c,p}\ N_p$		= 22.3	[kips]	D.5.3.1 (D-14)
Seismic design strength reduction	= x 1.0 not applicable		= 22.3	[kips]	D.3.3.3
	ratio = 0.28		> T_1	**OK**	
	$\Psi_{c,p}$ = 1 for cracked conc				D.5.3.6
	$\phi_{t,c}$ = 0.70	pullout strength is always Condition B			D.4.4(c)

Side Blowout Resistance — **Code Reference**

Failure Along Pedestal Width Edge — *ACI 318-08*

Tensile load carried by anchors close to edge which may cause side-face blowout

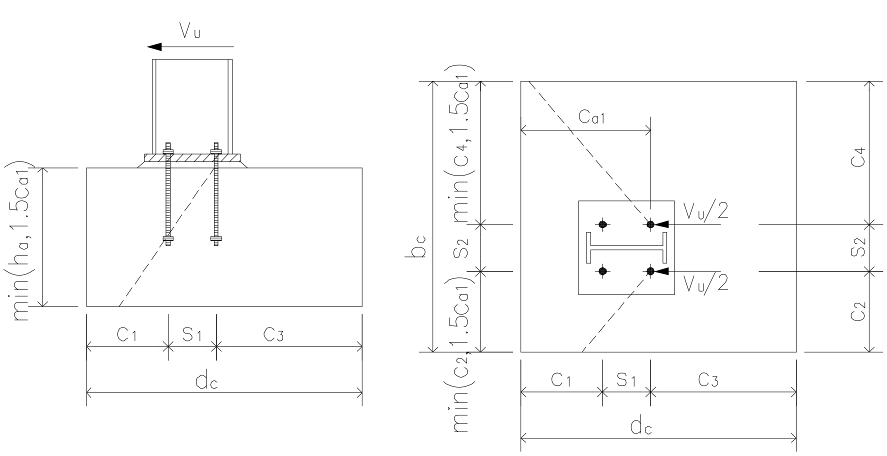
along pedestal width edge	N_{buw} = $n_{T1}\ T_1$	= 18.7	[kips]		RD.5.4.2
	c = min (c_1, c_3)	= 6.0	[in]		
Check if side blowout applicable	h_{ef} = 9.0 [in]				
	< 2.5c	side bowout is NOT applicable			D.5.4.1
Check if edge anchors work as a	s_{22} = 0.0 [in]	$s = s_2$ = 0.0	[in]		
a group or work individually	< 6c	side bowout is NOT applicable			D.5.4.2
Single anchor SB resistance	$\phi_{t,c}\ N_{sb} = \phi_{t,c}\left(160\,c\sqrt{A_{brg}}\right)\lambda\sqrt{f'_c}$	= 0.0	[kips]		D.5.4.1 (D-17)
Multiple anchors SB resistance	$\phi_{t,c} N_{sbg,w}$ =				
work as a group - not applicable	= (1+s/ 6c) x $\phi_{t,c}\ N_{sb}$	= 0.0	[kips]		D.5.4.2 (D-18)
work individually - not applicable	= n_{bw} x $\phi_{t,c}\ N_{sb}$ x [1+(c_2 or c_4) / c] / 4	= 0.0	[kips]		D.5.4.1
Seismic design strength reduction	= x 1.0 not applicable	= 0.0	[kips]		D.3.3.3
	ratio = 0.00	< N_{buw}		OK	
Group side blowout resistance	$\phi_{t,c}\ N_{sbg} = \phi_{t,c}\ \frac{N_{sbgr,w}}{n_{T1}} n_t$	= 0.0	[kips]		
Govern Tensile Resistance	$\mathbf{N_r}$ = min [$\phi_{t,s}\ n_t\ N_{sa}$, $\phi_{t,c}$ (N_{cbg}, $n_t\ N_{pn}$, N_{sbg})]	= **36.8**	[kips]		
Stud Shear Resistance	$\phi_{v,s}\ V_{sa} = \phi_{v,s}\ n_s\ A_{se}\ f_{uta}$	= 76.2	[kips]		D.6.1.2 (a) (D-19)
Reduction due to built-up grout pads	= x 1.0 , not applicable	= 76.2	[kips]		D.6.1.3
	ratio = 0.26	> V_u		OK	

Conc. Shear Breakout Resistance

Only Case 2 needs to be considered when anchors are rigidly connected to the attachment — Fig. RD.6.2.1(b) notes

This applies to welded stud case so only Mode 2 is considered for shear checking — in Case 2

Mode 2 Failure cone at back anchors



					Code Reference
Bolt edge distance	c_{a1} =		= 27.0	[in]	*ACI 318-08*
Limiting c_{a1} when anchors are influenced by 3 or more edges			= No		D.6.2.4
Bolt edge distance - adjusted	$\mathbf{c_{a1}}$ =	ca1 needs NOT to be adjusted	= **27.0**	[in]	D.6.2.4
	c_2 =		6.0	[in]	
	$1.5c_{a1}$ =		40.5	[in]	
	A_{Vc} =	$[\min(c_2, 1.5c_{a1}) + s_2 + \min(c_4, 1.5c_{a1})] \times \min(1.5c_{a1}, h_a)$	= 1215.0	[in^2]	D.6.2.1
	A_{Vco} =	$4.5c_{a1}^2$	= 3280.5	[in^2]	D.6.2.1 (D-23)
	A_{Vc} =	$\min(A_{Vc},\ n_2 A_{Vco})$	= 1215.0	[in^2]	D.6.2.1
	l_e =	$\min(8d_a, h_{ef})$	= 7.0	[in]	D.6.2.2
	V_b =	$\left[8\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right]\lambda\sqrt{f'_c}\ c_{a1}^{1.5}$	= 106.7	[kips]	D.6.2.3 (D-25)
Eccentricity effects	$\Psi_{ec,v}$ =	1.0 shear acts through center of group			D.6.2.5
Edge effects	$\Psi_{ed,v}$ =	$\min[(0.7+0.3c_2/1.5c_{a1}), 1.0]$	= 0.74		D.6.2.6
Concrete cracking	$\Psi_{c,v}$ =		= 1.20		D.6.2.7
Member thickness	$\Psi_{h,v}$ =	$\max[(\mathrm{sqrt}(1.5c_{a1} / h_a), 1.0]$	= 1.50		D.6.2.8
Conc shear breakout resistance	V_{cbg2} =	$\phi_{v,c}\frac{A_{Vc}}{A_{Vco}}\Psi_{ec,V}\ \Psi_{ed,V}\ \Psi_{c,V}\ \Psi_{h,V}\ V_b$	= 39.7	[kips]	D.6.2.1 (D-22)
Seismic design strength reduction	=	x 1.0 not applicable	= 39.7	[kips]	D.3.3.3
	ratio =	0.50	> V_u	**OK**	

Conc. Pryout Shear Resistance

	k_{cp} =	2.0			D.6.3
Factored shear pryout resistance	$\phi_{v,c} V_{cpg}$ =	$\phi_{v,c}\ k_{cp}\ N_{cbg}$	= 73.6	[kips]	D.6.3 (D-31)
	$\phi_{v,c}$ =	0.70 pryout strength is always Condition B			D.4.4(c)
Seismic design strength reduction	=	x 1.0 not applicable	= 73.6	[kips]	D.3.3.3
	ratio =	0.27	> V_u	**OK**	
Govern Shear Resistance	$\mathbf{V_r}$ =	$\min[\phi_{v,s}V_{sa}, \phi_{v,c}(V_{cbg}, V_{cpg})]$	= **39.7**	[kips]	

Tension Shear Interaction

Check if $N_u > 0.2\phi\ N_n$ and $V_u > 0.2\phi\ V_n$		Yes			D.7.1 & D.7.2
		$N_u/N_r + V_u/V_r$	= 1.14		D.7.3 (D-32)
	ratio =	0.95	< 1.2	**OK**	

Ductility Tension

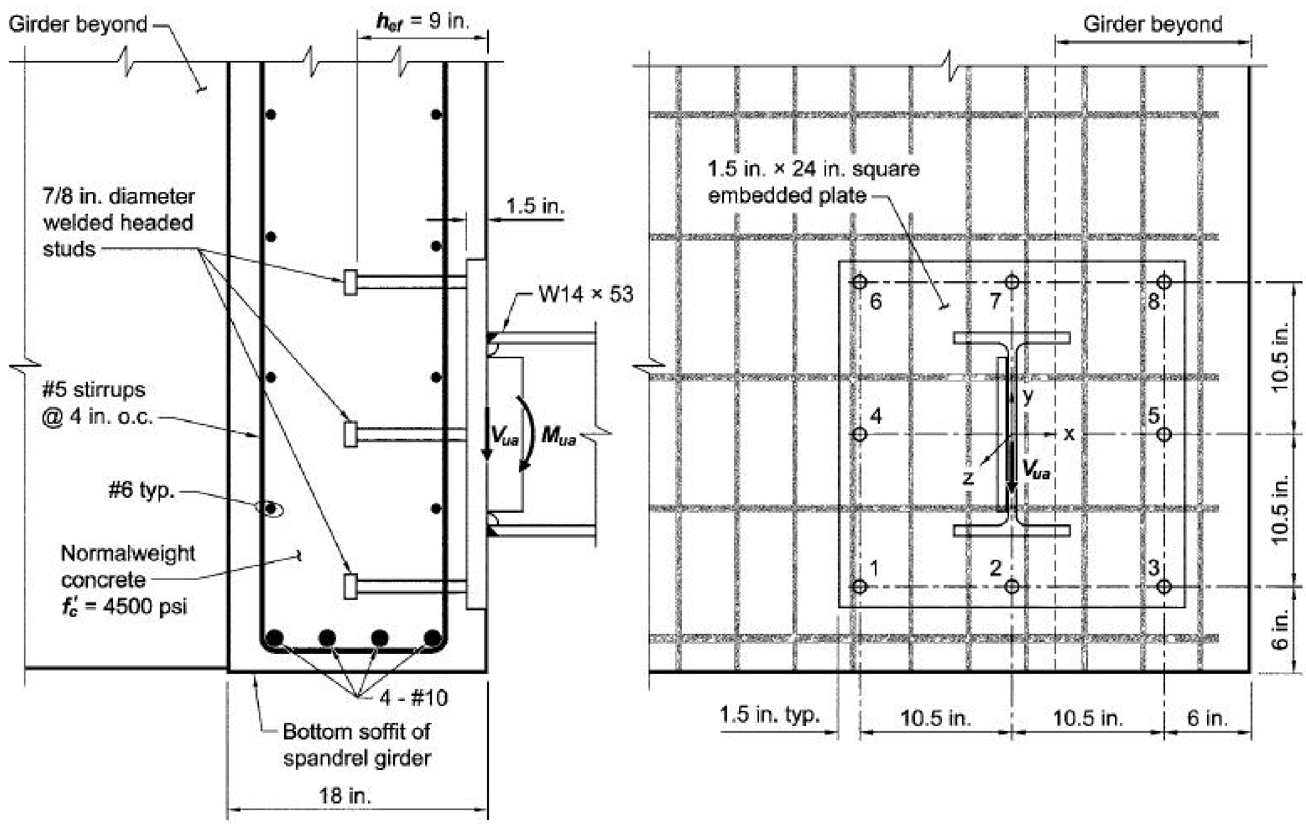
$\phi_{t,s}\ N_{sa}$ =	29.3 [kips]			
>	$\phi_{t,c}\min(N_{cbg}, N_{pn}, N_{sbg})$		= 22.3	[kips]
		Non-ductile		

Ductility Shear

$\phi_{v,s}\ V_{sa}$ =	76.2 [kips]			
>	$\phi_{v,c}\min(V_{cbg}, V_{cpg})$		= 39.7	[kips]
		Non-ductile		

Example 34: Welded Stud + No Anchor Reinft + Tension Shear & Moment + CSA A23.3-04 Code

This example taken from Example 10 on page 82 of *ACI 355.3R-11 Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D*



M_u = 40.7 kNm N_u = 0 kN, V_u=89 kN, f_c' = 31 MPa

Anchor stud d_a=7/8 in h_{ef} =229mm h_a=457mm

Supplementary reinforcement Tension → Condition B Shear → Condition A $\Psi_{c,V}$ =1.2

Provide built-up grout pad Seismic is not a consideration

Field welded plate washers to base plate at each anchor

Notes:

There are two locations in this calculation which are different from calculation in ACI 355.3R-11 Example 10

1. Concrete tension breakout A_{Nc} = 1215 in^2, different from A_{Nc} = 1519 in^2 , value in ACI 355.3R-11 page 86.
   We assume the moment may apply in both directions. When moment causes tensile anchors being close to the edge side, the A_{Nc} value is consequently reduced.
2. Concrete shear breakout c_{a1} reduction from 27" to 12" in ACI 355.3R-11 page 90 is not correct. It doesn't comply with <u>both</u> edge distances $c_{a2,1}$<1.5c_{a1} and $c_{a2,2}$<1.5c_{a1} . Refer to ACI 318-11 Fig. RD.6.2.4 for more details.

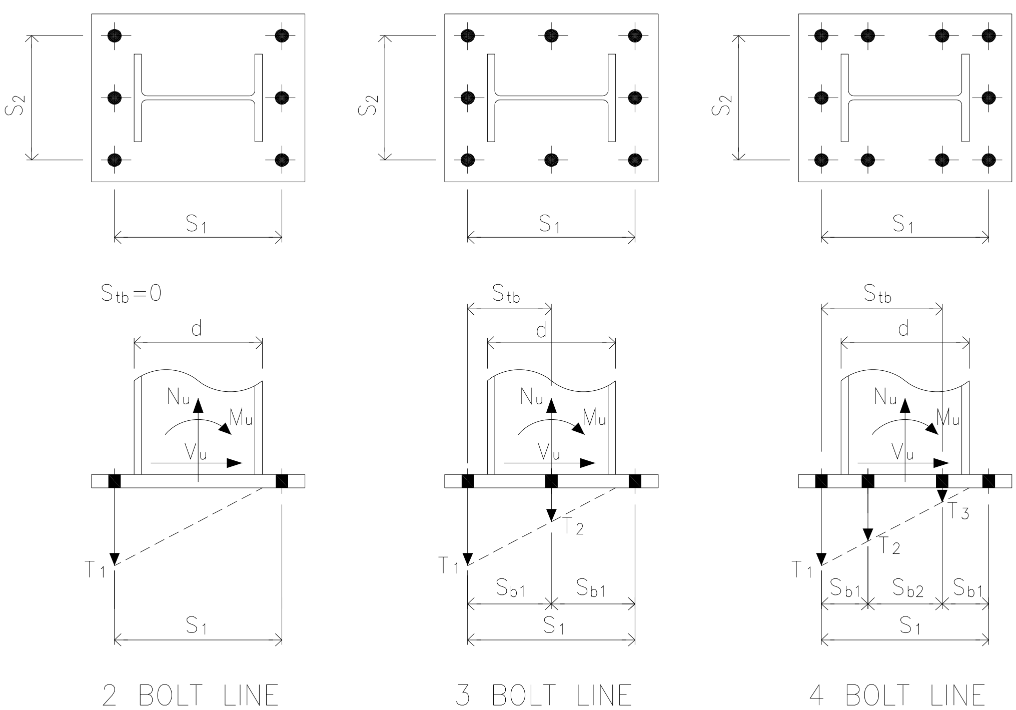
STUD ANCHOR DESIGN **Combined Tension, Shear and Moment**

Anchor bolt design based on	**Code Abbreviation**
CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D	*A23.3-04 (R2010)*
ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary	*ACI318 M-08*
PIP STE05121 Anchor Bolt Design Guide-2006	*PIP STE05121*

Assumptions	**Code Reference**
	A23.3-04 (R2010)
1. Concrete is cracked	
2. Condition B for tension - no supplementary reinforcement provided	D.5.4 (c)
3. Shear load acts through center of bolt group $\Psi_{ec,V}$ =1.0	D.7.2.5
4. For anchor group subject to moment, the anchor tensile load is designed using elastic analysis and there is no redistribution of the forces between highly stressed and less stressed anchors	D.4.1
5. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output	

Anchor Stud Data

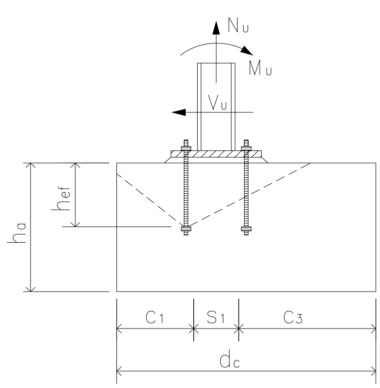
Factored moment	M_u =	40.7	[kNm]	= 30.0	[kip-ft]
Factored tension /compression	N_u =	0.0	[kN]	= 0.0	[kips]
Factored shear	$\mathbf{V_u}$ =	**89.0**	[kN]	= 20.0	[kips]

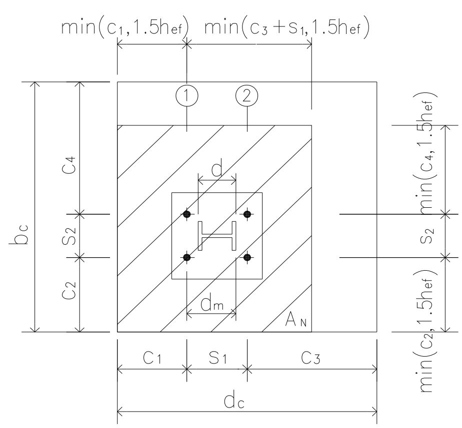


No of bolt line for resisting moment = 3 Bolt Line

No of bolt along outermost bolt line = 3

				min required				**Code Reference**
Outermost stud line spacing s_1	s_1 =	533	[mm]	89	**OK**			*PIP STE05121*
Outermost stud line spacing s_2	s_2 =	533	[mm]	89	**OK**			Page A -1 Table 1
Internal stud line spacing s_{b1}	s_{b1} =	267	[mm]	89	**OK**			
Internal stud line spacing s_{b2}	s_{b2} =	0	[mm]	89	**OK**			
Column depth	d =	353	[mm]					
Concrete strength	f'_c =	31	[MPa]			= 4.5	[ksi]	
Anchor bolt material	=	AWS D1.1 Grade B						
Anchor tensile strength	f_{uta} =	65	[ksi]			= 448	[MPa]	*A23.3-04 (R2010)*
		Stud is ductile steel element						D.2
Stud diameter	$\mathbf{d_a}$ =	**0.875**	[in]			= 22.2	[mm]	
Stud shank area	A_{se} =	0.60	[in^2]			= 388	[mm^2]	
Stud head bearing area	A_{brg} =	0.88	[in^2]			= 570	[mm^2]	
				min required				*PIP STE05121*
Anchor bolt embedment depth	h_{ef} =	229	[mm]	267	**Warn**			Page A -1 Table 1
Concrete thickness	h_a =	457	[mm]	305	**OK**			
Stud edge distance c_1	c_1 =	152	[mm]	115	**OK**			Page A -1 Table 1
Stud edge distance c_2	c_2 =	152	[mm]	115	**OK**			
Stud edge distance c_3	c_3 =	2540	[mm]	115	**OK**			
Stud edge distance c_4	c_4 =	2540	[mm]	115	**OK**			*A23.3-04 (R2010)*
$c_i > 1.5h_{ef}$ for at least two edges to avoid reducing of h_{ef} when $N_u > 0$					**Yes**			D.6.2.3
Adjusted h_{ef} for design	h_{ef} =	**229**	[mm]	267	**Warn**			D.6.2.3





No of stud at bolt line 1	n_1	= 3					**Code Reference**
No of stud at bolt line 2	n_2	= 3					*A23.3-04 (R2010)*
Total no of welded stud	n	= 8					
No of stud carrying tension	n_t	= 5					
No of stud carrying shear	n_s	= 3					
Seismic region where $I_EF_aS_a(0.2)$>=0.35		= No	?				D.4.3.5
Supplementary reinforcement							
For tension		= No	Condition B				D.5.4 (c)
For shear	$\Psi_{c,V}$	= 1.2	Condition A	?			D.7.2.7
Provide built-up grout pad ?		= No	?				D.7.1.3
Strength reduction factors							
Anchor reinforcement factor	ϕ_{as}	= 0.75					D.7.2.9
Steel anchor resistance factor	ϕ_s	= 0.85					8.4.3 (a)
Concrete resistance factor	ϕ_c	= 0.65					8.4.2
Resistance modification factors							
Anchor rod - ductile steel	$R_{t,s}$	= 0.80		$R_{v,s}$	= 0.75		D.5.4(a)
Concrete	$R_{t,c}$	= 1.00	Cdn-B	$R_{v,c}$	= 1.15	Cdn-A	D.5.4(c)

CONCLUSION

Abchor Rod Embedment, Spacing and Edge Distance		**Warn**	
Overall	ratio = **1.00**	**NG**	
Tension			
Stud Tensile Resistance	ratio = 0.23	**OK**	
Conc. Tensile Breakout Resistance	ratio = 0.69	**OK**	
Stud Pullout Resistance	ratio = 0.30	**OK**	
Side Blowout Resistance	ratio = 0.00	**OK**	
Shear			
Stud Shear Resistance	ratio = 0.27	**OK**	
Conc. Shear Breakout Resistance	ratio = 0.51	**OK**	
Conc. Pryout Shear Resistance	ratio = 0.29	**OK**	
Stud on Conc Bearing	ratio = 0.27	**OK**	
Tension Shear Interaction			
Tension Shear Interaction	ratio = 1.00	**NG**	
Ductility			*A23.3-04 (R2010)*
Tension **Non-ductile**	**Shear** **Non-ductile**		
Seismic Design Requirement		**OK**	D.4.3.6

IeFaSa(0.2)<0.35, A23.3-04 D.4.3.3 ductility requirement is NOT required

CALCULATION — **Code Reference**

Anchor Tensile Force — *A23.3-04 (R2010)*

Single stud tensile force	$T_1 = 27.7$	[kN]	No of stud for T_1 $n_{T1} = 3$		
	$T_2 = 11.0$	[kN]	No of stud for T_2 $n_{T2} = 2$		
	$T_3 = 0.0$	[kN]	No of stud for T_3 $n_{T3} = 0$		
Sum of stud tensile force	$\mathbf{N_u} = \Sigma\, n_i\, T_i$		= **105.1**	[kN]	

Tensile studs outer distance s_{tb} — $s_{tb} = 267$ [mm]

Eccentricity e'_N -- distance between resultant of tensile load and centroid of studs loaded in tension — $e'_N = 51$ [mm] — Figure D.8 (b)

Eccentricity modification factor — $\Psi_{ec,N} = \dfrac{1}{\left(1+\dfrac{2e'_N}{3h_{ef}}\right)}$ = 0.87 — D.6.2.4 (D-9)

Stud Tensile Resistance — $N_{sr} = A_{se}\,\phi_s\,f_{uta}\,R_{t,s}$ = 118.2 [kN] — D.6.1.2 (D-3)

ratio = 0.23 > T_1 **OK**

Conc. Tensile Breakout Resistance

$N_{br} = 10\,\phi_c\sqrt{f'_c}\,h_{ef}^{1.5}\,R_{t,c}$ if $h_{ef} \le 275$ or $h_{ef} \ge 625$ — D.6.2.2 (D-7)

$3.9\,\phi_c\sqrt{f'_c}\,h_{ef}^{5/3}\,R_{t,c}$ if $275 < h_{ef} < 625$ — D.6.2.2 (D-8)

= 125.4 [kN]

Projected conc failure area	$1.5h_{ef} =$	= 344	[mm]	
	$A_{Nc} = [s_{tb}+\min(c_1,1.5h_{ef})+\min(c_3,1.5h_{ef})]\times [s_2+\min(c_2,1.5h_{ef})+\min(c_4,1.5h_{ef})]$	= 7.8E+05	[mm^2]	
	$A_{Nco} = 9\,h_{ef}^2$	= 4.7E+05	[mm^2]	D.6.2.1 (D-6)
	$A_{Nc} = \min(A_{Nc},\ n_t\,A_{Nco})$	= 7.8E+05	[mm^2]	D.6.2.1
Min edge distance	$c_{min} = \min(c_1, c_2, c_3, c_4)$	= 152	[mm]	
Eccentricity effects	$\Psi_{ec,N} =$	= 0.87		D.6.2.4 (D-9)
Edge effects	$\Psi_{ed,N} = \min[(0.7+0.3c_{min}/1.5h_{ef}),\ 1.0]$	= 0.83		D.6.2.5
Concrete cracking	$\Psi_{c,N}$ = 1.0 for cracked concrete			D.6.2.6
Concrete splitting	$\Psi_{cp,N}$ = 1.0 for cast-in anchor			D.6.2.7

Concrete breakout resistance — $N_{cbgr} = \dfrac{A_{Nc}}{A_{Nco}}\,\Psi_{ec,N}\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N}\,N_{br}$ = 151.2 [kN] — D.6.2.1 (D-5)

Seismic design strength reduction = x 1.0 not applicable = 151.2 [kN] — D.4.3.5

ratio = 0.69 > N_u **OK**

Stud Pullout Resistance

Single bolt pullout resistance	$N_{pr} = 8\,A_{brg}\,\phi_c\,f'_c\,R_{t,c}$	= 91.9	[kN]	D.6.3.4 (D-16)
	$N_{cpr} = \Psi_{c,p}\,N_{pr}$	= 91.9	[kN]	D.6.3.1 (D-15)
Seismic design strength reduction	= x 1.0 not applicable	= 91.9	[kN]	D.4.3.5
	ratio = 0.30	> T_1	**OK**	
	$\Psi_{c,p}$ = 1 for cracked conc			D.6.3.6
	$R_{t,c}$ = 1.00 pullout strength is always Condition B			D.5.4(c)

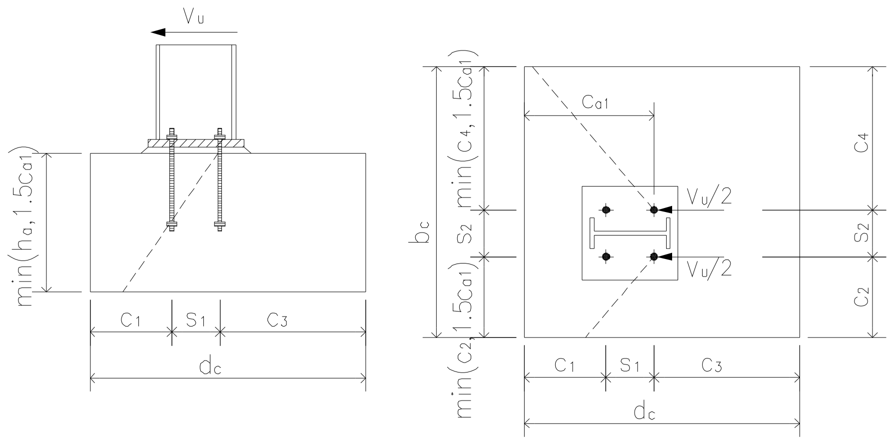
Side Blowout Resistance					**Code Reference**
Failure Along Pedestal Width Edge					
Tensile load carried by anchors close to edge which may cause side-face blowout					*ACI318 M-08*
along pedestal width edge	N_{buw}	$= n_{T1}\, T_1$	= 83.0	[kN]	RD.5.4.2
	c	= min (c_1, c_3)	= 152	[mm]	
Check if side blowout applicable	h_{ef}	= 229 [mm]			*A23.3-04 (R2010)*
		< 2.5c side bowout is NOT applicable			D.6.4.1
Check if edge anchors work as a	s_{22}	= 0 [mm]	$s = s_2$ = 0	[mm]	
a group or work individually		< 6c side bowout is NOT applicable			D.6.4.2
Single anchor SB resistance	$N_{sbr,w}$	$= 13.3c\sqrt{A_{brg}}\ \phi_c\ \sqrt{f'_c}\ R_{t,c}$	= 0.0	[kN]	D.6.4.1 (D-18)
Multiple anchors SB resistance	$N_{sbgr,w}$	=			
work as a group - not applicable		= (1+s/ 6c) x $N_{sbr,w}$	= 0.0	[kN]	D.6.4.2 (D-19)
work individually - not applicable		= n_{bw} x $N_{sbr,w}$ x [1+(c_2 or c_4) / c] / 4	= 0.0	[kN]	D.6.4.1
Seismic design strength reduction		= x 1.0 not applicable	= 0.0	[kN]	D.4.3.5
	ratio	= 0.00	< N_{buw}	**OK**	
Group side blowout resistance	N_{sbgr}	$= \dfrac{N_{sbgr,w}}{n_{T1}} n_t$	= 0.0	[kN]	
Govern Tensile Resistance	$\mathbf{N_r}$	= min ($n_t N_{sr}$, N_{rbr}, $n_t N_{cpr}$, N_{sbgr})	= **151.2**	[kN]	
Stud Shear Resistance	V_{sr}	$= n_s\, A_{se}\, \phi_s\, f_{uta}\, R_{v,s}$	= 332.5	[kN]	D.7.1.2 (a) (D-20)
Reduction due to built-up grout pads		= x 1.0 , not applicable	= 332.5	[kN]	D.7.1.3
	ratio	= 0.27	> V_u	**OK**	

Conc. Shear Breakout Resistance *ACI318 M-08*

Only Case 2 needs to be considered when anchors are rigidly connected to the attachment — Fig. RD.6.2.1(b) notes

This applies to welded stud case so only Mode 2 is considered for shear checking — in Case 2

Mode 2 Failure cone at back anchors



A23.3-04 (R2010)

Bolt edge distance	c_{a1}	= c_1 + s_1	= 685	[mm]	
Limiting c_{a1} when anchors are influenced by 3 or more edges			= No		D.7.2.4
Bolt edge distance - adjusted	$\mathbf{c_{a1}}$	= ca1 needs NOT to be adjusted	= **685**	[mm]	D.7.2.4

	c_2 =		152	[mm]	**Code Reference**
	$1.5c_{a1}$ =		1028	[mm]	*A23.3-04 (R2010)*
	A_{Vc} =	$[\min(c_2, 1.5c_{a1}) + s_2 + \min(c_4, 1.5c_{a1})] \times \min(1.5c_{a1}, h_a)$	= 7.8E+05	[mm^2]	D.7.2.1
	A_{Vco} =	$4.5c_{a1}^2$	= 2.1E+06	[mm^2]	D.7.2.1 (D-24)
	A_{Vc} =	$\min(A_{Vc},\ n_2 A_{Vco})$	= 7.8E+05	[mm^2]	D.7.2.1
	l_e =	$\min(8d_a, h_{ef})$	= 178	[mm]	D.3
	V_{br} =	$0.66\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\ \phi_c\sqrt{f'_c}\ c_{a1}^{1.5} R_{v,c}$	= 352.2	[kN]	D.7.2.3 (D-26)
Eccentricity effects	$\Psi_{ec,v}$ =	1.0 shear acts through center of group			D.7.2.5
Edge effects	$\Psi_{ed,v}$ =	$\min[(0.7+0.3c_2/1.5c_{a1}), 1.0]$	= 0.74		D.7.2.6
Concrete cracking	$\Psi_{c,v}$ =		= 1.20		D.7.2.7
Member thickness	$\Psi_{h,v}$ =	$\max[(\mathrm{sqrt}(1.5c_{a1}/h_a)), 1.0]$	= 1.50		D.7.2.8
Conc shear breakout resistance	V_{cbgr} =	$\frac{A_{Vc}}{A_{Vco}}\Psi_{ec,V}\ \Psi_{ed,V}\ \Psi_{c,V}\ \Psi_{h,V}\ V_{br}$	= 174.8	[kN]	D.7.2.1 (D-23)
Seismic design strength reduction	=	x 1.0 not applicable	= 174.8	[kN]	D.4.3.5
	ratio =	0.51	> V_u	**OK**	
Conc. Pryout Shear Resistance					
	k_{cp} =	2.0			D.7.3
Factored shear pryout resistance	V_{cpgr} =	$k_{cp} N_{cbgr}$	= 302.4	[kN]	D.7.3 (D-32)
	$R_{v,c}$ =	1.00 pryout strength is always Condition B			D.5.4(c)
Seismic design strength reduction	=	x 1.0 not applicable	= 302.4	[kN]	D.4.3.5
	ratio =	0.29	> V_u	**OK**	
					CSA S16-09
Stud on Conc Bearing	B_r =	$n_s \times 1.4 \times \phi_c \times \min(8d_a, h_{ef}) \times d_a \times f_c'$	= 334.4	[kN]	25.3.3.2
	ratio =	0.27	> V_u	**OK**	
Govern Shear Resistance	$\mathbf{V_r}$ =	$\min(V_{sr}, V_{cbgr}, V_{cpgr}, B_r)$	= **174.8**	[kN]	
					A23.3-04 (R2010)
Tension Shear Interaction					
Check if $N_u > 0.2\ N_r$ and $V_u > 0.2\ V_r$		Yes			D.8.2 & D.8.3
		$N_u/N_r + V_u/V_r$	= 1.20		D.8.4 (D-35)
	ratio =	1.00	> 1.2	**NG**	
Ductility Tension					
	N_{sr} =	118.2 [kN]			
	>	$\min(N_{cbgr}, N_{cpr}, N_{sbgr})$	= 91.9	[kN]	
		Non-ductile			
Ductility Shear					
	V_{sr} =	332.5 [kN]			
	>	$\min(V_{cbgr}, V_{cpgr}, B_r)$	= 174.8	[kN]	
		Non-ductile			

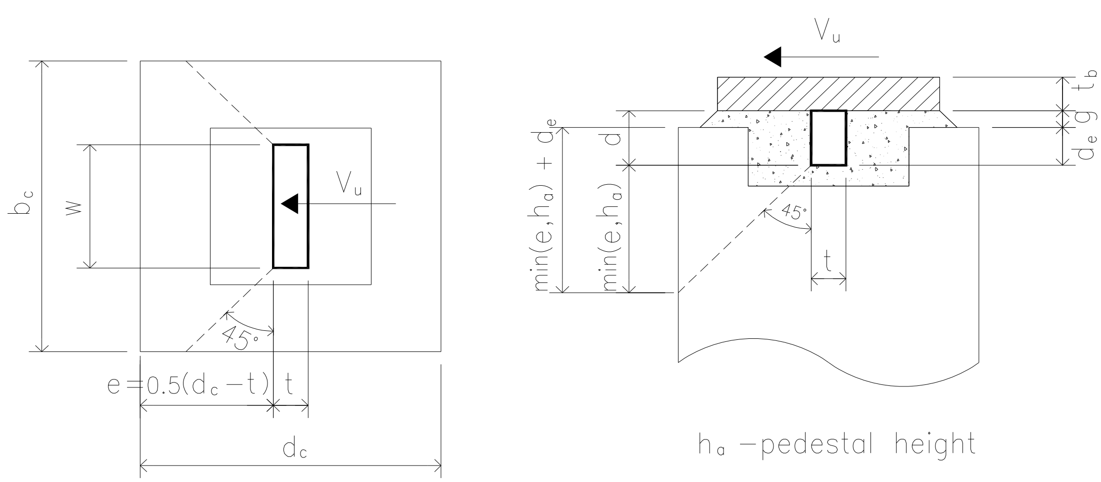
Example 41: Shear Lug Design ACI 349-06 Code

SHEAR LUG / SHEAR KEY DESIGN

Shear Lug / Shear Key design based on	**Code Abbreviation**
ACI 349-06 Code Requirements for Nuclear Safety-Related Concrete Structures & Commentary	*ACI 349-06*
AISC Design Guide 1: Base Plate and Anchor Rod Design - 2nd Edition	*AISC Design Guide 1*
AISC 360-05 Specification for Structural Steel Buildings	*AISC 360-05*

INPUT DATA **Code Reference**

Factored shear along strong axis	V_{ux}	=	75.0	[kips]					
Factored shear along weak axis	V_{uy}	=	50.0	[kips]	applicable for W Shape only				
Pedestal width	b_c	=	26.0	[in]					
Pedestal depth	d_c	=	26.0	[in]					
Pedestal height	h_a	=	30.0	[in]					
Grout thickness	g	=	2.0	[in]					
Shear key type		=	W_Shape		W8X40				
Shear key width Shape	w	=	8.07	[in]	Applicable				
Shear key width used for design	w	=	**8.07**	[in]					
Shear key embed depth	d	=	8.0	[in]					
					suggest				
Concrete strength	f'_c	=	4.5	[ksi]	4		= 31.0	[MPa]	
					A36	A992			
Shear key steel strength	F_y	=	50	[ksi]	36	50	= 344.8	[MPa]	
	F_u	=	65	[ksi]	58	65	= 448.2	[MPa]	
Weld electrode		=	E70XX						*AISC 360-05*
Electrode ultimate tensile	F_{EXX}	=	70	[ksi]	70		= 482.7	[MPa]	
Fillet weld leg size	A_m	=	5	[1/16 in]	5/16		= 7.9	[mm]	Table J2.4



Code Reference

CONCLUSION *ACI 349-06*

OVERALL ratio = **0.94** **OK**

Concrete Bearing ratio = 0.41 **OK** D.4.6.2

Shear Toward Free Edge ratio = 0.81 **OK** D.11.2

Shear Key Section Flexure & Shear Check ratio = 0.94 **OK**

Shear Key To Base Plate Fillet Weld ratio = 0.69 **OK**

CALCULATION

Concrete Bearing

A_b = $w\ d_e = w\ (d\text{-}g)$	= 48.42	[in^2]		
V_b = $1.3\ \phi\ f_c'\ A_b$	= 184.1	[kips]		D.4.6.2
ratio = 0.41	> V_{ux}		**OK**	
ϕ = 0.65 for anchor controlled by concrete bearing				D.4.4 (d)

Shear Toward Free Edge

e = $0.5x(d_c - t)$	= 12.38	[in]		
e = $\min(e\ ,\ h_a)$	= 12.38	[in]		
A_{eff} = $[\ e + (d\text{-}g)\ ]\ x\ b_c - wx(d\text{-}g)$	= 429.3	[in^2]		
ϕV_n = $4\phi\sqrt{f_c'}\ A_{eff}$	= 92.2	[kips]		D.11.2
ratio = 0.81	> V_u		**OK**	
ϕ = 0.80				D.4.4 (f)

Shear Key Section Flexure & Shear Check

Shear Key Plate Sect This case does not apply

THIS SECTION NOT APPLICABLE

	M_{ux} = $V_{ux}\ x\ [\ 0.5x(d\text{-}g) + g\ ]$	= 375.0	[kip-in]	
	Z = $w\ x\ t^2\ /\ 4$	= 3.15	[in^3]	
Flexure	$\phi\ M_n$ = $0.9\ x\ Z\ x\ F_y$	= 141.9	[kip-in]	
	ratio = 0.00	< M_{ux}		**OK**
Shear	$\phi\ V_n$ = $0.9\ x\ A_w\ x\ 0.6F_y$	= 272.4	[kips]	
	ratio = 0.00	> V_{ux}		**OK**

Shear Key Pipe Sect This case does not apply

THIS SECTION NOT APPLICABLE

	M_{ux} = $V_{ux}\ x\ [\ 0.5x(d\text{-}g) + g\ ]$	= 375.0	[kip-in]	
	Z =	= 0.00	[in^3]	
Flexure	$\phi\ M_n$ = $0.9\ x\ Z\ x\ F_y$	= 0.0	[kip-in]	
	ratio = 0.00	< M_{ux}		**OK**
Shear	A_w =	= 0.000	[in^2]	
	$\phi\ V_n$ = $0.9\ x\ A_w\ x\ 0.6F_y$	= 0.0	[kips]	
	ratio = 0.00	< V_{ux}		**OK**

Code Reference

Shear Key HSS Sect	This case does not apply			
	M_{ux} = V_{ux} x [0.5x(d-g) + g]	= 375.0	[kip-in]	
	Z =	= 0.00	[in^3]	
Flexure	ϕ M_n = 0.9 x Z x F_y	= 0.0	[kip-in]	
	ratio = 0.00	< M_{ux}	**OK**	
Shear	A_w =	= 0.000	[in^2]	
	ϕ V_n = 0.9 x A_w x 0.6F_y	= 0.0	[kips]	
	ratio = 0.00	< V_{ux}	**OK**	

THIS SECTION NOT APPLICABLE

Shear Key W Sect	**This case applies**			
Flexure strong axis	M_{ux} = V_{ux} x [0.5x(d-g) + g]	= 375.0	[kip-in]	
	Z_x =	= 39.80	[in^3]	
	ϕ M_{nx} = 0.9 x Z_x x F_y	= 1791.0	[kip-in]	
	ratio = 0.21	> M_{ux}	**OK**	
	M_{uy} = V_{uy} x [0.5x(d-g) + g]	= 250.0	[kip-in]	
	Z_y =	= 18.50	[in^3]	
Flexure weak axis	ϕ M_{ny} = 0.9 x Z_y x F_y	= 832.5	[kip-in]	
	ratio = 0.30	> M_{uy}	**OK**	
	b_f = 8.07 [in]	d = 8.25	[in]	
	t_w = 0.360 [in]	t_f = 0.560	[in]	
Shear strong axis	A_w = t_w x d	= 2.97	[in^2]	
	ϕ V_{nx} = 0.9 x A_w x 0.6F_y	= 80.2	[kips]	
	ratio = 0.94	> V_{ux}	**OK**	
Shear weak axis	A_w = 2 x t_f x b_f	= 9.04	[in^2]	
	ϕ V_{ny} = 0.9 x A_w x 0.6F_y	= 244.0	[kips]	
	ratio = 0.20	> V_{uy}	**OK**	

Shear Key To Base Plate Fillet Weld

Resultant angle	θ =	= 90	[deg]	*AISC 360-05*
Nominal fillet weld strength	F_w = 0.6 F_{EXX} (1.0+0.5 $\sin^{1.5}\theta$)	= 63.0	[kips]	Eq J2-5
	ϕ = 0.75			
Weld metal shear strength	ϕ r_{n1} = ϕ (0.707 x A_m) x F_w	= 10.44	[kips/in]	Eq J2-4
For PLATE shear key only	not applicable			
Base metal thickness	t =	= 0.000	[in]	
Base metal shear strength	ϕ r_{n2} = min [1.0(0.6F_y t) , 0.75(0.6F_u t)]	= 0.00	[kips/in]	Eq J4-3 & Eq J4-4
Shear strength used for design	ϕ r_n = min (ϕ r_{n1} , ϕ r_{n2})	= **10.44**	[kips/in]	Eq J2-2

Code Reference

Factored moment to base plate M_{ux} = V_{ux} x [0.5x(d-g) + g] = 375.0 [kip-in]

M_{uy} = V_{uy} x [0.5x(d-g) + g] = 250.0 [kip-in]

Shear Key Plate This case does not apply

s = t + (1/3)A_m x 2 = 1.458 [in]

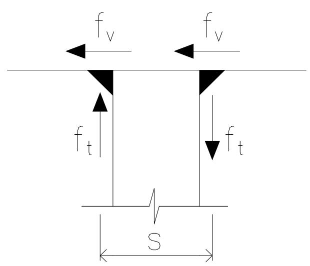
THIS SECTION NOT APPLICABLE

f_t = M_{ux} / (s x w) = 0.00 [kips/in]

f_v = V_{ux} / (w x 2) = 0.00 [kips/in]

f_r = $\sqrt{f_t^2 + f_v^2}$ = 0.00 [kips/in]

ratio = 0.00 < ϕ r_n **OK**



Force on Shear Key Plate Weld

Shear Key Pipe Sect This case does not apply

Weld ring diameter D = = 8.07 [in]

THIS SECTION NOT APPLICABLE

f_t = M_{ux} / (π D^2 /4) = 0.00 [kips/in]

f_v = V_{ux} / (π D x 1) = 0.00 [kips/in]

f_r = $\sqrt{f_c^2 + f_v^2}$ = 0.00 [kips/in]

ratio = 0.00 < ϕ r_n **OK**

Shear Key HSS Sect This case does not apply

Weld box width/depth b = 8.07 [in] d = 0.00 [in]

THIS SECTION NOT APPLICABLE

f_t = M_{ux} / (bd + d^2/3) = 0.00 [kips/in]

f_v = V_{ux} / (2xd) = 0.00 [kips/in]

f_r = $\sqrt{f_c^2 + f_v^2}$ = 0.00 [kips/in]

ratio = 0.00 < ϕ r_n **OK**

Shear Key W Sect **This case applies**

b = 8.07 [in] d = 8.25 [in]

Strong Axis f_t = M_{ux} / (bxd) = 5.63 [kips/in]

f_v = V_{ux} / (2xd) = 4.55 [kips/in]

f_r = $\sqrt{f_c^2 + f_v^2}$ = 7.24 [kips/in]

ratio = 0.69 < ϕ r_n **OK**

Weak Axis f_t = M_{uy} / [(1xb^2/6) x 4] = 5.76 [kips/in]

f_v = V_{uy} / (4xb) = 1.55 [kips/in]

f_r = $\sqrt{f_c^2 + f_v^2}$ = 5.96 [kips/in]

ratio = 0.57 < ϕ r_n **OK**

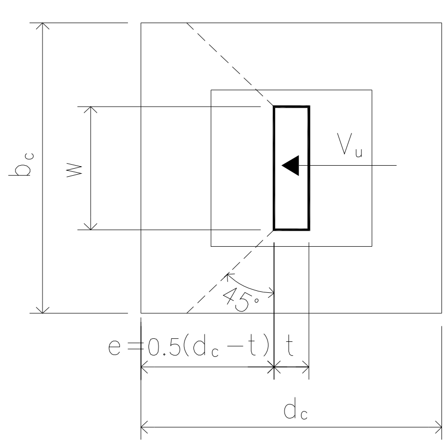
Example 42: Shear Lug Design ACI 349M-06 Code

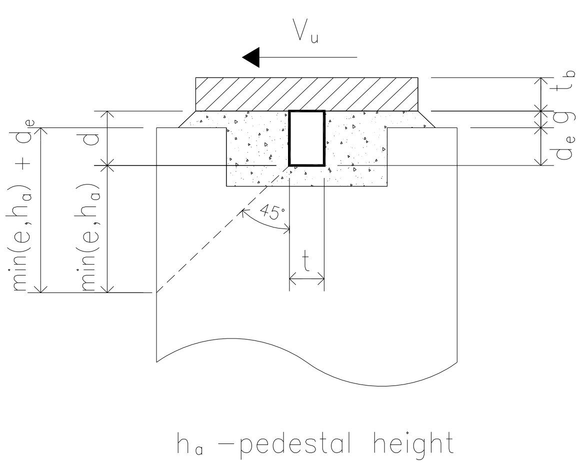
SHEAR LUG / SHEAR KEY DESIGN

Shear Lug / Shear Key design based on	**Code Abbreviation**
ACI 349M-06 Metric Code Requirements for Nuclear Safety-Related Concrete Structures & Commentary	*ACI 349M-06*
AISC Design Guide 1: Base Plate and Anchor Rod Design - 2nd Edition	*AISC Design Guide 1*
CSA S16-09 Design of Steel Structures	*CSA S16-09*

INPUT DATA **Code Reference**

Factored shear along strong axis	V_{ux} =	333.6	[kN]				
Factored shear along weak axis	V_{uy} =	222.4	[kN]	applicable for W Shape only			
Pedestal width	b_c =	660	[mm]				
Pedestal depth	d_c =	660	[mm]				
Pedestal height	h_a =	762	[mm]				
Grout thickness	g =	51	[mm]				
Shear key type	=	W_Shape		W200x59			
Shear key width Shape	w =	205	[mm]	Applicable			
Shear key width used for design	w =	**205**	[mm]				
Shear key embed depth	d =	203	[mm]				
				suggest			
Concrete strength	f'_c =	31	[MPa]	30	=	4.5	[ksi]
Shear key steel strength	F_y =	345	[MPa]	300	=	50.0	[ksi]
	F_u =	448	[MPa]	450	=	65.0	[ksi]
Weld electrode	=	E49XX			X_u =	490	[MPa]
Fillet weld leg size	D =	8	[mm]				





Code Reference

CONCLUSION *ACI 349M-06*

OVERALL	ratio = **0.94**	**OK**	
Concrete Bearing	ratio = 0.41	**OK**	D.4.6.2
Shear Toward Free Edge	ratio = 0.81	**OK**	D.11.2
Shear Key Section Flexure & Shear Check	ratio = 0.94	**OK**	
Shear Key To Base Plate Fillet Weld	ratio = 0.79	**OK**	

CALCULATION

Concrete Bearing

A_b = $w\, d_e = w\,(d-g)$	= 31242	[mm^2]		
V_b = $1.3\,\phi\, f_c'\, A_b$	= 818.4	[kN]		D.4.6.2
ratio = 0.41	> V_{ux}		**OK**	
ϕ = 0.65 for anchor controlled by concrete bearing				D.4.4 (d)

Shear Toward Free Edge

e = $0.5x(d_c - t)$	= 314	[mm]		
e = $\min(e\,,\, h_a)$	= 314	[mm]		
A_{eff} = $[\, e + (d-g)\,] \times b_c - wx(d-g)$	= 2.8E+05	[mm^2]		
ϕV_n = $4\phi\sqrt{f_c'}\ A_{eff}$	= 409.6	[kN]		D.11.2
ratio = 0.81	> V_u		**OK**	
ϕ = 0.80				D.4.4 (f)

Shear Key Section Flexure & Shear Check

Shear Key Plate Sect This case does not apply

THIS SECTION NOT APPLICABLE

	M_{ux} = $V_{ux} \times [\, 0.5x(d-g) + g\,]$	= 42.4	[kNm]	
	Z = $w \times t^2 / 4$	= 52.5	[$x10^3 mm^3$]	
Flexure	ϕM_n = $0.9 \times Z \times F_y$	= 16.3	[kNm]	
	ratio = 0.00	< M_{ux}		**OK**
Shear	ϕV_n = $0.9 \times A_w \times 0.6F_y$	= 1222.1	[kN]	
	ratio = 0.00	> V_{ux}		**OK**

Shear Key Pipe Sect This case does not apply

THIS SECTION NOT APPLICABLE

	M_{ux} = $V_{ux} \times [\, 0.5x(d-g) + g\,]$	= 42.4	[kNm]	
	Z =	= 0.0	[$x10^3 mm^3$]	
Flexure	ϕM_n = $0.9 \times Z \times F_y$	= 0.0	[kNm]	
	ratio = 0.00	< M_{ux}		**OK**
Shear	A_w =	= 0	[mm^2]	
	ϕV_n = $0.9 \times A_w \times 0.6F_y$	= 0.0	[kN]	
	ratio = 0.00	< V_{ux}		**OK**

				Code Reference
Shear Key HSS Sect	This case does not apply			
	M_{ux} = V_{ux} x [0.5x(d-g) + g]	= 42.4	[kNm]	
	Z =	= 0.0	[x10^{3}mm^3]	
Flexure	ϕ M_n = 0.9 x Z x F_y	= 0.0	[kNm]	
	ratio = 0.00	< M_{ux}	**OK**	
Shear	A_w =	= 0	[mm^2]	
	ϕ V_n = 0.9 x A_w x 0.6F_y	= 0.0	[kN]	
	ratio = 0.00	< V_{ux}	**OK**	

THIS SECTION NOT APPLICABLE

Shear Key W Sect	**This case applies**			
Flexure strong axis	M_{ux} = V_{ux} x [0.5x(d-g) + g]	= 42.4	[kNm]	
	Z_x =	= 653	[x10^{3}mm^3]	
	ϕ M_{nx} = 0.9 x Z_x x F_y	= 202.8	[kNm]	
	ratio = 0.21	> M_{ux}	**OK**	
	M_{uy} = V_{uy} x [0.5x(d-g) + g]	= 28.2	[kNm]	
	Z_y =	= 303	[x10^{3}mm^3]	
Flexure weak axis	ϕ M_{ny} = 0.9 x Z_y x F_y	= 94.1	[kNm]	
	ratio = 0.30	> M_{uy}	**OK**	
	b_f = 205.0 [mm]	d = 210.0	[mm]	
	t_w = 9.1 [mm]	t_f = 14.2	[mm]	
Shear strong axis	A_w = t_w x d	= 1911	[mm^2]	
	ϕ V_{nx} = 0.9 x A_w x 0.6F_y	= 356.0	[kN]	
	ratio = 0.94	> V_{ux}	**OK**	
Shear weak axis	A_w = 2 x t_f x b_f	= 5822	[mm^2]	
	ϕ V_{ny} = 0.9 x A_w x 0.6F_y	= 1084.6	[kN]	
	ratio = 0.21	> V_{uy}	**OK**	

Shear Key To Base Plate Fillet Weld

Base metal resistance	A_m = D x 1mm	= 8.00	[mm^2]	
	v_{rm} = 0.67 ϕ_w A_m F_u	= 1.61	[kN/mm]	13.13.2.2
	ϕ_w = 0.67			13.1 (h)
Weld metal resistance	A_w = 0.707 x D x 1mm	= 5.66	[mm^2]	
Fillet weld resistance - shear	θ =	= 90		
	v_{rw} = 0.67 ϕ_w A_w X_u (1+ 0.5 sinθ ^1.5)	= 1.87	[kN/mm]	13.13.2.2
	v_r = min(v_{rm} , v_{rw})	= **1.61**	[kN/mm]	

Code Reference

Factored moment to base plate $M_{ux} = V_{ux} \times [\, 0.5x(d\text{-}g) + g \,]$ = 42.4 [kNm]

$M_{uy} = V_{uy} \times [\, 0.5x(d\text{-}g) + g \,]$ = 28.2 [kNm]

Shear Key Plate This case does not apply

$s = t + (1/3)D \times 2$ = 37.3 [mm]

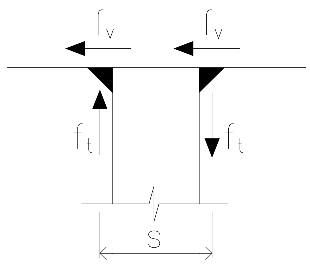
THIS SECTION NOT APPLICABLE

$f_t = M_{ux} / (s \times w)$ = 0.00 [kN/mm]

$f_v = V_{ux} / (w \times 2)$ = 0.00 [kN/mm]

$f_r = \sqrt{f_t^2 + f_v^2}$ = 0.00 [kN/mm]

ratio = 0.00 < $\phi\, r_n$ **OK**



Force on Shear Key Plate Weld

Shear Key Pipe Sect This case does not apply

Weld ring diameter $D =$ = 205.0 [mm]

THIS SECTION NOT APPLICABLE

$f_t = M_{ux} / (\pi\, D^2 /4)$ = 0.00 [kN/mm]

$f_v = V_{ux} / (\pi\, D \times 1)$ = 0.00 [kN/mm]

$f_r = \sqrt{f_c^2 + f_v^2}$ = 0.00 [kN/mm]

ratio = 0.00 < $\phi\, r_n$ **OK**

Shear Key HSS Sect This case does not apply

Weld box width/depth b = 205.0 [in] d = 205.0 [mm]

THIS SECTION NOT APPLICABLE

$f_t = M_{ux} / (bd + d^2/3)$ = 0.00 [kN/mm]

$f_v = V_{ux} / (2xd)$ = 0.00 [kN/mm]

$f_r = \sqrt{f_c^2 + f_v^2}$ = 0.00 [kN/mm]

ratio = 0.00 < $\phi\, r_n$ **OK**

Shear Key W Sect **This case applies**

b = 205.0 [in] d = 210.0 [mm]

Strong Axis

$f_t = M_{ux} / (bxd)$ = 0.98 [kN/mm]

$f_v = V_{ux} / (2xd)$ = 0.79 [kN/mm]

$f_r = \sqrt{f_c^2 + f_v^2}$ = 1.26 [kN/mm]

ratio = 0.79 < $\phi\, r_n$ **OK**

Weak Axis

$f_t = M_{uy} / [(1xb^2/6) \times 4]$ = 1.01 [kN/mm]

$f_v = V_{uy} / (4xb)$ = 0.27 [kN/mm]

$f_r = \sqrt{f_c^2 + f_v^2}$ = 1.04 [kN/mm]

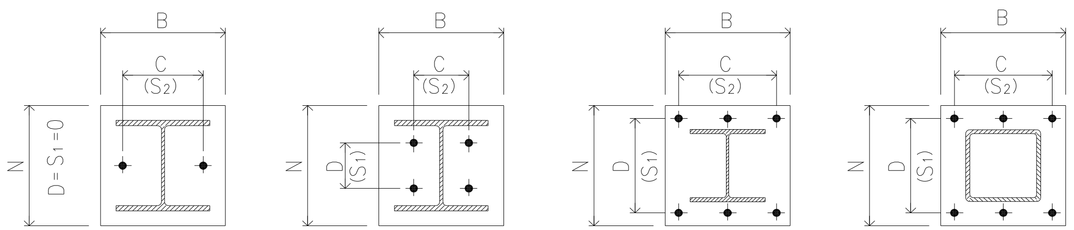
ratio = 0.65 < $\phi\, r_n$ **OK**

Example 51: Base Plate (LRFD) & Anchor Bolt (ACI 318-08) Design With Anchor Reinforcement

BASE PLATE & ANCHOR BOLT DESIGN - MOMENT CONNECTION

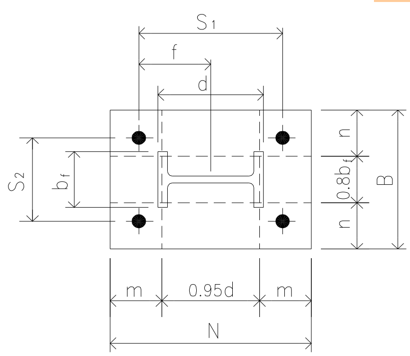
Base Plate Data

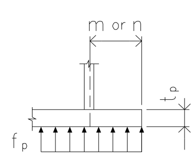
Column section type		=	W_Shape				
Column size		=	W14X53				
Depth	d	=	13.900	[in]	Flange thickness t_f	= 0.660	[in]
Flange width	b_f	=	8.060	[in]	Web thickness t_w	= 0.370	[in]
Base plate anchor bolt pattern		=	4 or 6-Bolt MC WF		? base plate is moment connection		
Base plate anchor bolt location		=	Bolt Outside Flange Only		?		



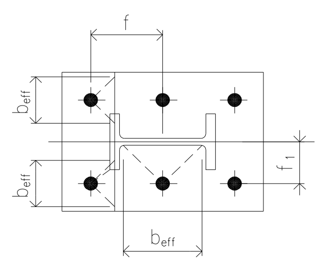
2-BOLT PIN 4-BOLT PIN 4 or 6-Bolt MC WF 4 or 6-Bolt MC HS

					suggest
Base plate width	B	=	22.0	[in]	15.0
Base plate depth	N	=	22.0	[in]	21.0
Base plate thickness	t_p	=	2.00	[in]	1.75
Anchor bolt spacing s_2 = C	C	=	18.0	[in]	11.0
Anchor bolt spacing s_1 = D	D	=	18.0	[in]	17.0





BASE PLATE GEOMETRIC



BASE PLATE SUBJECT TO TENSILE LOAD

Bolt to column center dist.	f	=	9.0	[in]
Bolt to column web center dist.	f_1	=	9.0	[in]

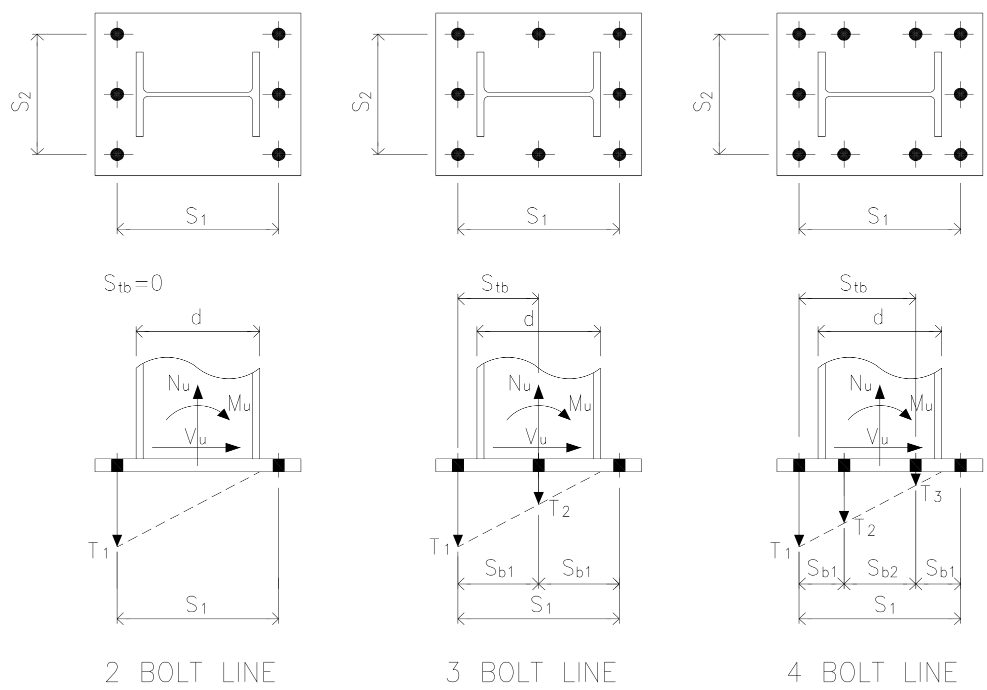
Suggested plate thickness for rigidity: t_p = max. of m/4 and n/4 = No ?

Factored column load

LCB	Cases	P_u [kips]	V_u [kips]	M_u [kip-ft]
LCB1	Axial Comp.	100.0	15.0	0.0
LCB2	Axial Comp. + M	0.0	20.0	30.0
LCB3	Axial Comp. + M	15.0	20.0	30.0
LCB4	Axial Tensile	10.0	35.0	0.0

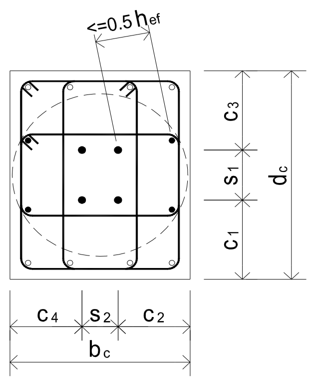
Code Reference

Anchor Bolt Data

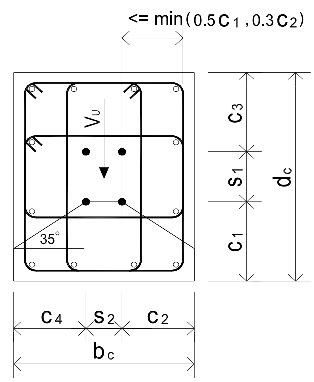


No of bolt line for resisting moment		=	3 Bolt Line					
No of bolt along outermost bolt line		=	3					
					min required			*PIP STE05121*
Outermost bolt line spacing s_1	s_1	=	18.0	[in]	3.5	OK		Page A -1 Table 1
Outermost bolt line spacing s_2	s_2	=	18.0	[in]	3.5	OK		
Internal bolt line spacing s_{b1}	s_{b1}	=	9.0	[in]	3.5	OK		
Internal bolt line spacing s_{b2}	s_{b2}	=	0.0	[in]	3.5	OK		
Anchor bolt material		=	F1554 Grade 55					
Anchor tensile strength	f_{uta}	=	75.0	[ksi]		= 517	[MPa]	*ACI 318-08*
			Anchor is ductile steel element					D.1
Anchor bolt diameter	d_a	=	**0.875**	[in]	max 1.5 in	= 22.2	[mm]	*PIP STE05121*
Bolt sleeve diameter	d_s	=	2.0	[in]				Page A -1 Table 1
Bolt sleeve height	h_s	=	7.0	[in]				
					min required			
Anchor bolt embedment depth	h_{ef}	=	20.0	[in]	10.5	OK		Page A -1 Table 1
Pedestal height	h_a	=	23.0	[in]	23.0	OK		
Pedestal width	b_c	=	124.0	[in]				
Pedestal depth	d_c	=	124.0	[in]				

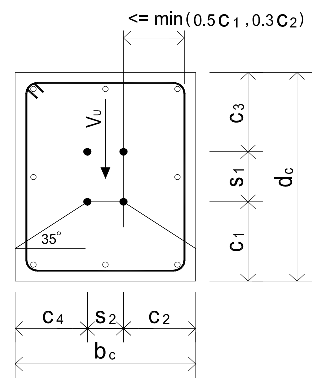
						Code Reference
Bolt edge distance c_1	c_1 = 6.0	[in]	5.3		OK	*PIP STE05121*
Bolt edge distance c_2	c_2 = 6.0	[in]	5.3		OK	Page A -1 Table 1
Bolt edge distance c_3	c_3 = 100.0	[in]	5.3		OK	
Bolt edge distance c_4	c_4 = 100.0	[in]	5.3		OK	



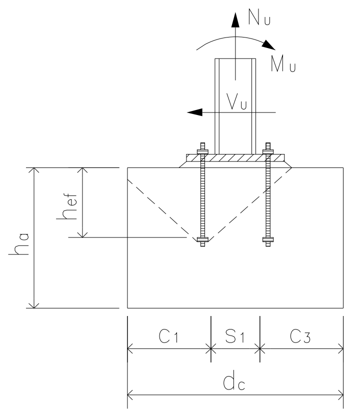
Ver. Reinft For Tension

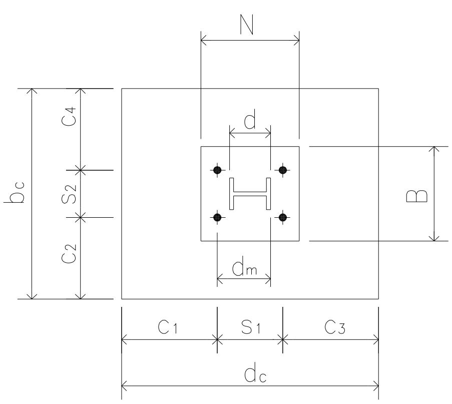


Hor. Ties For Shear - 4 Legs



Hor. Ties For Shear - 2 Legs





ACI 318-08

To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within $0.5h_{ef}$ from the outmost anchor's centerline. In this design $0.5h_{ef}$ value is limited to 8 in. — RD.5.2.9

$0.5h_{ef}$ = **8.0** [in]

No of ver. rebar that are effective for resisting anchor tension n_v = 6

Ver. bar size No. 8 1.000 [in] dia single bar area A_s = 0.79 [in^2]

To be considered effective for resisting anchor shear, hor. reinft shall be located within min($0.5c_1$, $0.3c_2$) from the outmost anchor's centerline — RD.6.2.9

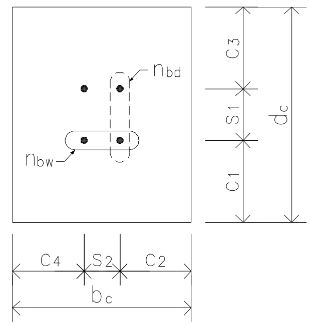
min($0.5c_1$, $0.3c_2$) = **1.8** [in]

No of tie leg that are effective to resist anchor shear n_{leg} = 2 ?

No of tie layer that are effective to resist anchor shear n_{lay} = 2 ?

Hor. tie bar size No. 4 0.500 [in] dia single bar area A_s = 0.20 [in^2]

For anchor reinft shear breakout strength calc 100% hor. tie bars develop full yield strength ?



Bolt No Input for Side-Face Blowout Check Use

				suggest
Concrete strength	f'_c =	4.5	[ksi]	4
Rebar yield strength	f_y =	60.0	[ksi]	60
Base plate yield strength	F_y =	36.0	[ksi]	36
Total no of anchor bolt	n =	8		
No of anchor bolt carrying shear	n_s =	8		
For side-face blowout check use				
No of bolt along width edge	n_{bw} =	3		
No of bolt along depth edge	n_{bd} =	3		
Anchor head type	=	Heavy Hex		?
Anchor effective cross sect area	A_{se} =	0.462	[in^2]	
Bearing area of one head	A_{brg} =	**1.188**	[in^2]	
	A_{brg}		[in^2]	not applicable

				Code Reference
Bolt 1/8" (3mm) corrosion allowance	=	No	?	
Provide shear key ?	=	No	?	*ACI 318-08*
Seismic design category >= C	=	No	?	D.3.3.3
Provide built-up grout pad ?	=	Yes	?	D.6.1.3

CONCLUSION

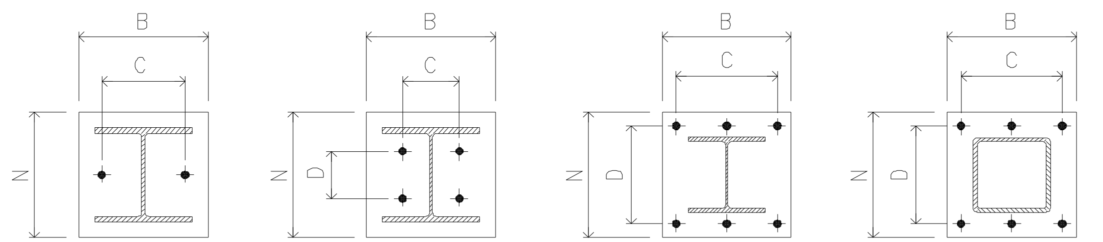
OVERALL	ratio = **0.97**	**OK**
BASE PLATE		
Base Plate Size and Anchor Bolt Tensile		**OK**
Base Plate Thickness	ratio = **0.52**	**OK**
ANCHOR BOLT		
LCB1 Axial Compression		
Abchor Rod Embedment, Spacing and Edge Distance		**OK**
Min Rquired Anchor Reinft. Development Length	ratio = 0.97	**OK**
Overall Ratio	ratio = **0.42**	**OK**
LCB2 Axial Compression + Moment		
Abchor Rod Embedment, Spacing and Edge Distance		**OK**
Min Rquired Anchor Reinft. Development Length	ratio = 0.97	**OK**
Overall Ratio	ratio = **0.83**	**OK**
LCB3 Axial Compression + Moment		
Abchor Rod Embedment, Spacing and Edge Distance		**OK**
Min Rquired Anchor Reinft. Development Length	ratio = 0.97	**OK**
Overall Ratio	ratio = **0.72**	**OK**
LCB4 Axial Tensile		
Abchor Rod Embedment, Spacing and Edge Distance		**OK**
Min Rquired Anchor Reinft. Development Length	ratio = 0.97	**OK**
Overall Ratio	ratio = **0.97**	**OK**

BASE PLATE DESIGN

Base plate design based on	**Code Abbreviation**
AISC Design Guide 1: Base Plate and Anchor Rod Design 2nd Edition	*AISC Design Guide 1*
ACI 318-08 Building Code Requirements for Structural Concrete and Commentary	*ACI 318-08*

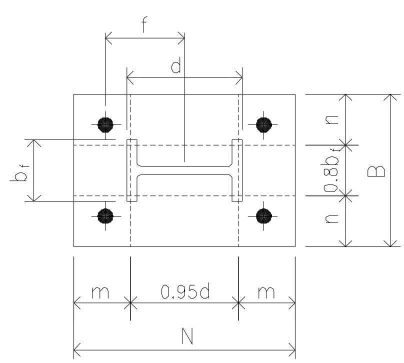
DESIGN DATA

Column section type		W_Shape					
Column size		W14X53					
Depth	d =	13.900	[in]	Flange thickness t_f	=	0.660	[in]
Flange width	b_f =	8.060	[in]	Web thickness t_w	=	0.370	[in]
Base plate anchor bolt pattern		4 or 6-Bolt MC WF		base plate is moment connection			

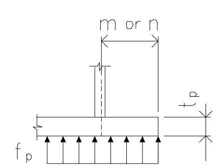


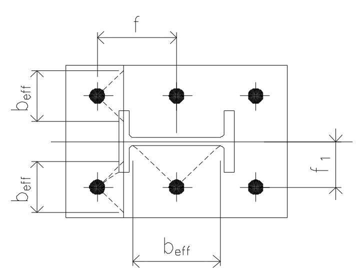
2-BOLT PIN 4-BOLT PIN 4 or 6-Bolt MC WF 4 or 6-Bolt MC HS

				suggest
Base plate width	B =	22.0	[in]	15.0
Base plate depth	N =	22.0	[in]	21.0
Base plate thickness	t_p =	2.000	[in]	1.8
Anchor bolt spacing	C =	18.0	[in]	11.0
Anchor bolt spacing	D =	18.0	[in]	17.0
Anchor bolt diameter	d =	0.875	[in]	max 1.5 in



BASE PLATE GEOMETRIC





BASE PLATE SUBJECT TO TENSILE LOAD

				suggest
Bolt to column center dist.	f =	9.0	[in]	9 in
Bolt to column web center dist.	f_1 =	9.0	[in]	9 in
Pedestal width	b_c =	124.0	[in]	>= 28.5 in
Pedestal depth	d_c =	124.0	[in]	>= 28.5 in

Factored column load

LCB	Cases	P_u [kips]	M_u [kip-ft]	t_p (in)	Base Plate Size
LCB1	Axial Compressive	100.0	0.0	0.88	**Base Plate B x N OK**
LCB2	Compression + M	0.0	30.0	0.89	**Base Plate B x N OK**
LCB3	Compression + M	15.0	30.0	1.04	**Base Plate B x N OK**
LCB4	Axial Tensile	10.0	0.0	0.28	**Anchor Bolt Tensile OK**
Min required plate thickness				**1.04**	

suggest max plate thickness 1.75 in

Suggested plate thickness for rigidity: t_p = max. of m/4 and n/4 = No

For base plate subject to tensile force only

Total No of anchor bolt n = 8

Bolt pattern Bolt Outside Flange Only

For base plate subject to large moment

No of bolt resisting tensile force n_t = 5

Anchor rod material F1554 Grade 55

Anchor rod tensile strength f_{uta} = 75.0 [ksi]

Bolt 1/8" (3mm) corrosion allowance No

Anchor rod effective area A_{se} = 0.462 [in^2]

Concrete strength f'_c = 4.5 [ksi]

Base plate yield strength F_y = 36.0 [ksi]

Strength reduction factor *ACI 318-08*

Bearing on concrete ϕ_c = 0.65 9.3.2.4

Base plate bending ϕ_b = 0.90

CONCLUSION

[Base Plate Size and Anchor Bolt Tensile Is Adequate] **OK**

[The Base Plate Thickness Is Adequate] ratio= **0.52**

DESIGN CHECK

				Code Reference
For base plate subject to large moment				*ACI 318-08*
Anchor rod tensile resistance	$T_r = \phi_{t,s}\, n_t\, A_{se}\, f_{uta}$	= 129.9	[kips]	D.5.1.2 (D-3)
	$\phi_{t,s}$ = 0.75 for ductile steel element			D.4.4 (a)
				AISC Design Guide 1
W Shapes	m = (N - 0.95d) / 2	= **4.40**	[in]	
	n = (B -0.8b_f) / 2	= **7.78**	[in]	3.1.2 on Page 15
HSS Rectangle Shapes	m = (N - 0.95d) / 2	= 4.40	[in]	3.1.3 on Page 16
	n = (B -0.95b_f) / 2	= 7.17	[in]	
HSS Round Shapes	m = (N - 0.8d) / 2	= 5.44	[in]	3.1.3 on Page 16
	n = (B -0.8b_f) / 2	= 5.44	[in]	
m value used for design	m =	= **4.40**	[in]	
n value used for design	n =	= **7.78**	[in]	
Suggested plate thickness for rigidity: t_p = max. of m/4 and n/4		= **1.94**	[in]	
Base plate area	A_1 = B x N	= 484.0	[in^2]	
Pedestal area	$A_2 = b_c \times d_c$	= 15376.0	[in^2]	
				ACI 318-08
	k = min [sqrt(A_2/A_1), 2]	= 2.000		10.14.1
	$\phi_c P_n = \phi_c\, 0.85\, f_c'\, A_1\, k$	= 2406.7	[kips]	10.14.1
		> P_u	**OK**	
LCB1: Axial Compressive				*AISC Design Guide 1*
	$X = \frac{4db_f}{(d+b_f)^2}\frac{P_u}{\phi_c P_p}$	= 0.039		3.1.2 on Page 16
	$\lambda = \min\left(\frac{2\sqrt{X}}{1+\sqrt{1-X}}, 1\right)$	= 0.2		
	$\lambda n' = \lambda$ sqrt(d x b_f) / 4	= 0.53	[in]	
For W shape	L = max (m, n, λn')	= 7.78	[in]	3.1.2 on Page 15
For HSS and Pipe	L = max (m, n)	= 7.78	[in]	3.1.3 on Page 16
L value used for design	L =	**7.78**	[in]	
	$t_p = L\sqrt{\frac{2\,P_u}{\phi_b\,F_y\,B\,N}}$	= **0.88**	[in]	

THIS SECTION NOT APPLICABLE

THIS SECTION NOT APPLICABLE

Base Plate B x N OK

LCB2: Axial Compression + Moment **Code Reference**

P_u = 0.1 [kips] M_u = 30.0 [kip-ft]

$e = M_u / P_u$ = 3600.00 [in]

$f_{p(max)} = \phi_c\ 0.85\ f_c'\ k$ = 4.97 [ksi]

$q_{max} = f_{p(max)} \times B$ = 109.40 [kips/in]

$e_{crit} = N/2 - P_u / (2q_{max})$ = 11.00 [in]

$e > e_{crit}$ Large moment case applied

Small moment case **This case does not apply** *AISC Design Guide 1*

Bearing length $Y = N - 2e$ = 0.00 [in]

Verify linear bearing pressure $q = P_u / Y$ = 0.00 [kips/in]

$< q_{max}$ **OK**

THIS SECTION NOT APPLICABLE

$f_p = P_u / BY$ = 0.00 [ksi]

$m = \max(m , n)$ = 7.78 [in]

If Y>=m $t_{req1} = 1.49m\ \mathrm{sqrt}(f_p / F_y)$ = 0.00 [in] Eq. 3.3.14a-1

If Y<m $t_{req2} = 2.11\sqrt{\dfrac{f_p Y\left(m - \dfrac{Y}{2}\right)}{F_y}}$ = 0.00 [in] Eq. 3.3.15a-1

$t_{min} = \max(t_{req1}, t_{req2})$ = **0.00** [in]

Large moment case **This case applies**

Check if real solution of Y exist $var_1 = (f + N/2)^2$ = 400 [in^2]

$var_2 = 2P_u (e+f) / q_{max}$ = 7 [in^2]

$var_1 > var_2$ **OK**

Bearing length $Y = \left(f + \dfrac{N}{2}\right) \pm \sqrt{\left(f + \dfrac{N}{2}\right)^2 - \dfrac{2P_u(e+f)}{q_{max}}}$ = 0.17 [in] Eq. 3.4.3

Anchor rod tension force $T_u = q_{max}Y - P_u$ = 18.0 [kips] Eq. 3.4.2

ratio = 0.14 $< T_r$ **OK**

At anchor rod tension interface

$x = f - d/2 + t_f / 2$ = 2.38 [in] Eq. 3.4.6

$t_{req\text{-}t} = 2.11\sqrt{\dfrac{T_u\ x}{BF_y}}$ = 0.49 [in] Eq. 3.4.7a

At conc. bearing interface

$m = \max(m , n)$ = 7.78 [in]

If Y>=m $t_{req\text{-}b} = 1.49m\ \mathrm{sqrt}(f_{p(max)} / F_y)$ = 0.00 [in] Eq. 3.3.14a-2

If Y<m $t_{req\text{-}b} = 2.11\sqrt{\dfrac{f_{p(max)} Y\left(m - \dfrac{Y}{2}\right)}{F_y}}$ = 0.89 [in] Eq. 3.3.15a-2

$t_{min} = \max(t_{req\text{-}t}, t_{req\text{-}b})$ = **0.89** [in]

Base Plate B x N OK

LCB3: Axial Compression + Moment

Code Reference

P_u = 15.0 [kips] M_u = 30.0 [kip-ft]

$e = M_u / P_u$ = 24.00 [in]

$f_{p(max)} = \phi_c\, 0.85\, f_c'\, k$ = 4.97 [ksi]

$q_{max} = f_{p(max)} \times B$ = 109.40 [kips/in]

$e_{crit} = N/2 - P_u / (2q_{max})$ = 10.93 [in]

$e > e_{crit}$ Large moment case applied

Small moment case **This case does not apply** *AISC Design Guide 1*

THIS SECTION NOT APPLICABLE

Bearing length	$Y = N - 2e$	= 0.00	[in]	
Verify linear bearing pressure	$q = P_u / Y$	= 0.00	[kips/in]	
		$< q_{max}$	OK	
	$f_p = P_u / BY$	= 0.00	[ksi]	
	$m = \max(m, n)$	= 7.78	[in]	
If Y>=m	$t_{req1} = 1.49m \sqrt{f_p / F_y}$	= 0.00	[in]	Eq. 3.3.14a-1
If Y<m	$t_{req2} = 2.11\sqrt{\dfrac{f_p Y\left(m - \frac{Y}{2}\right)}{F_y}}$	= 0.00	[in]	Eq. 3.3.15a-1
	$t_{min} = \max(t_{req1}, t_{req2})$	= **0.00**	[in]	

Large moment case **This case applies**

Check if real solution of Y exist	$var_1 = (f + N/2)^2$	= 400	[in^2]	
	$var_2 = 2P_u (e+f) / q_{max}$	= 9	[in^2]	
	$var_1 > var_2$		OK	
Bearing length	$Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_u(e+f)}{q_{max}}}$	= 0.23	[in]	Eq. 3.4.3
Anchor rod tension force	$T_u = q_{max} Y - P_u$	= 9.9	[kips]	Eq. 3.4.2
	ratio = 0.08	$< T_r$	OK	
At anchor rod tension interface				
	$x = f - d/2 + t_f / 2$	= 2.38	[in]	Eq. 3.4.6
	$t_{req\text{-}t} = 2.11\sqrt{\dfrac{T_u\, x}{B F_y}}$	= 0.36	[in]	Eq. 3.4.7a
At conc. bearing interface				
	$m = \max(m, n)$	= 7.78	[in]	
If Y>=m	$t_{req\text{-}b} = 1.49m \sqrt{f_{p(max)} / F_y}$	= 0.00	[in]	Eq. 3.3.14a-2
If Y<m	$t_{req\text{-}b} = 2.11\sqrt{\dfrac{f_{p(max)} Y\left(m - \frac{Y}{2}\right)}{F_y}}$	= 1.04	[in]	Eq. 3.3.15a-2
	$t_{min} = \max(t_{req\text{-}t}, t_{req\text{-}b})$	= **1.04**	[in]	

Base Plate B x N OK

LCB4: Axial Tensile

Factored tensile load P_u = = 10.0 [kips]

For base plate subject to tensile force only *ACI 318-08*

Anchor rod tensile resistance $T_r = \phi_{t,s}\, n\, A_{se}\, f_{uta}$ = 207.9 [kips] D.5.1.2 (D-3)

$\phi_{t,s}$ = 0.75 for ductile steel element D.4.4 (a)

ratio = 0.05 > P_u **OK**

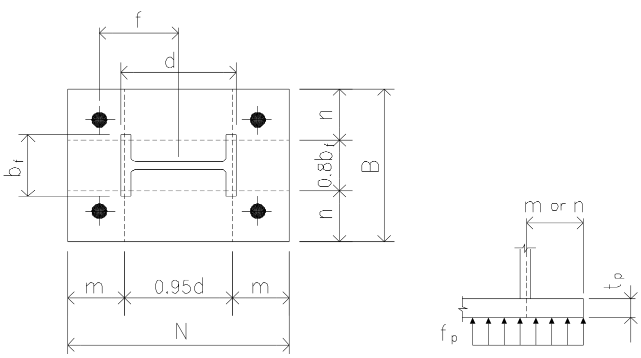
Bolt pattern Bolt Outside Flange Only

Total No of anchor bolt n = 8

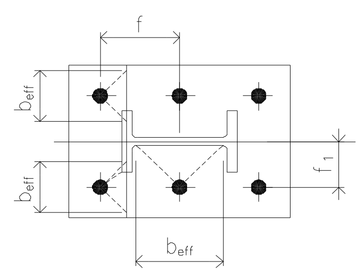
Bolt to column center dist. f = 9.0 [in]

Bolt to column web center dist. f_1 = 9.0 [in]

Each bolt factored tensile load T_u = 1.3 [kips]



BASE PLATE GEOMETRIC



BASE PLATE SUBJECT TO TENSILE LOAD

Bending to Column Flange

Moment lever arm a = 2.38 [in]

Moment to column flange M_u = 0.25 [kip-ft]

Effective plate width b_{eff} = 2 x a = 4.76 [in]

Base plate required thickness $t_{p1} = \sqrt{\dfrac{4\, M_u}{b_{eff}\, \phi_b\, F_y}}$ = 0.28 [in]

Bending to Column Web

Moment lever arm a = 8.82 [in]

Moment to column flange M_u = 0.92 [kip-ft]

Effective plate width b_{eff} = 2 x a = 17.63 [in]

Base plate required thickness $t_{p2} = \sqrt{\dfrac{4\, M_u}{b_{eff}\, \phi_b\, F_y}}$ = 0.00 [in]

t_{min} = max (t_{p1}, t_{p2}) = **0.28** [in]

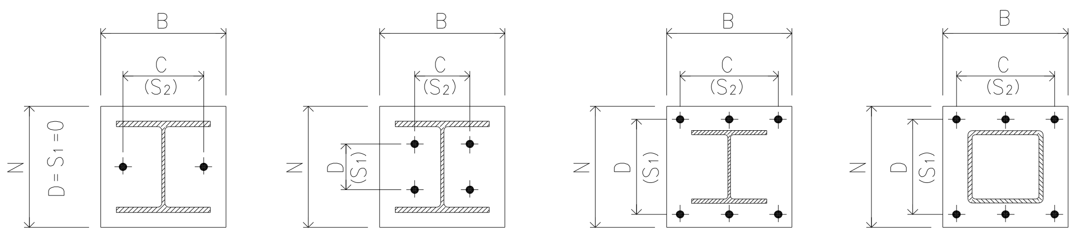
Anchor Bolt Tensile OK

Example 52: Base Plate (S16-09) & Anchor Bolt (CSA A23.3-04) Design With Anchor Reinforcement

BASE PLATE & ANCHOR BOLT DESIGN - MOMENT CONNECTION

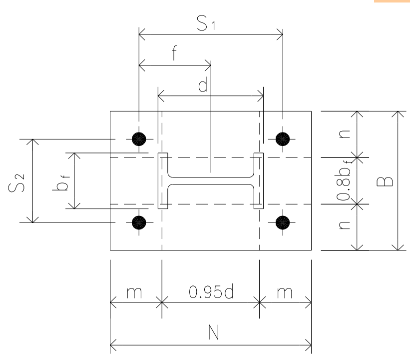
Base Plate Data

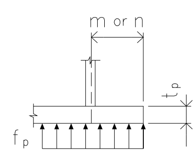
Column section type	=	W_Shape				
Column size	=	W360x79				
Depth	d =	354.0	[mm]	Flange thickness t_f	= 16.8	[mm]
Flange width	b_f =	205.0	[mm]	Web thickness t_w	= 9.4	[mm]
Base plate anchor bolt pattern	=	4 or 6-Bolt MC WF		? base plate is moment connection		
Base plate anchor bolt location	=	Bolt Outside Flange Only		?		



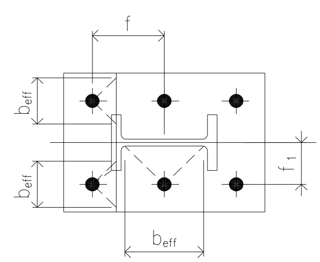
2-BOLT PIN 4-BOLT PIN 4 or 6-Bolt MC WF 4 or 6-Bolt MC HS

				suggest
Base plate width	B =	559	[mm]	380
Base plate depth	N =	559	[mm]	530
Base plate thickness	t_p =	51	[mm]	45
Anchor bolt spacing s_2 = C	C =	457	[mm]	280
Anchor bolt spacing s_1 = D	D =	457	[mm]	430





BASE PLATE GEOMETRIC



BASE PLATE SUBJECT TO TENSILE LOAD

Bolt to column center dist.	f =	229	[mm]
Bolt to column web center dist.	f_1 =	229	[mm]

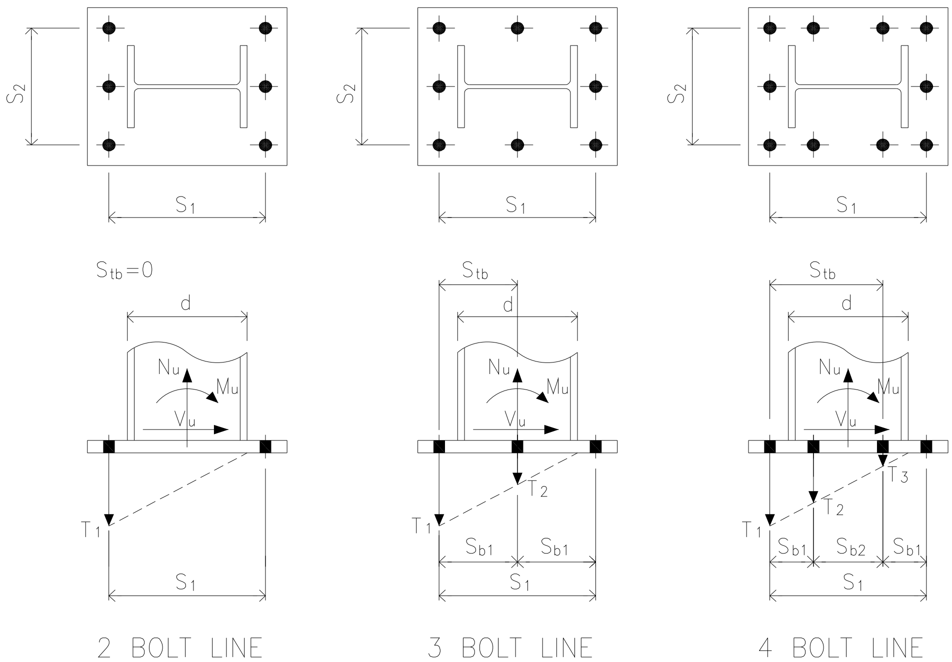
Suggested plate thickness for rigidity: t_p = max. of m/4 and n/4 = No ?

Factored column load

LCB	Cases	P_u [kN]	V_u [kN]	M_u [kNm]
LCB1	Axial Comp.	444.8	66.7	0.0
LCB2	Axial Comp. + M	0.0	89.0	40.7
LCB3	Axial Comp. + M	66.7	89.0	40.7
LCB4	Axial Tensile	44.5	155.7	0.0

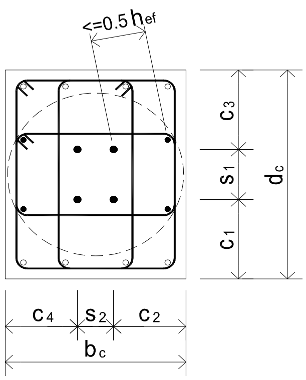
Code Reference

Anchor Bolt Data

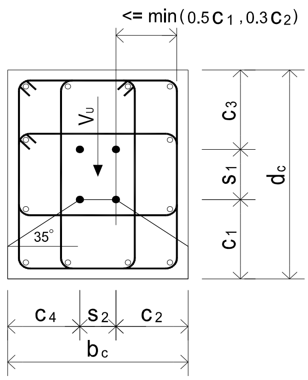


No of bolt line for resisting moment		= 3 Bolt Line					
No of bolt along outermost bolt line		= 3					
				min required			
Outermost bolt line spacing s_1	s_1	= 457	[mm]	89	OK		*PIP STE05121*
Outermost bolt line spacing s_2	s_2	= 457	[mm]	89	OK		Page A -1 Table 1
Internal bolt line spacing s_{b1}	s_{b1}	= 229	[mm]	89	OK		
Internal bolt line spacing s_{b2}	s_{b2}	= 0	[mm]	89	OK		
Anchor bolt material		= F1554 Grade 55					
Anchor tensile strength	f_{uta}	= 75.0	[ksi]		= 517	[MPa]	*A23.3-04 (R2010)*
		Anchor is ductile steel element					D.2
Anchor bolt diameter	d_a	= 0.875	[in]	max 1.5 in	= 22.2	[mm]	*PIP STE05121*
Bolt sleeve diameter	d_s	= 51	[mm]				Page A -1 Table 1
Bolt sleeve height	h_s	= 178	[mm]				
				min required			
Anchor bolt embedment depth	h_{ef}	= 508	[mm]	267	OK		Page A -1 Table 1
Pedestal height	h_a	= 584	[mm]	584	OK		
Pedestal width	b_c	= 3150	[mm]				
Pedestal depth	d_c	= 3150	[mm]				

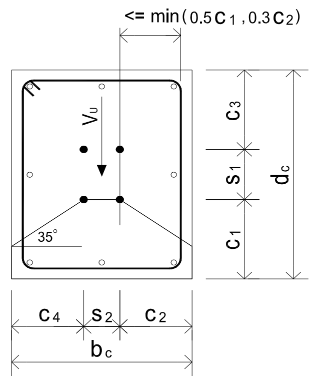
						Code Reference
Bolt edge distance c_1	c_1 =	152	[mm]	135	**OK**	*PIP STE05121*
Bolt edge distance c_2	c_2 =	152	[mm]	135	**OK**	Page A -1 Table 1
Bolt edge distance c_3	c_3 =	2540	[mm]	135	**OK**	
Bolt edge distance c_4	c_4 =	2540	[mm]	135	**OK**	



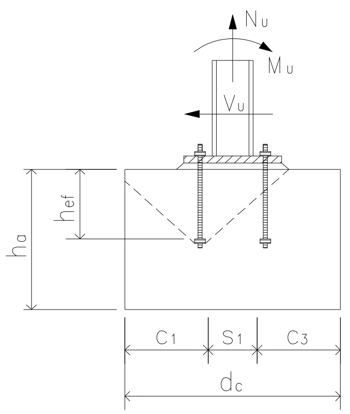
Ver. Reinft For Tension

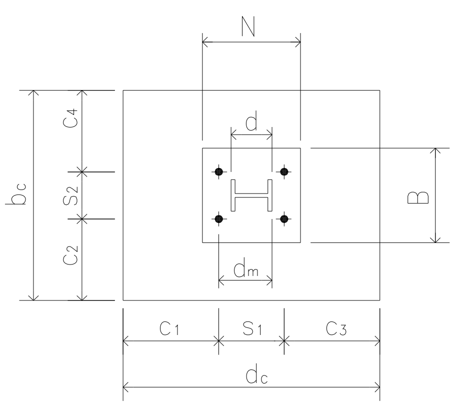


Hor. Ties For Shear - 4 Legs



Hor. Ties For Shear - 2 Legs





ACI318 M-08

To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within $0.5h_{ef}$ from the outmost anchor's centerline. In this design $0.5h_{ef}$ value is limited to 200mm. RD.5.2.9

$0.5h_{ef}$ = **200** [mm]

No of ver. rebar that are effective for resisting anchor tension n_v = 6

Ver. bar size d_b = 25 single bar area A_s = 500 [mm^2]

To be considered effective for resisting anchor shear, hor. reinft shall be located within min($0.5c_1$, $0.3c_2$) from the outmost anchor's centerline RD.6.2.9

min($0.5c_1$, $0.3c_2$) = **46** [mm]

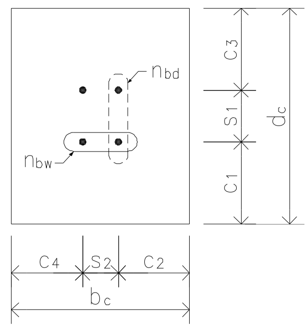
No of tie <u>leg</u> that are effective to resist anchor shear n_{leg} = 2 ?

No of tie <u>layer</u> that are effective to resist anchor shear n_{lay} = 2 ?

Tie bar size d_b = 15 single bar area A_s = 200 [mm^2]

For anchor reinft shear breakout strength calc 100% hor. tie bars develop full yield strength ?

				suggest
Concrete strength	f'_c =	31	[MPa]	30
Rebar yield strength	f_y =	414	[MPa]	400
Base plate yield strength	F_y =	248	[MPa]	300
Total no of anchor bolt	n =	8		
No of anchor bolt carrying shear	n_s =	8		
For side-face blowout check use				
No of bolt along width edge	n_{bw} =	3		
No of bolt along depth edge	n_{bd} =	3		



Bolt No Input for Side-Face Blowout Check Use

Anchor head type	=	Heavy Hex		?		
Anchor effective cross sect area	A_{se} =	0.462	[in^2]		= 298	[mm^2]
Bearing area of head	A_{brg} =	**1.188**	[in^2]		= 766	[mm^2]
	A_{brg}		[in^2]	not applicable		

				Code Reference
Bolt 1/8" (3mm) corrosion allowance	=	No	?	
Provide shear key ?	=	No	?	*A23.3-04 (R2010)*
Seismic region where $I_E F_a S_a(0.2)$>=0.35	=	No	?	D.4.3.5
Provide built-up grout pad ?	=	Yes	?	D.7.1.3

CONCLUSION

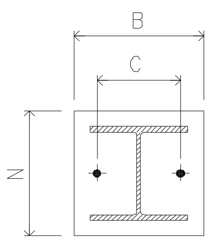
OVERALL	ratio = **0.94**	**OK**
BASE PLATE		
Base Plate Size and Anchor Bolt Tensile		**OK**
Base Plate Thickness	ratio = **0.52**	**OK**
ANCHOR BOLT		
LCB1 Axial Compression		
Abchor Rod Embedment, Spacing and Edge Distance		**OK**
Min Rquired Anchor Reinft. Development Length	ratio = 0.94	**OK**
Overall Ratio	ratio = **0.29**	**OK**
LCB2 Axial Compression + Moment		
Abchor Rod Embedment, Spacing and Edge Distance		**OK**
Min Rquired Anchor Reinft. Development Length	ratio = 0.94	**OK**
Overall Ratio	ratio = **0.57**	**OK**
LCB3 Axial Compression + Moment		
Abchor Rod Embedment, Spacing and Edge Distance		**OK**
Min Rquired Anchor Reinft. Development Length	ratio = 0.94	**OK**
Overall Ratio	ratio = **0.49**	**OK**
LCB4 Axial Tensile		
Abchor Rod Embedment, Spacing and Edge Distance		**OK**
Min Rquired Anchor Reinft. Development Length	ratio = 0.94	**OK**
Overall Ratio	ratio = **0.68**	**OK**

BASE PLATE DESIGN

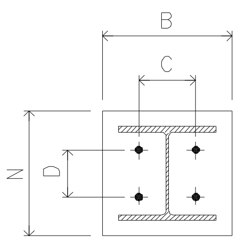
Base plate design based on	**Code Abbreviation**
AISC Design Guide 1: Base Plate and Anchor Rod Design 2nd Edition	*AISC Design Guide 1*
CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D	*A23.3-04 (R2010)*
ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary	*ACI318 M-08*

DESIGN DATA

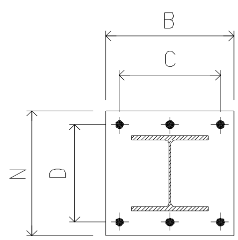
Column section type		W_Shape					
Column size		W360x79					
Depth	d =	354.0	[mm]	Flange thickness t_f	=	16.8	[mm]
Flange width	b_f =	205.0	[mm]	Web thickness t_w	=	9.4	[mm]
Base plate anchor bolt pattern		4 or 6-Bolt MC WF		base plate is moment connection			



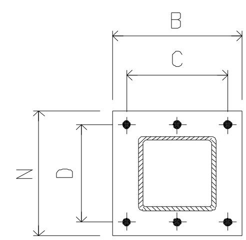
2-BOLT PIN



4-BOLT PIN

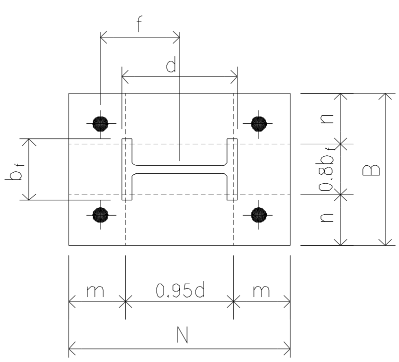


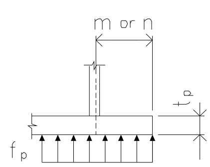
4 or 6-Bolt MC WF



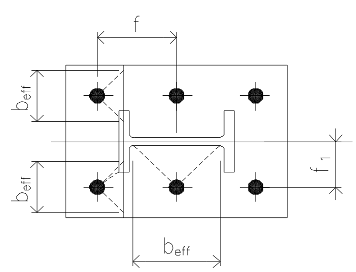
4 or 6-Bolt MC HS

				suggest
Base plate width	B =	559	[mm]	380
Base plate depth	N =	559	[mm]	530
Base plate thickness	t_p =	51	[mm]	45
Anchor bolt spacing	C =	457	[mm]	280
Anchor bolt spacing	D =	457	[mm]	430
Anchor bolt diameter	d =	0.875	[in]	max 1.5 in





BASE PLATE GEOMETRIC



BASE PLATE SUBJECT TO TENSILE LOAD

				suggest
Bolt to column center dist.	f =	229	[mm]	228.6 mm
Bolt to column web center dist.	f_1 =	229	[mm]	228.6 mm
Pedestal width	b_c =	3150	[mm]	>= 724 mm
Pedestal depth	d_c =	3150	[mm]	>= 724 mm

Factored column load

LCB	Cases	P_u [kN]	M_u [kNm]	t_p (mm)	Base Plate Size
LCB1	Axial Compressive	444.8	0.0	22.3	**Base Plate B x N OK**
LCB2	Compression + M	0.0	40.7	22.5	**Base Plate B x N OK**
LCB3	Compression + M	66.7	40.7	26.3	**Base Plate B x N OK**
LCB4	Axial Tensile	44.5	0.0	7.1	**Anchor Bolt Tensile OK**
Min required plate thickness				**26.3**	

suggest max plate thickness 45 mm

Suggested plate thickness for rigidity: t_p = max. of m/4 and n/4 = No

For base plate subject to tensile force only

Total No of anchor bolt n = 8

Bolt pattern Bolt Outside Flange Only

For base plate subject to large moment

No of bolt resisting tensile force n_t = 5

Anchor rod material F1554 Grade 55

Anchor rod tensile strength f_{uta} = 75.0 [ksi] = 517 [MPa]

Bolt 1/8" (3mm) corrosion allowance No

Anchor rod effective area A_{se} = 0.462 [in^2] = 298 [mm^2]

Concrete strength f'_c = 31 [MPa]

Base plate yield strength F_y = 248 [MPa]

		Code Reference
Strength reduction factor		*A23.3-04 (R2010)*
Bearing on concrete	ϕ_c = 0.65	8.4.2
Steel anchor resistance factor	ϕ_s = 0.85	8.4.3 (a)
Base plate bending	ϕ_b = 0.90	

CONCLUSION

[Base Plate Size and Anchor Bolt Tensile Is Adequate] **OK**

[The Base Plate Thickness Is Adequate] ratio= **0.52**

3 of 6

DESIGN CHECK

				Code Reference
For base plate subject to large moment				*A23.3-04 (R2010)*
Anchor rod tensile resistance	$T_r = n_t A_{se} \phi_s f_{uta} R_{t,s}$	= 524.0	[kN]	D.6.1.2 (D-3)
	$R_{t,s}$ = 0.80 for ductile steel in tension			D.5.4(a)
W Shapes	m = (N - 0.95d) / 2	= **111.3**	[mm]	*AISC Design Guide 1*
	n = (B -0.8b_f) / 2	= **197.4**	[mm]	3.1.2 on Page 15
HSS Rectangle Shapes	m = (N - 0.95d) / 2	= 111.3	[mm]	3.1.3 on Page 16
THIS SECTION NOT APPLICABLE	n = (B -0.95b_f) / 2	= 182.0	[mm]	
HSS Round Shapes	m = (N - 0.8d) / 2	= 137.8	[mm]	3.1.3 on Page 16
THIS SECTION NOT APPLICABLE	n = (B -0.8b_f) / 2	= 137.8	[mm]	
m value used for design	m =	= **111.3**	[mm]	
n value used for design	n =	= **197.4**	[mm]	
Suggested plate thickness for rigidity: t_p = max. of m/4 and n/4		= **49.4**	[mm]	
Base plate area	A_1 = B x N	= 3.1E+05	[mm^2]	
Pedestal area	$A_2 = b_c \times d_c$	= 9.9E+06	[mm^2]	
				A23.3-04 (R2010)
	k = min [sqrt(A_2/A_1), 2]	= 2.00		10.8.1
	$\phi_c P_n = \phi_c\, 0.85\, f_c'\, A_1\, k$	= 10696.4	[kN]	
		> P_u	**OK**	
LCB1: Axial Compressive				*AISC Design Guide 1*
	$X = \frac{4db_f}{(d+b_f)^2}\frac{P_u}{\phi_c P_p}$	= 0.039		3.1.2 on Page 16
	$\lambda = \min\left(\frac{2\sqrt{X}}{1+\sqrt{1-X}}, 1\right)$	= 0.2		
	$\lambda n' = \lambda\, \mathrm{sqrt}(d x b_f) / 4$	= 13.4	[mm]	
For W shape	L = max (m, n, $\lambda n'$)	= 197.4	[mm]	3.1.2 on Page 15
For HSS and Pipe	L = max (m, n)	= 197.4	[mm]	3.1.3 on Page 16
L value used for design	L =	**197.4**	[mm]	
	$t_p = L\sqrt{\frac{2P_u}{\phi_b F_y B N}}$	= **22.3**	[mm]	

Base Plate B x N OK

LCB2: Axial Compression + Moment **Code Reference**

P_u = 0.1 [kN] M_u = 40.7 [kNm]

$e = M_u / P_u$ = 407000 [mm]

$f_{p(max)} = \phi_c\ 0.85\ f_c'\ k$ = 34.3 [MPa]

$q_{max} = f_{p(max)} \times B$ = 19142 [N/mm]

$e_{crit} = N/2 - P_u / (2q_{max})$ = 279.4 [mm]

$e > e_{crit}$ Large moment case applied

Small moment case	**This case does not apply**			*AISC Design Guide 1*
Bearing length	$Y = N - 2e$	= 0.0	[mm]	
Verify linear bearing pressure	$q = P_u / Y$	= 0	[N/mm]	
		< q_{max}	**OK**	
	$f_p = P_u / BY$	= 0.0	[MPa]	
	$m = \max(m\ ,\ n)$	= 197.4	[mm]	
If Y>=m	$t_{req1} = 1.49m\ \mathrm{sqrt}(f_p / F_y)$	= 0.0	[mm]	Eq. 3.3.14a-1
If Y<m	$t_{req2} = 2.11\sqrt{\dfrac{f_p Y\left(m - \frac{Y}{2}\right)}{F_y}}$	= 0.0	[mm]	Eq. 3.3.15a-1
	$t_{min} = \max(t_{req1}, t_{req2})$	= **0.0**	[mm]	

THIS SECTION NOT APPLICABLE

Large moment case	**This case applies**			
Check if real solution of Y exist	$var_1 = (f + N/2)^2$	= 258064	[mm^2]	
	$var_2 = 2P_u\ (e+f) / q_{max}$	= 4255	[mm^2]	
	$var_1 > var_2$		**OK**	
Bearing length	$Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_u(e+f)}{q_{max}}}$	= 4.2	[mm]	Eq. 3.4.3
Anchor rod tension force	$T_u = q_{max}Y - P_u$	= 80.4	[kN]	Eq. 3.4.2
	ratio = 0.15	< T_r	**OK**	
At anchor rod tension interface				
	$x = f - d/2 + t_f / 2$	= 60.0	[mm]	Eq. 3.4.6
	$t_{req\text{-}t} = 2.11\sqrt{\dfrac{T_u\ x}{BF_y}}$	= 12.4	[mm]	Eq. 3.4.7a
At conc. bearing interface				
	$m = \max(m\ ,\ n)$	= 197.4	[mm]	
If Y>=m	$t_{req\text{-}b} = 1.49m\ \mathrm{sqrt}(f_{p(max)} / F_y)$	= 0.0	[mm]	Eq. 3.3.14a-2
If Y<m	$t_{req\text{-}b} = 2.11\sqrt{\dfrac{f_{p(max)} Y\left(m - \frac{Y}{2}\right)}{F_y}}$	= 22.5	[mm]	Eq. 3.3.15a-2
	$t_{min} = \max(t_{req\text{-}t}, t_{req\text{-}b})$	= **22.5**	[mm]	

Base Plate B x N OK

LCB3: Axial Compression + Moment **Code Reference**

P_u = 66.7 [kN] M_u = 40.7 [kNm]

$e = M_u / P_u$ = 610 [mm]

$f_{p(max)} = \phi_c \, 0.85 \, f_c' \, k$ = 34.3 [MPa]

$q_{max} = f_{p(max)} \times B$ = 19142 [N/mm]

$e_{crit} = N/2 - P_u / (2q_{max})$ = 277.7 [mm]

$e > e_{crit}$ Large moment case applied

Small moment case **This case does not apply** *AISC Design Guide 1*

THIS SECTION NOT APPLICABLE

Bearing length $Y = N - 2e$ = 0.0 [mm]

Verify linear bearing pressure $q = P_u / Y$ = 0 [N/mm]

$< q_{max}$ **OK**

$f_p = P_u / BY$ = 0.0 [MPa]

$m = \max(m\ ,\ n)$ = 197.4 [mm]

If Y>=m $t_{req1} = 1.49 m \,\mathrm{sqrt}(f_p / F_y)$ = 0.0 [mm] Eq. 3.3.14a-1

If Y<m $t_{req2} = 2.11\sqrt{\dfrac{f_p Y\left(m - \dfrac{Y}{2}\right)}{F_y}}$ = 0.00 [mm] Eq. 3.3.15a-1

$t_{min} = \max(t_{req1}, t_{req2})$ = **0.0** [mm]

Large moment case **This case applies**

Check if real solution of Y exist $var_1 = (f + N/2)^2$ = 258064 [mm^2]

$var_2 = 2P_u (e+f) / q_{max}$ = 5846 [mm^2]

$var_1 > var_2$ **OK**

Bearing length $Y = \left(f + \dfrac{N}{2}\right) \pm \sqrt{\left(f + \dfrac{N}{2}\right)^2 - \dfrac{2P_u(e+f)}{q_{max}}}$ = 5.8 [mm] Eq. 3.4.3

Anchor rod tension force $T_u = q_{max} Y - P_u$ = 44.1 [MPa] Eq. 3.4.2

ratio = 0.08 $< T_r$ **OK**

At anchor rod tension interface

$x = f - d/2 + t_f / 2$ = 60.0 [mm] Eq. 3.4.6

$t_{req\text{-}t} = 2.11\sqrt{\dfrac{T_u\, x}{B F_y}}$ = 9.2 [mm] Eq. 3.4.7a

At conc. bearing interface

$m = \max(m\ ,\ n)$ = 197.4 [mm]

If Y>=m $t_{req\text{-}b} = 1.49 m \,\mathrm{sqrt}(f_{p(max)} / F_y)$ = 0.00 [mm] Eq. 3.3.14a-2

If Y<m $t_{req\text{-}b} = 2.11\sqrt{\dfrac{f_{p(max)} Y\left(m - \dfrac{Y}{2}\right)}{F_y}}$ = 26.3 [mm] Eq. 3.3.15a-2

$t_{min} = \max(t_{req\text{-}t}, t_{req\text{-}b})$ = **26.3** [mm]

Base Plate B x N OK

LCB4: Axial Tensile

Factored tensile load P_u = = 44.5 [kN]

For base plate subject to tensile force only *A23.3-04 (R2010)*

Anchor rod tensile resistance $T_r = n\, A_{se}\, \phi_s\, f_{uta}\, R_{t,s}$ = 838.5 [kN] D.6.1.2 (D-3)

$R_{t,s}$ = 0.80 for ductile steel in tension D.5.4(a)

ratio = 0.05 > P_u **OK**

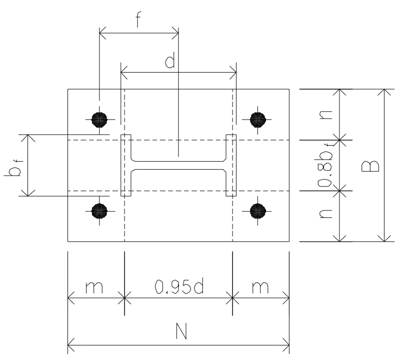
Bolt pattern Bolt Outside Flange Only

Total No of anchor bolt n = 8

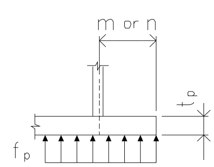
Bolt to column center dist. f = 229 [mm]

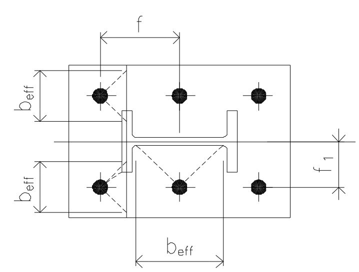
Bolt to column web center dist. f_1 = 229 [mm]

Each bolt factored tensile load T_u = 5.6 [kN]



BASE PLATE GEOMETRIC





BASE PLATE SUBJECT TO TENSILE LOAD

Bending to Column Flange

Moment lever arm a = 60 [mm]

Moment to column flange M_u = 0.3 [kNm]

Effective plate width $b_{eff} = 2 \times a$ = 120 [mm]

Base plate required thickness $t_{p1} = \sqrt{\dfrac{4\, M_u}{b_{eff}\, \phi_b\, F_y}}$ = 7.1 [mm]

Bending to Column Web

Moment lever arm a = 224 [mm]

Moment to column flange M_u = 1.2 [kNm]

Effective plate width $b_{eff} = 2 \times a$ = 448 [mm]

Base plate required thickness $t_{p2} = \sqrt{\dfrac{4\, M_u}{b_{eff}\, \phi_b\, F_y}}$ = 0.0 [mm]

$t_{min} = \max(t_{p1}, t_{p2})$ = **7.1** [mm]

Anchor Bolt Tensile OK

5.0 REFERENCES

1. ACI 318-08 Building Code Requirements for Structural Concrete and Commentary

2. ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary

3. ACI 349-06 Code Requirements for Nuclear Safety-Related Concrete Structures & Commentary

4. ACI 349.2R-07 Guide to the Concrete Capacity Design (CCD) Method - Embedment Design Examples

5. ACI 355.3R-11 Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D

6. Design of Anchor Reinforcement in Concrete Pedestals by Widianto, Chandu Patel, and Jerry Owen

7. CSA A23.3-04 (R2010) - Design of Concrete Structures

8. AISC Design Guide 1: Base Plate and Anchor Rod Design 2nd Edition

9. PIP STE05121 Anchor Bolt Design Guide-2006