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1.0 INTRODUCTION

Anchorage to concrete Concrete Capacity Design (CCD) Method was first introduced in ACI 318-02 and ACI 349-01 Appendix D, followed by CSA A23.3-04 Annex D. Anchorage design provisions in ACI 318-08 and ACI 349-06 Appendix D, CSA A23.3-04 Annex D are similar except that ACI 349-06 imposes a more severe penalty on non-ductile anchor design (ACI 349-06 D3.6.3) and also ACI 349-06 provides additional provisions for shear transfer using friction and shear lugs.

Since ACI 318-02 the ACI has released ACI 318-05, ACI 318-08, and recently ACI 318-11. In ACI 318-08 the definition for Anchor Reinforcement is introduced, and the strength of Anchor Reinforcement used to preclude concrete breakout in tension and in shear is codified (ACI 318-08 D.5.2.9 and D.6.2.9.), guidance for detailing the Anchor Reinforcement is given in ACI 318-08 RD.5.2.9 and RD.6.2.9.

Since CSA A23.3-04 CSA has released several updates to catch up ACl's revisions on anchorage design, with the latest CSA A23.3-04 (R2010, Reaffirmed 2010) partially incorporated Anchor Reinforcement (CSA A23.3-04 R2010 D.7.2.9). It's expected that the same Anchor Reinforcement provisions as ACI 318-08 will be amended in the next revision of CSA A23.3-04 update.

This technical writing includes a series of design examples covering mainly the anchorage design of grouped anchors and studs, in both ACI 318-08 and CSA A23.3-04 R2010 code. The design examples are categorized in Anchor Bolt and Anchor Stud, with Anchor Reinforcement and without Anchor Reinforcement, with moment presence and without moment presence.

Anchor Bolt and Anchor Stud

The main difference between anchor bolt and anchor stud is the way how they attach to the base plate. For anchor bolt normally the anchor bolt holes on base plate are much bigger than anchor bolt diameter due to cast-in anchor bolt construction tolerance, while the anchor stud is rigidly welded to the base plate. This different approach of attachment will cause the difference on shear transfer mechanism during anchorage design (ACI 318-08 RD.6.2.1(b)).

Anchor Reinforcement and Supplementary Reinforcement

In all concrete failure modes, the tensile and shear concrete breakout strengths are most of the time the lowest strengths among all concrete failure modes. The concrete breakout strength limits the anchor design strength and make anchor bolt design not practical in many applications such as concrete pedestal, which has limited edge distances surrounding anchor bolts.

In ACI 318-08 the definition for Anchor Reinforcement is introduced, and the strength of Anchor Reinforcement used to preclude concrete breakout in tension and in shear is codified (ACI 318-08 D.5.2.9 and D.6.2.9.), guidance for detailing the Anchor Reinforcement is given in ACI 318-08 RD.5.2.9 and RD.6.2.9. The use of Anchor Reinforcement in many times is the only choice to make a practical anchor bolt design in applications such as concrete pedestal.

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Anchor reinforcement

placed symmetrically

Anchor Reinforcement for Tension ACI 318-08 RD.5.2.9

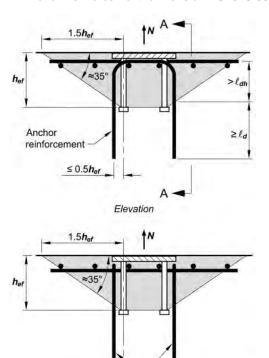


Fig. RD.5.2.9—Anchor reinforcement for tension.

Section A-A

 $\leq 0.5 h_{ef}$

Anchor Reinforcement for Shear ACI 318-08 RD.6.2.9

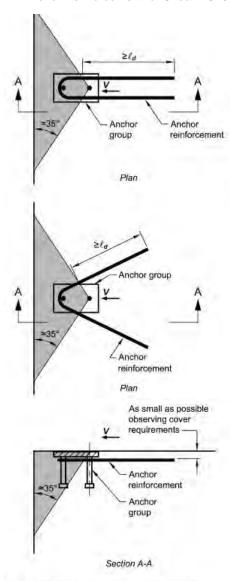


Fig. RD.6.2.9(a)—Hairpin anchor reinforcement for shear.

The use of supplementary reinforcement is similar to the anchor reinforcement, but it isn't specifically designed to transfer loads. If supplementary reinforcement is used, the concrete strength reduction factor ϕ is increase 7% from 0.70 to 0.75, which is not that significant in terms of increasing concrete breakout strength.

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Concrete

Supplementary Reinforcement

Design of Anchorage to Concrete Using ACI 318-08 & CSA-A23.3-04 Code

 $\phi_{to} = 0.70$

ACI 318-08

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Cdn-B D.4.4 (c)

Supplementary reinforcement	-		
Fortension	No 💌 Condition B		D.4.4 (c)
For shear	Ψ _{c,V} = 1 Condition B	2	D.6.2.7
Provide built-up grout pad?	Yeŧ▼ ?		D.6.1.3
Strength reduction factors			
Anchor reinforcement	$\phi_8 = 0.75$		D.5.2.9 & D.6.2.9
Anchor rod - ductile steel	$\phi_{ts} = 0.75$	$\phi_{u,s} = 0.65$	D.4.4 (a)

Condition B

Supplementary Reinforcement ACI 318-08 Condition A

Supplementary reinforcement			
Fortension	Yes▼ Condition A		D.4.4 (c)
For shear	Ψ _{c,V} = 1.2 Condition A	2	D.6.2.7
Provide built-up grout pad?	Yes.▼ ?		D.6.1.3
Strength reduction factors			
Anchor reinforcement	$\phi_8 = 0.75$		D.5.2.9 & D.6.2.9
Anchor rod - ductile steel	$\phi_{ts} = 0.75$	$\phi_{u,s} = 0.65$	D.4.4 (a)
Concrete	$\phi_{tc} = 0.75$ Cdn-A	$\phi_{u,c} = 0.75$ Cdn-A	D.4.4 (c)

Anchor Ductility

When an anchor's overall design strength, for both tension and shear, is equal to the design strength of anchor rod steel element, and all potential concrete failure modes have design strengths greater than the anchor rod steel element design strength, this anchor design is considered as ductile anchor design.

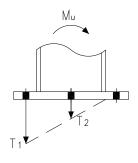
Anchor's ductility is its own characteristic related to anchor rod material, embedment depth, anchor bolt spacing and edge distances etc, and has nothing to do with the applied loadings. If high strength anchor rod material is used, it would be more difficult to achieve the ductile design as deeper embedment depth, larger edge distances are required for concrete failure modes design strengths to surpass anchor rod material design strength. The high strength anchor bolt material shall only be used when it's necessary, such as for anchorages required pre-tensioned or subjected to dynamic impact load in cold temperature environment (A320 Grade L7). In most cases the anchorage design won't benefit from the high strength bolt material as the concrete failure modes will govern, and the use of high strength bolt will make the anchor ductile design almost impossible.

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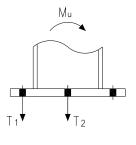
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For anchorage design in moderate to high seismic zone (ACI 318-08 SDC>=C and CSA A23.3-04 R2010 $I_EF_aS_a(0.2)>=0.35$) ductile anchor design is mandatory as specified in ACI 318-08 D.3.3.4 and CSA A23.3-04 R2010 D.4.3.6.

For anchorage design in low seismic zone (ACI 318-08 SDC<C and CSA A23.3-04 R2010 $I_EF_aS_a(0.2)<0.35$), the non-ductile anchor design is permitted, but when calculating anchor bolt force distribution, the plastic analysis approach is not permitted for non-ductile anchor as specified in ACI 318-08 D.3.1 and CSA A23.3-04 R2010 D.4.1.



ELASTIC ANALYSIS



PLASTIC ANALYSIS

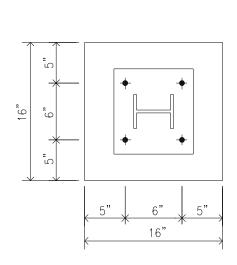
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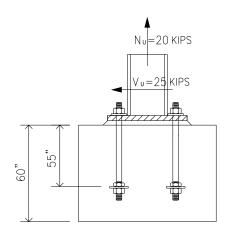


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2.0 DESIGN EXAMPLES

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

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	ombined Tension	n and Sh	ear			On to Although the
Anchor bolt design based on	nata fan Otmortonal	0				Code Abbreviation
ACI 318-08 Building Code Requireme		Concrete	and Commentary Ap	ppenaix D		ACI 318-08
PIP STE05121 Anchor Bolt Design G	uide-2006					PIP STE05121
						Code Reference
Assumptions						ACI 318-08
1. Concrete is cracked						D 4.47)
Condition A - supplementary reinform	·					D.4.4 (c)
3. Load combinations shall be as per						D.4.4
4. Anchor reinft strength is used to rep		nsion / sh	ear breakout strength	as per		
ACI318-08 Appendix D clause D.5.						D.5.2.9 & D.6.2.9
5. For tie reinft, only the top most 2 or	,		•			
Strut-and-Tie model is used to anly	ze the shear tran	sfer and t	to design the required	tie reinft		
7. Anchor bolt washer shall be tack w	elded to base pla	te for all a	anchor bolts to transfe	er shear		AISC Design Guide 1 section 3.5.3
Anchor Bolt Data						
Endowski ()			compression	20.5	FI 5 13	
Factored <u>tension</u> for design	$N_u = 20.0$	[kips]		= 89.0	[kN]	
Factored shear	$V_u = 25.0$	[kips]		= 111.2	[kN]	
Factored shear for design	$V_u = 25.0$	[kips]	$V_u = 0$ if shear key			
Concrete strength	f' _c = 4.0	[ksi]	=	= 27.6	[MPa]	
Anchor bolt material		4 Grade	<mark>36</mark> ▼			
Anchor tensile strength	$f_{uta} = 58$	[ksi]		= 400	[MPa]	ACI 318-08
			steel element			D.1
Anchor bolt diameter	$d_a = 1$	T [ii	n]	= 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]				
<=0.5 hef		< =	min(0.5 C ₁ , 0.3 C ₂)		<= min	0(0.5 C ₁ ,0.3 C ₂)
C1 81 C3	35°		C1	35°		C1 81 C3 dc
$ \begin{array}{c cccc} & c_4 & s_2 & c_2 \\ & b_c & & \\ \end{array} $	C4	b _c	S ₂	* .	C ₂ C ₂	
Ver. Reinft For Tension	Hor. Tie	s For She	ear - 4 Legs	Hor. Ties F	or Shear	r - 2 Legs

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			min required		2 of 6
Bolt edge distance c ₁	$c_1 = 5.0$	[in]	4.5	ОК	Code Reference
Bolt edge distance c ₂	$c_2 = 5.0$	[in]	4.5	ок	PIP STE05121
Bolt edge distance c ₃	$c_3 = 5.0$	[in]	4.5	ОК	Page A -1 Table 1
Bolt edge distance c ₄	$c_4 = 5.0$	[in]	4.5	ОК	r age / C T ag. e T
	4 0.0	[]			
Outermost bolt line spacing s ₁	$s_1 = 6.0$	[in]	4.0	ОК	Page A -1 Table 1
Outermost bolt line spacing s ₂	$s_2 = 6.0$	[in]	4.0	ОК	131
	2	t3			ACI 318-08
To be considered effective for resisting	ng anchor tension.	ver reinfo	rcing bars shall be	located	RD.5.2.9
within 0.5h _{ef} from the outmost ancho	-		-		
G.		J		5h _{ef} = 8.0	[in]
				- 61	F., -1
No of ver. rebar that are effective for	resisting anchor te	ension		n _v = 8	
Ver. bar size No.	8 1.000	[in] dia	single bar area		[in ²]
			-		
To be considered effective for resisting	ng anchor shear, h	or. reinft s	hall be located		RD.6.2.9
within min($0.5c_1$, $0.3c_2$) from the out	•		min(0.5c ₁ , 0.3	$3c_2$) = 1.5	[in]
No of tie <u>leg</u> that are effective to re	sist anchor shear			n _{leg} = 4	?
No of tie <u>layer</u> that are effective to re	sist anchor shear			$n_{lay} = 2$?
Hor. tie bar size No.	4 • 0.500	[in] dia	single bar area		in ²]
For anchor reinft shear breakout stre			tie bars develop		
	,		suggest		
Rebar yield strength	$f_{v} = 60$	[ksi]	60	= 414	[MPa]
No of bolt carrying tension	$n_t = 4$				l m
No of bolt carrying shear	$n_s = 4$				• [•] Nbd
					S O
For side-face blowout check use				n	bw bw
No of bolt along width edge	$n_{bw} = 2$				5
No of bolt along depth edge	$n_{bd} = 2$				
				—	C4 S2 C2
Anchor head type	= Hex		₩	? ←	bc'
Anchor effective cross sect area	$A_{se} = 0.606$	[in ²]			
Bearing area of head	$A_{brg} = 1.163$	[in ²]			olt No Input for Side—Face owout Check Use
	A _{brg}	[in ²]	not applicable	יוט	Swout Check Ose
Bolt 1/8" (3mm) corrosion allowance	= No =	?			
Provide shear key ?	= No -	?			ACI 318-08
Soismic design astagomes C	= Yes -	?			D.3.3.3
Seismic design category >= C		₹			D 6 4 3
Provide built-up grout pad?	= Yes -	?			D.6.1.3
	= Yes ▼	?			D.6.1.3
Provide built-up grout pad ?	= Yes ▼ φ _s = 0.75	?			D.6.1.3 D.5.2.9 & D.6.2.9
Provide built-up grout pad ? Strength reduction factors	_	?		$\phi_{v,s} = 0.65$	

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CONCLUSION				Code Reference
Abchor Rod Embedment, Spacing	and Edge Distance		ок	ACI 318-08
Min Rquired Anchor Reinft. Develo	ppment Length	ratio = 0.25	ок	12.2.1
Overall		ratio = 0.70	ок	
Tension				
Anchor Rod Tensile Resistance		ratio = 0.19	ок	
Anchor Reinft Tensile Breakout Re	esistance	ratio = 0.09	ок	
Anchor Pullout Resistance		ratio = 0.26	ок	
Side Blowout Resistance		ratio = 0.27	ок	
Shear				
Anchor Rod Shear Resistance		ratio = 0.57	ок	
Anchor Reinft Shear Breakout Res	sistance			
Strut Bearing Strength		ratio = 0.59	ок	
Tie Reinforcement		ratio = 0.46	ок	
Conc. Pryout Not Govern When he	_{of} >= 12d _a		ок	
Tension Shear Interaction	**			
Tension Shear Interaction		ratio = 0.70	ок	
Ductility				
	Tension Non-ductile	Shear Ductile	е	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		
CACULATION				g
				ACI 318-08
Anchor Rod Tensile	$\phi_{t,s} N_{sa} = \phi_{t,s} n_t A_{se} f_{uta}$	= 105.4	[kips]	D.5.1.2 (D-3)
Resistance	ratio = 0.19	> N _u	OK	2.0.1.2 (2.0)
recordance	10.10	, . · · · ·		
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	$I_a = h_{ef} - c (2 in) - 8 in x tan 3$	5 = 47.4	[in]	
	a el ()	> 12.0		12.2.1
.⊑		> 12.0	Oit	12.2.1
_				
35°				
35°				
35°				
35°				
35°				
35°				ACI 318-08
35°	$N_{rb} = \phi_s x f_y x n_v x A_s x (I_a / I_d)$	if $l_a < l_d$) = 284.2	[kips]	ACI 318-08 12.2.5
Seismic design strength reduction	$N_{rb} = \phi_s x f_y x n_v x A_s x (I_a / I_d)$	if $I_a < I_d$) = 284.2 = 213.1	[kips]	

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							4 o
						Code Reference	
Anchor Pullout Resistance						ACI 318-08	
Single bolt pullout resistance	$N_p = 8 A_{brg} f$	c 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			$> N_u$	ок		
	$\Psi_{c,p} = 1$ for cr	racked conc				D.5.3.6	
	$\varphi_{t,c} = 0.70$	pullout s	trength is always (Condition B		D.4.4(c)	
Side Blowout Resistance							
Failure Along Pedestal Width Edge							
Tensile load carried by anchors close to	o edge which m	nay cause si	de-face blowout				
along pedestal width edge	$N_{buw} = N_u \times n_b$	_w / n _t		= 10.0	[kips]	RD.5.4.2	
· · · · · · · · · · · · · · · · · · ·	c = min (c			= 5.0	[in]		
Check if side blowout applicable	$h_{ef} = 55.0$	[in]					
••	> 2.5c		side bowout is ap	plicable		D.5.4.1	
Check if edge anchors work as a	$s_{22} = 6.0$		·	$s_2 = 6.0$	[in]		
a group or work individually	< 6c		edge anchors wor			D.5.4.2	
9 .	$_{,c}$ $N_{sb} = \phi_{t,c}$ (16		J	= 40.9	[kips]	D.5.4.1 (D-17)	
	$N_{\text{sbq,w}} =$	V γ r brg) π	V · c		[,	
•	= (1+s/6	Sc) x de a Nac		= 49.1	[kips]	D.5.4.2 (D-18)	
0 1 11	,	, 11,0 00	c ₂ or c ₄) / c] / 4		[kips]	D.5.4.1	
Seismic design strength reduction		applicable	52 01 04/7 0] 7 4	= 36.8	[kips]	D.3.3.3	
• •	ratio = 0.27	арріїоавіс		> N _{buw}	OK	2.0.0.0	
Failure Along Pedestal Depth Edge	1410 - 0.27			> *Buw	Oit		
Tensile load carried by anchors close to	o edge which m	nav cause si	de-face blowout				
•	$N_{bud} = N_u \times n_b$	•	de-race blowout	= 10.0	[kino]	RD.5.4.2	
along pedestal depth edge	c = min(c			= 10.0	[kips]	ND.3.4.2	
Check if aids blowert applicable	,	,		= 5.0	[in]		
Check if side blowout applicable	$h_{ef} = 55.0$	[in]	aida bawar± :=	nliaakla		D 5 4 4	
Charle if adap analysis	> 2.5c	[im]	side bowout is ap	•	[in]	D.5.4.1	
Check if edge anchors work as a	$s_{11} = 6.0$	[in]		$s_1 = 6.0$	[in]	D 5 4 0	
a group or work individually	< 6c	o /	edge anchors wor			D.5.4.2	
	$_{\rm c}$ $N_{\rm sb} = \phi_{\rm t,c} (16)$	$0c\sqrt{A_{brg}}$) λ	√f' _c	= 40.9	[kips]	D.5.4.1 (D-17)	
	$N_{\text{sbg,d}} =$						
work as a group - applicable		sc) $x \phi_{t,c} N_{sb}$		= 49.1	[kips]	D.5.4.2 (D-18)	
work individually - not applicable			$c_1 \text{ or } c_3) / c] / 4$	= 0.0	[kips]	D.5.4.1	
Seismic design strength reduction		applicable		= 36.8	[kips]	D.3.3.3	
	ratio = 0.27			$> N_{bud}$	OK		
Group side blowout resistance $\phi_{t,c}$	$N_{\rm sbg} = \phi_{\rm t,c}$ min	$n \left(-\frac{N_{sbg,w}}{n_{bw}} n_t \right)$, $\frac{N_{\text{sbg,d}}}{n_{\text{bd}}} n_{\text{t}}$	= 73.7	[kips]		
Govern Tensile Resistance	$\mathbf{N_r} = \phi_{t,c} \min$	n (N _s , N _{rb} , N	$ m I_{cp},N_{sbg}$)	= 73.7	[kips]		

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

ACI 318-08

ΟK

Code Reference

Anchor Rod Shear $\phi_{v,s} V_{sa} = \phi_{v,s} n_s 0.6 A_{se} f_{uta} = 54.8$ [kips] D.6.1.2 (b) (D-20)

Resistance

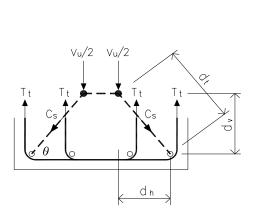
Reduction due to built-up grout pads = $x \cdot 0.8$, applicable = 43.9 [kips] D.6.1.3

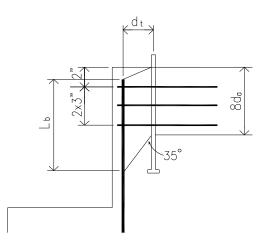
ratio = 0.57 > V_u

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 $\phi_{\rm st} = 0.75$ 9.3.2.6





Strut-and-Tie model geometry

 $d_v = 2.250$ [in]

 $d_h = 2.250$ [in]

 $\theta = 45$

 $d_t = 3.182$

Strut compression force $C_s = 0.5 V_u / \sin \theta$

= 17.7 [kips]

[in]

ACI 318-08

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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 $I_e = min(8d_a, h_{ef})$

= 8.0

[in] D.6.2.2

Anchor bearing area $A_{brg} = I_e x d_a$

= 8.0

[in²]

Anchor bearing resistance

 $C_r = n_s x \phi_{st} x f_{ce} x A_{brg}$

= 81.6 > V_u

OK

[kips]

* Bearing of ver reinft bar Ver bar bearing area

 $A_{brg} = (I_e + 1.5 \times d_t - d_a/2 - d_b/2) \times d_b$

= 11.8

[in²]

Ver bar bearing resistance

 $C_r = \phi_{st} \times f_{ce} \times A_{brq}$

= 30.0

[kips]

ratio = 0.59

 $> C_s$

ΟK

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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
$$n = n_{leg} (leg) \times n_{lay} (layer) = 8$$

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
$$T_r = \phi_s x f_v x A_s$$
 = 9.0 [kips]

Total tie bar tension resistance
$$V_{rb} = 1.0 \text{ x n x Tr}$$
 = 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
$$V_r = min (\phi_{v,s} V_{sa}, V_{rb}) = 43.9$$
 [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)

Ductility Tension

$$\begin{array}{lll} \phi_{t,s} \; N_{sa} \; = \; 105.4 & \text{[kips]} \\ \\ \; > \; \min \left[\; N_{rb} \; , \; \phi_{t,c} \left(\; N_{pn}, \; N_{sbg} \; \right) \; \right] & = \; 73.7 & \text{[kips]} \\ \\ \hline & \quad \text{Non-ductile} \end{array}$$

Ductility Shear

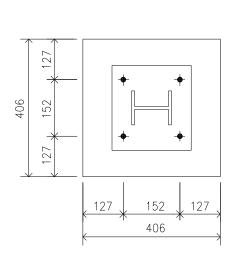
$$\phi_{v,s} V_{sa} = 43.9$$
 [kips]
$$< V_{rb} = 54.0$$
 [kips]

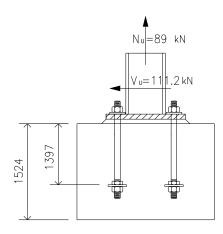
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Example 02: Anchor Bolt + Anchor Reinft + Tension & Shear + CSA A23.3-04 Code





 N_u = 89 kN (Tension) V_u = 111.2 kN

Concrete f_c '= 27.6 MPa Rebar f_v = 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 $I_E F_a S_a(0.2) >= 0.35$

Anchor reinforcement Tension \rightarrow 8-25M ver. bar

Shear → 2-layer, 4-leg 15M hor. bar

Provide built-up grout pad

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ANCHOR BOLT DESIGN Con	nbined Tensi	on and She	ear			
Anchor bolt design based on						Code Abbreviation
CSA-A23.3-04 (R2010) Design of Cond	rete Structure	s Annex D				A23.3-04 (R2010)
ACI 318M-08 Metric Building Code Req	uirements for	Structural C	Concrete and Comme	entary		ACI318 M-08
PIP STE05121 Anchor Bolt Design Gui	de-2006					PIP STE05121
Assumptions						Code Reference
Concrete is cracked						A23.3-04 (R2010)
2. Condition A - supplementary reinforc	ement is prov	ded				D.5.4 (c)
3. Anchor reinft strength is used to repla	ace concrete t	ension / she	ear breakout strength	n as per		ACI318 M-08
ACI318 M-08 Appendix D clause D.5	.2.9 and D.6.2	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	n TOC and 2x75mm	after) are effect	ctive	
5. Strut-and-Tie model is used to anlyze	the shear tra	insfer and to	design the required	l tie reinft		
6. Anchor bolt washer shall be tack wel	ded to base p	ate for all a	nchor bolts to transfe	er shear		AISC Design Guide 1
						section 3.5.3
Input Data						
	set N	$_{i} = 0$ if it's co	ompression			
Factored <u>tension</u> for design	$N_u = 89.0$	[kN]		= 20.0	[kips]	
Factored shear	$V_u = 111.2$	[kN]		= 25.0	[kips]	
Factored shear for design	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	= F15	54 Grade 3	36 ▼			
Anchor tensile strength	f _{uta} = 58	[ksi]		= 400	[MPa]	A23.3-04 (R2010)
			steel element			D.2
Anchor bolt diameter	d _a = 1	▼ [ir	n]	= 25.4	[mm]	PIP STE05121
Bolt sleeve diameter	$d_{s} = 76$	[mm]	•			Page A -1 Table 1
Bolt sleeve height	$h_s = 254$	[mm]				
Zen electe height	50 .	[]	min required			
Anchor bolt embedment depth	$h_{ef} = 1397$	[mm]	305	ОК		Page A -1 Table 1
Pedestal height	h = 1524	[mm]	1473	ОК		rago / rabio r
Pedestal width	$b_c = 406$	[mm]	1473	OIL		
Pedestal depth	$d_c = 406$ $d_c = 406$	[mm]				
'	u _c = 400		min(0.5 C ₁ ,0.3 C ₂)		<= min	(0.5 C ₁ ,0.3 C ₂)
<=0.5 hef		<u> </u>			() () () () () ()	(0.564,0.562)
C ₄ S ₂ C ₂ b _c	355 C			35° C4 S	*	C1
Ver. Reinft For Tension	├── Hor. T	ies For She	ar - 4 Legs	Hor. Ties F		- 2 Legs

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			min required			2 of 6
Bolt edge distance c ₁	$C_1 = 127$	[mm]	114		ок	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		ок	Page A -1 Table 1
Bolt edge distance c₄	$c_4 = 127$	[mm]	114		ок	rage A - rrable r
Bon dage distance o4	04 - 121	[]	114		OK	
Outermost bolt line spacing s ₁	$S_1 = 152$	[mm]	102		ок	Page A -1 Table 1
Outermost bolt line spacing s ₂	$s_1 = 152$ $s_2 = 152$	[mm]	102		OK	rage / Trable T
Outenhost bolt line spacing 32	02 - 132	[iiiiii]	102		OIX	ACI318 M-08
To be considered effective for resistir	ng anchor tension.	ver reinfo	orcing bars shall b	ne located	I	RD.5.2.9
within 0.5h _{ef} from the outmost anchor			· ·			. 1.5.10.12.10
		3		0.5h _{ef} = 2		[mm]
No of ver. rebar that are effective for	resisting anchor te	ension		$n_v = 8$		······
Ver. bar size	$d_b = 25$	_	single bar ar			[mm ²]
0.12	· b	_	3			[]
To be considered effective for resistir	ng anchor shear. h	or. reinft s	shall be located			RD.6.2.9
within min($0.5c_1$, $0.3c_2$) from the out			min(0.5c ₁ , ($0.3c_2) = 3$	38	[mm]
No of tie leg that are effective to res			,	n _{leg} = 4		?
No of tie <u>laver</u> that are effective to res	sist anchor shear				2 -	?
Hor. bar size	d _b = 15	₹	single bar ar			[mm²]
For anchor reinft shear breakout stre		100% ho	r. tie bars devel			
			suggest			_
Rebar yield strength	$f_{v} = 414$	[MPa]	400	= 6	60.0	[ksi]
	,					
No of bolt carrying tension	$n_t = 4$					C3
No of bolt carrying shear	$n_s = 4$					• (• \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \
For side-face blowout check use						8 J
No of bolt along width edge	$n_{bw} = 2$				n _{bw} -	
No of bolt along depth edge	$n_{bd} = 2$					5
Anchor head type	= Hex		₩	?		
	A _{se} = 391	[mm ²]			← C4	* * 1
Bearing area of head	$A_{brg} = 750$	[mm ²]				Dc >
-	A_{brq}	[mm ²]	not applicable		Rolt	No Input for Side-Face
Bolt 1/8" (3mm) corrosion allowance	= No -	?				out Check Use
Provide shear key ?	= No -	?				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 ?	= Yes T	?				D.7.1.3
Strength reduction factors		-				
Anchor reinforcement factor	$\phi_{as} = 0.75$					D.7.2.9
Steel anchor resistance factor	$\phi_{s} = 0.85$					8.4.3 (a)
Concrete resistance factor	$\phi_{c} = 0.65$					8.4.2
	.0					
Resistance modification factors						
Anchor rod - ductile steel	$R_{t,s} = 0.80$			$R_{v,s} = 0$	0.75	D.5.4(a)
	.,5			v,5	-	- · · (/

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CONCLUSION							Code Reference	
Abchor Rod Embedment, Spacing	and Edge	Distance				ОК	A23.3-04 (R2010)	
Min Rquired Anchor Reinft. Develo	pment Len	gth		ratio =	0.25	ОК	12.2.1	
Overall				ratio =	0.71	ОК		
Tension								
Anchor Rod Tensile Resistance	•			ratio =	: 0.21	ОК		
Anchor Reinft Tensile Breakout Re	esistance			ratio =	0.10	ОК		
Anchor Pullout Resistance				ratio =	: 0.28	ОК		
Side Blowout Resistance				ratio =	0.27	ОК		
Shear								
Anchor Rod Shear Resistance	•			ratio =	0.58	ОК		
Anchor Reinft Shear Breakout Res	sistance							
Strut Bearing Strength				ratio =	: 0.60	OK		
Tie Reinforcement				ratio =	: 0.30	ОК		
Conc. Pryout Not Govern When he	_{ef} >= 12d _a					ОК		
Anchor Rod on Conc Bearing				ratio =	: 0.21	ок		
Tension Shear Interaction								
Tension Shear Interaction				ratio =	: 0.71	ОК		
Ductility								
	Tension	Non-ductile	Si	hear	Ductile			
Seismic Design Requirement						NG	D.4.3.6	
	leFaSa(0.:	2)>=0.35, A23.3-04 [D.4.3.7 or D.4.3.	.8 must	be satisfie	d for non	-ductile design	
CACULATION							A23.3-04 (R2010)	
Anchor Rod Tensile	N _{sr} :	= n _t A _{se} φ _s f _{uta} R _{t,s}		=	425.3	[kN]	D.6.1.2 (D-3)	
Resistance	ratio =			>	· N _u	ОК		
Anchor Reinft Tensile Breakout	Resistance	•						
Min tension development length	I _d =	=		=	: 887	[mm]	12.2.3	
for ver. 25M bar								
Actual development lenngth	l _a :	= h _{ef} - c (50mm) - 20	0mm x tan35	=	: 1207	[mm]		
				>	300	ОК	12.2.1	
≤0.5her 200								
200								
	\							
35°	/							
35	r							
	N _{shr} :	= φ _{as} x f _y x n _v x A _s x (ام / ام if ام < ام)	=	: 1242.0	[kN]	12.2.5	
Seismic design strength reduction		= x 0.75 applicable	a. u,a '0/		931.5	[kN]	D.4.3.5	
Section addigit stronger roadblore	ratio =				N _u	OK		
	ialio =	- 0.10			· • Vu	UK		

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				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	ОК		
	$\Psi_{c,p} = 1$ for cracked conc			D.6.3.6	
	$R_{t,c} = 1.00$ pullout strength is always	s Condition B		D.5.4(c)	
Side Blowout Resistance					
Failure Along Pedestal Width Edge					
Tensile load carried by anchors clos	e 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 a	applicable		D.6.4.1	
Check if edge anchors work as a	$s_{22} = 152$ [mm] s	$s = s_2 = 152$	[mm]		
a group or work individually	< 6c edge anchors w	vork as a group		D.6.4.2	
Single anchor SB resistance	$N_{\text{sbr,w}} = 13.3 \text{c} \sqrt{A_{\text{brg}}} \phi_{\text{c}} \sqrt{f'_{\text{c}}} R_{\text{t,c}}$	= 181.7	[kN]	D.6.4.1 (D-18)	
Multiple anchors SB resistance	$N_{\text{sbgr,w}} =$				
work as a group - applicable	$= (1+s/6c) \times N_{sbr.w}$	= 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}	ОК		
Failure Along Pedestal Depth Edge					
Tensile load carried by anchors clos	e to edge which may cause side-face blowout			ACI318 M-08	
along pedestal depth edge	$N_{\text{bud}} = N_{\text{u}} \times n_{\text{bd}} / n_{\text{t}}$	= 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 a	applicable		D.6.4.1	
Check if edge anchors work as a		$s = s_1 = 152$	[mm]		
a group or work individually	< 6c edge anchors w		[]	D.6.4.2	
Single anchor SB resistance	$N_{\rm sbr,d} = 13.3 \text{c} \sqrt{A_{\rm brg}} \phi_{\rm c} \sqrt{f'_{\rm c}} R_{\rm t,c}$	= 181.7	[kN]	D.6.4.1 (D-18)	
Multiple anchors SB resistance	$N_{\text{sbgr,d}} = N_{\text{sbgr,d}} = N_{\text{sbgr,d}$	_ 101	[1414]	2.0.1.1 (2.10)	
work as a group - applicable	$= (1+s/6c) \times \phi_{t,c} N_{shr,d}$	= 217.9	[kN]	D.6.4.2 (D-19)	
work as a group - applicable 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	
Seismic design strength reduction	ratio = 0.27		OK	D.4.3.3	
	Talio = 0.27	> N _{bud}	OK		
Group side blowout resistance	$N_{\text{sbgr}} = \min \left(\frac{N_{\text{sbgr,w}}}{n_{\text{bw}}} n_{\text{t}}, \frac{N_{\text{sbgr,d}}}{n_{\text{bd}}} n_{\text{t}} \right)$	= 326.9	[kN]		
Govern Tensile Resistance	$\mathbf{N_r} = \min (N_{sr}, N_{rbr}, N_{cpr}, N_{sbgr})$	= 323.1	[kN]		

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

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

Anchor Rod Shear $V_{sr} = n_s A_{se} \phi_s 0.6 f_{uta} R_{v,s} = 239.2 [kN] D.7.1.2 (b) (D-21)$

Resistance

Reduction due to built-up grout pads = x 0.8, applicable = 191.4 [kN] D.7.1.3

ratio = 0.58 > V_u

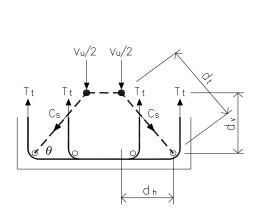
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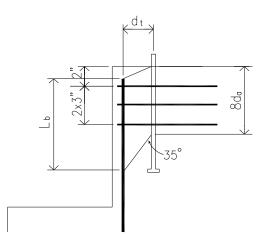
Anchor Reinft Shear Breakout Resistance

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 $\phi_{st} = 0.75$ 9.3.2.6





Strut-and-Tie model geometry

 $d_v = 57$ [mm]

 $d_h = 57$

[mm]

 $\theta = 45$

 $d_t = 81$

[mm] [kN]

Strut compression force $C_s = 0.5 V_u / \sin \theta$

= 78.6

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 $I_e = min(8d_a, h_{ef})$

= 203

[mm] D.6.2.2

Anchor bearing area

 $A_{brg} = I_e x d_a$

= 5161 = 363.3 [mm²] [kN]

Anchor bearing resistance

 $C_r \; = \; n_s \; x \; \; \varphi_{st} \; x \; f_{ce} \, x \; A_{brg}$

> V_u

oĸ

* Bearing of ver reinft bar

Ver bar bearing resistance

Ver bar bearing area $A_{brg} \ = \ (I_e + 1.5 \ x \ d_t - d_a/2 \ - d_b/2) \ x \ d_b$

= 7473

 $C_r = \phi_{st} \times f_{ce} \times A_{brq}$

= 131.5

ratio = 0.60

 $> C_s$

ΟK

[mm²]

[kN]

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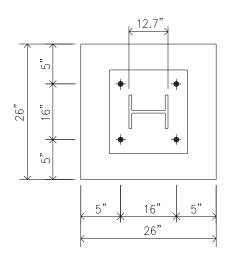
6 of 6 Tie Reinforcement **Code Reference** * For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective * For enclosed tie, at hook location the tie cannot develop full yield strength f_v . Use the pullout resistance in tension of a single J-bolt as per A23.3-04 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 $T_h = 0.9 \phi_c f_c' e_h d_b R_{t.c}$ D.6.3.5 (D-17) = 16.3[kN] $e_h = 4.5 d_h$ = 68 [mm] $T_r = \phi_{as} x f_v x A_s$ Single tie bar tension resistance = 62.1 [kN] $V_{rbr} = 1.0 \times n \times Tr$ Total tie bar tension resistance = 496.8[kN] Seismic design strength reduction = x 0.75 applicable = 372.6[kN] D.4.3.5 ratio = 0.30> V_{II} 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 $B_r = n_s x 1.4 x \phi_c x min(8d_a, h_{ef}) x d_a x f_c'$ = 518.5[kN] 25.3.3.2 $> V_u$ ratio = 0.21OK Govern Shear Resistance $V_r = min(V_{sr}, V_{rbr}, B_r)$ = 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 $N_u/N_r + V_u/V_r$ = 0.86D.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]

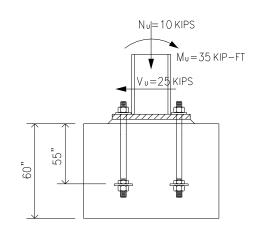
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Ductile

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





 $M_u = 35 \text{ kip-ft}$ $N_u = 10 \text{ kips}$ (Compression) $V_u = 25 \text{ kips}$

Concrete f_c '= 4 ksi Rebar f_v = 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

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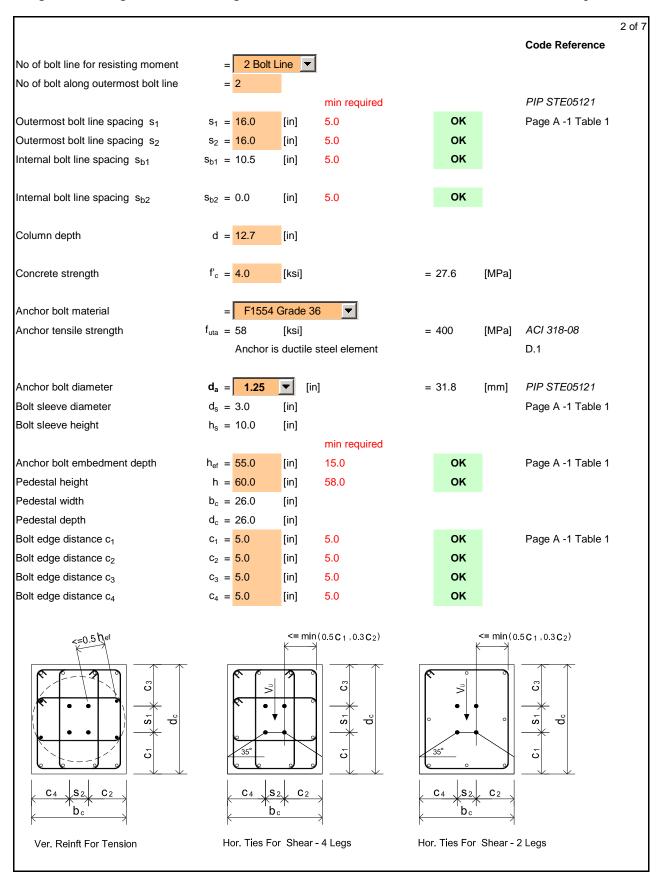


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1 of 7 ANCHOR BOLT DESIGN **Combined Tension, Shear and Moment Code Abbreviation** Anchor bolt design based on ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D ACI 318-08 PIP STE05121 Anchor Bolt Design Guide-2006 PIP STE05121 Code Reference ACI 318-08 **Assumptions** 1. Concrete is cracked 2. Condition A - supplementary reinforcement is provided D.4.4 (c) D.4.4 3. Load combinations shall be as per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2 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 D.3.1 and there is no redistribution of the forces between highly stressed and less stressed anchors 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** $M_u = 35.0$ Factored moment [kip-ft] = 47.5 [kNm] Factored tension /compression $N_u = -10.0$ [kN] [kips] = -44.5in compression $V_u = 25.0$ Factored shear [kips] = 111.2 [kN] Factored shear for design $V_u = 25.0$ [kips] $V_{II} = 0$ if shear key is provided \$2 \$2 S₁ S₁ S₁ $S_{tb} = 0$ S_{tb} d S_{b2} S_{b1} S_{b1} 2 BOLT LINE 3 BOLT LINE 4 BOLT LINE

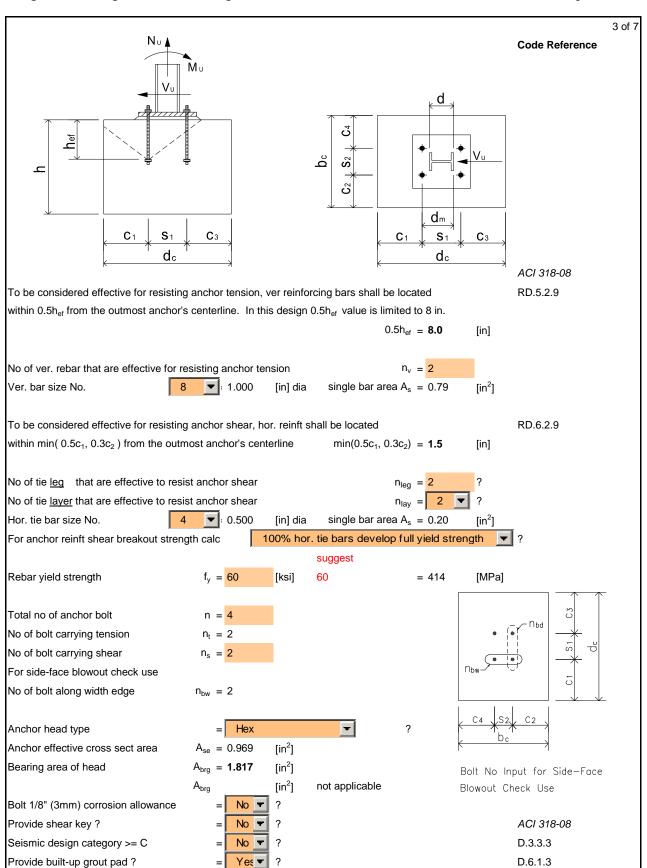
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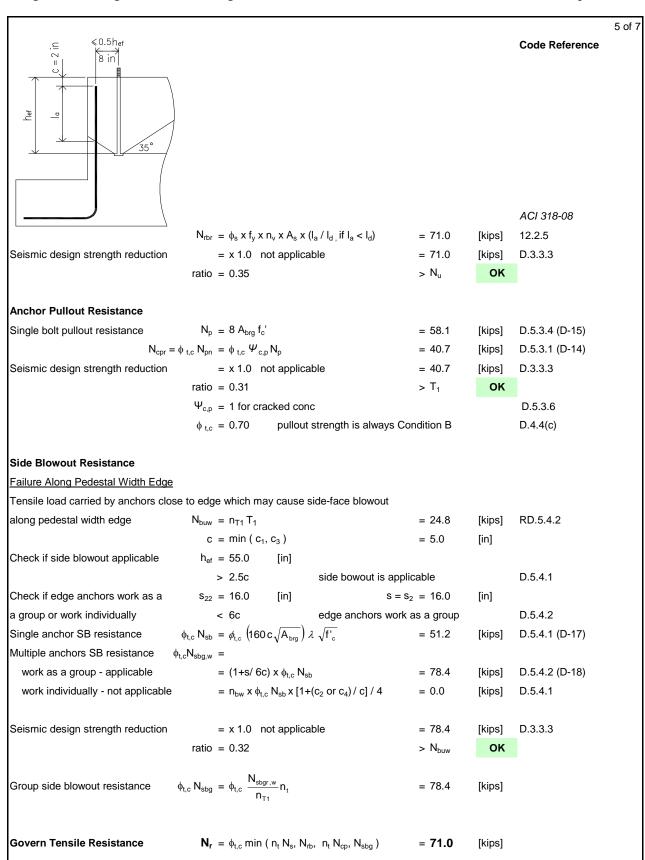


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								4 of 7
							Code Reference	
Strength reduction factors							ACI 318-08	
Anchor reinforcement	φ _s :	= 0.75					D.5.2.9 & D.6.2.9	
Anchor rod - ductile steel	$\phi_{t,s}$:	= 0.75		$\phi_{v,s}$	= 0.65		D.4.4(a)	
Concrete - condition A	$\phi_{t,c}$	= 0.75		$\phi_{v,c}$	= 0.75		D.4.4(c)	
CONCLUSION								
Abchor Rod Embedment, Spacing	g and Edge	Distance				ОК		
Min Rquired Anchor Reinft. Devel	opment Len	ngth		ratio	= 0.25	ОК	12.2.1	
Overall	_			ratio	= 0.89	OK		
Tension								
Anchor Rod Tensile Resistance				ratio	= 0.29	OK		
Anchor Reinft Tensile Breakout R	esistance			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 Re	sistance							
Strut Bearing Strength				ratio	= 0.51	ОК		
Tie Reinforcement				ratio	= 0.69	ок		
Conc. Pryout Not Govern When h	_{ef} >= 12d _a					OK		
Tension Shear Interaction								
Tension Shear Interaction	_			ratio	= 0.89	OK		
Ductility								
	Tension	Non-du	ıctile	Shear	Ductile		ACI 318-08	
Seismic Design Requirement						OK	D.3.3.4	
	SDC< C,	ACI318-0	8 D.3.3 du	ctility requirement is NO	OT required			
CACULATION								
Anchor Tensile Force							ACI 318-08	
Single bolt tensile force		= 12.42	[kips]	No of bolt for $T_1 n_{T1}$				
		= 0.00	[kips]	No of bolt for $T_2 n_{T2}$				
		= 0.00	[kips]	No of bolt for $T_3 n_{T3}$				
Sum of bolt tensile force	N _u	$= \sum n_i T_i$			= 24.8	[kips]		
Anahan Dad Tarati	1 1	1 4			40.0	flate - 3	D 5 4 0 (D 0)	
Anchor Rod Tensile	,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	$= \phi_{t,s} A_{se}$	T _{uta}		= 42.2	[kips]	D.5.1.2 (D-3)	
Resistance	ratio	= 0.29			> T ₁	OK		
Anchar Painft Tanaila Prochant	Dociotoro	•						
Anchor Reinft Tensile Breakout					_ 47 4	[in]	10 0 1 10 0 0 40	2.4
Min tension development length	l _d :	=			= 47.4	[in]	12.2.1, 12.2.2, 12.	∠. 4
for yer #8 har								
for ver. #8 bar Actual development lenngth	ı	- h o /′	2 in) - 8 in :	v tan35	= 47.4	[in]		

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

Code Reference ACI 318-08

D.6.1.2 (b) (D-20)

Anchor Rod Shear $\phi_{v,s} V_{sa} = \phi_{v,s} n_s 0.6 A_{se} f_{uta}$ = 43.8

Resistance

Reduction due to built-up grout pads

= x 0.8, applicable = 35.1

D.6.1.3 [kips]

[kips]

ratio = 0.71

 $> V_u$

OK

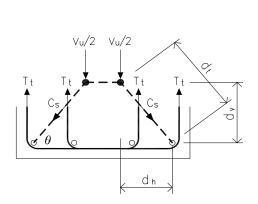
Anchor Reinft Shear Breakout Resistance

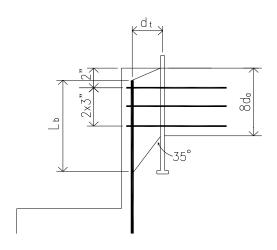
Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft

STM strength reduction factor

 $\phi_{\rm st} = 0.75$

9.3.2.6





Strut-and-Tie model geometry

 $d_v = 2.250$ [in] $d_h = 2.250$ [in]

 $\theta = 45$

 $d_t = 3.182$

 $C_s = 0.5 V_u / \sin\theta$

= 17.7 [kips]

[in]

ACI 318-08

Strut Bearing Strength

Strut compression force

Strut compressive strength

 $f_{ce} = 0.85 \, f'_{c}$

= 3.4

[ksi] A.3.2 (A-3)

Bearing of anchor bolt

Anchor bearing length

 $I_e = min(8d_a, h_{ef})$

 $C_r = \phi_{st} \times f_{ce} \times A_{brq}$

 $C_r = n_s x \phi_{st} x f_{ce} x A_{brg}$

= 10.0

[in] D.6.2.2

Anchor bearing area

 $A_{brg} = I_e x d_a$

= 12.5

= 63.8 $> V_u$

[in²] [kips]

OK

Bearing of ver reinft bar

Anchor bearing resistance

Ver bar bearing area

Ver bar bearing resistance

 $A_{brg} = (I_e + 1.5 \times d_t - d_a/2 - d_b/2) \times d_b$

= 13.6

= 34.8

[in²] [kips]

ratio = 0.51

 $> C_s$

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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_v . Use the pullout resistance in tension of a single hooked bolt as per ACl318-08 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

$$T_h = \phi_{t,c} 0.9 f_c' e_h d_a$$

$$e_{h} = 4.5 d_{h}$$

Single tie bar tension resistance

$$T_r = \phi_s x f_v x A_s$$

Total tie bar tension resistance

Seismic design strength reduction

$$V_{rb} = 1.0 \times n \times Tr$$

= x 1.0 not applicable = 36.0

$$> V_u$$

D.3.3.3

[kips]

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

$$12d_a = 15.0$$

$$h_{ef} = 55.0$$

$$12d_a = 15.0$$
 [in]

$$V_r = min (\phi_{v,s} V_{sa}, V_{rb})$$

> 12d_a

[kips]

Tension Shear Interaction

Check if $N_u > 0.2 \phi N_n$ and $V_u > 0.2 \phi V_n$

$$N_u/\phi N_n + V_u/\phi V_n$$

ratio = 0.89

= 40.7

D.7.3 (D-32) OK

Ductility Tension

$$\phi_{t,s} N_{sa} = 42.2$$
 [kips]

Yes

$$> \phi_{t,c} min (N_{rb}, N_{pn}, N_{sbg})$$

Non-ductile

Ductility Shear

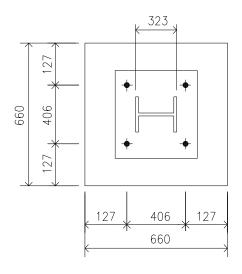
$$\phi_{v,s} V_{sa} = 35.1$$
 [kips]

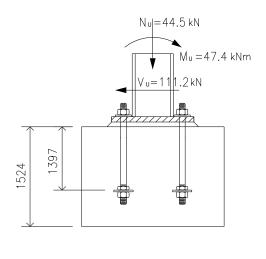
Ductile

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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_v = 414 MPa

Pedestal size 660mm x 660mm

Anchor bolt F1554 Grade 36 1.25" dia Hex Head $h_{ef} = 1397$ mm $h_a = 1524$ mm

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

Provide built-up grout pad

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						1 of
	Combined Tensior	i, Shear ar	nd Moment			On the Albhamaterian
Anchor bolt design based on CSA-A23.3-04 (R2010) Design of C	Canarata Structuras	Annov D				Code Abbreviation
			anarata and Camm	ontoni		A23.3-04 (R2010) ACI318 M-08
ACI 318M-08 Metric Building Code PIP STE05121 Anchor Bolt Design	•	liuctural Co	oncrete and Comm	ынагу		PIP STE05121
PIP STEUSTZT ANCHOL BOIL DESIGN	Guide-2006					
Assumptions						Code Reference
Concrete is cracked						A23.3-04 (R2010)
Condition A - supplementary rein	forcement is provide	ad				, ,
,	•		or brookout strongt	n ac par		D.5.4 (c) ACI318 M-08
3. Anchor reinft strength is used to a	•		ar breakout strengt	i as pei		
ACI318 M-08 Appendix D clause			2 and 2x2" after) a	a offootive		D.5.2.9 & D.6.2.9
4. For tie reinft, only the top most 2	,		,			A22.2.04 (F2040)
5. Strut-and-Tie model is used to ar	•					A23.3-04 (R2010)
6. For anchor group subject to mom	•		· · ·			D.4.1
and there is no redistribution of the		0 ,				
7. For anchor tensile force calc in a	0 ,					
resultant is at the outside edge of	•	•	base plate exhibits	rigid-body		
rotation. This simplified approach	•	•				
Shear carried by only half of total	anchor bolts due to	oversized	holes in column ba	ise plate		AISC Design Guide 1
						section 3.5.3
Anchor Bolt Data				05.0	n : 63	
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	$V_u = 111.2$	[kN]	$V_u = 0$ if shear key	is provided		
+	→	- +	—	_ + +	+ -	
					· []	
SS	P SS I ◀		\[\sigma_{\sigma} \]	•	─	-
	▶	<u> </u>		_ •"•	— ⊔ — — —	_
		т		1		
S1	,	S ₁		<u> </u>	S1 ,	
$S_{tb}=0$	İ	Stb		K-Stb		
<u></u>		K d	\longrightarrow	 	d	
Nu		Nu		Nu		
Mu			Mu		Mull	
Vu →			<u>► </u>	_ \	/ u 🕌	\neg
	<u> </u>	T _r			T3	_
- V	T 1	: ا ` ` ر ا	2	- V - 1 1 2		
T1 *	''i	S _{b1}	S _{b1}	T1 S _{b1}	Sb2 Sb1	

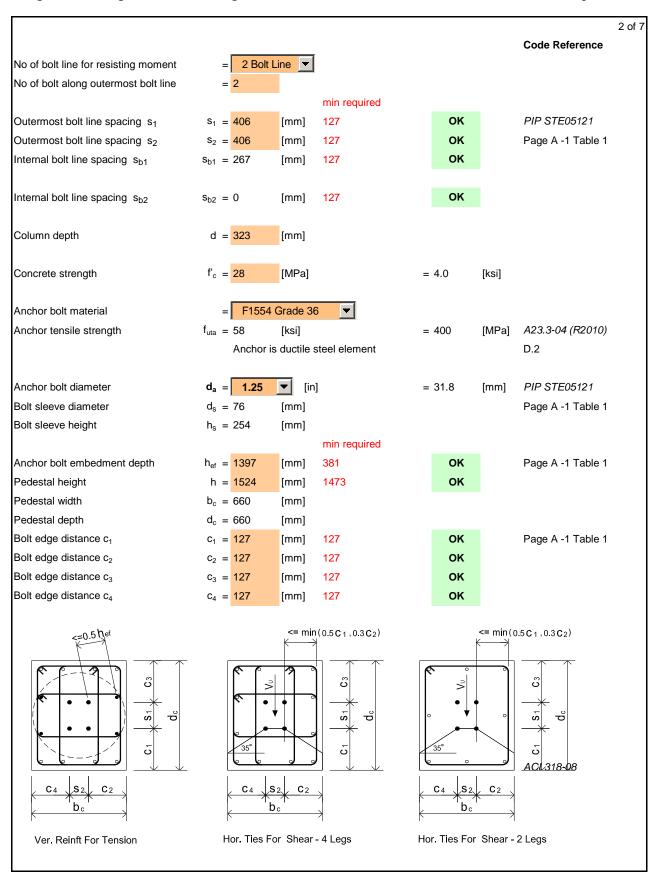
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4 BOLT LINE

3 BOLT LINE

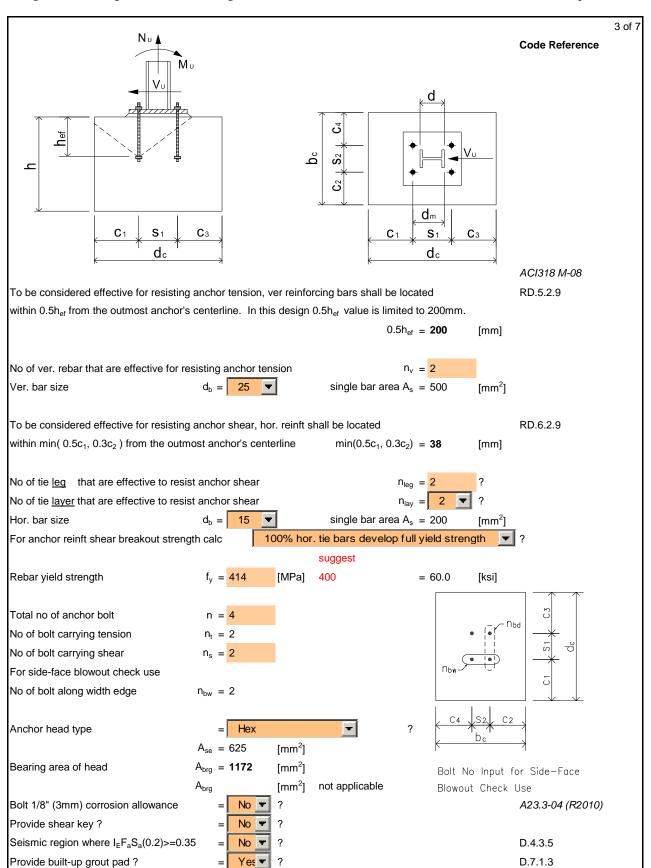
2 BOLT LINE

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							Code Reference	4 01 7
Strength reduction factors							A23.3-04 (R2010)	
Anchor reinforcement factor	φ -	= 0.75					D.7.2.9	
Steel anchor resistance factor		0.85					8.4.3 (a)	
Concrete resistance factor	φ _c =	= 0.65					8.4.2	
Resistance modification factors	В	0.00		Р	0.75		D 5 4(-)	
Anchor rod - ductile steel		= 0.80		R _{v,s} =			D.5.4(a)	
Concrete - condition A	K _{t,c} =	= 1.15		$R_{v,c} =$: 1.15		D.5.4(c)	
CONCLUSION		5 :-1				OK		
Abchor Rod Embedment, Spacing					0.05	OK	1001	
Min Rquired Anchor Reinft. Develo	opment Len	gth		ratio =		OK	12.2.1	
Overall				ratio =	∈ 0.90	OK		
Tension								
Anchor Rod Tensile Resistance				ratio =		OK		
Anchor Reinft Tensile Breakout Re	esistance			ratio =		OK		
Anchor Pullout Resistance				ratio =	0.33	ОК		
Side Blowout Resistance	_			ratio =	0.32	ок		
Shear								
Anchor Rod Shear Resistance				ratio =	0.73	OK		
Anchor Reinft Shear Breakout Res	sistance							
Strut Bearing Strength				ratio =	0.52	ок		
Tie Reinforcement				ratio =	0.45	ок		
Conc. Pryout Not Govern When he	= 12d _a					ок		
Anchor Rod on Conc Bearing				ratio =	0.27	ок		
Tension Shear Interaction								
Tension Shear Interaction				ratio =	0.90	ок		
Ductility							A23.3-04 (R2010)	
	Tension	Non-du	ctile	Shear	Ductile			
Seismic Design Requirement						ОК	D.4.3.6	
leFaSa(0.2)<0.35, A23.3-04 D.4.3	.3 ductility r	equiremer	nt 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			
		= 0.0	[kN]	No of bolt for $T_2 n_{T2} =$				
		= 0.0	[kN]	No of bolt for $T_3 n_{T3} =$				
Sum of bolt tensile force		$= \sum n_i T_i$: 110.3	[kN]		
Anchor Rod Tensile	N _{sr} =	= A _{se} ϕ_s f _{ut}	a R _{t,s}	=	= 170.0	[kN]	D.6.1.2 (D-3)	
Resistance	ratio =	= 0.32		>	- T ₁	ок		
Anchor Reinft Tensile Breakout	Resistance)						
Min topoion dovolonment longth	I _d =	=		=	= 887	[mm]	12.2.3	
Min tension development length	·n -							

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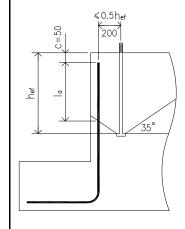
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Code Reference Actual development lenngth $l_a = h_{ef} - c (50mm) - 200mm \times tan 35$ = 1207 A23.3-04 (R2010) [mm]

> OK 12.2.1 > 300

> > OK

[mm]



$$N_{rbr} = \phi_{as} x f_y x n_v x A_s x (l_a / l_d, if l_a < l_d)$$
 = 310.5 [kN] 12.2.5
= x 1.0 not applicable = 310.5 [kN] D.4.3.5

ratio = 0.36 $> N_u$

Anchor Pullout Resistance

Seismic design strength reduction

Single bolt pullout resistance
$$N_{pr} = 8 A_{brg} \phi_c f_c^{T} R_{t,c} = 168.2 [kN] D.6.3.4 (D-16)$$

 $N_{cpr} = \Psi_{c,p} N_{pr} = 168.2 [kN] D.6.3.1 (D-15)$

$$\Psi_{c,p} = 1$$
 for cracked conc D.6.3.6
 $R_{t,c} = 1.00$ pullout strength is always Condition B D.5.4(c)

 $s = s_2 = 406$

Side Blowout Resistance

Check if edge anchors work as a

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]

 $S_{22} = 406$

Check if side blowout applicable
$$h_{ef} = 1397$$
 [mm] A23.3-04 (R2010)

$$> 2.5c$$
 side bowout is applicable D.6.4.1 theck if edge anchors work as a $s_{22} = 406$ [mm] $s = s_2 = 406$ [mm]

Single anchor SB resistance
$$N_{\text{sbr,w}} = 13.3 \text{c} \sqrt{A_{\text{brg}}} \phi_{\text{c}} \sqrt{f'_{\text{c}}} R_{\text{t,c}} = 227.1 \text{ [kN]} D.6.4.1 (D-18)$$

Single affelior of resistance
$$N_{\rm SDr,W} = 10.50 \sqrt{N_{\rm brg}} \, \varphi_{\rm c} \, N_{\rm t,c} = 227.11 \, [{\rm RN}] \, D.0.4.1 \, [D.10]$$

Multiple anchors SB resistance $N_{sbgr,w} =$

work as a group - applicable =
$$(1+s/6c) \times N_{sbr,w}$$
 = 348.1 [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

ratio =
$$0.32$$
 > N_{buw} **OK**

Group side blowout resistance
$$N_{sbgr} = \frac{N_{sbgr,w}}{n_{bw}} n_t$$
 = 348.1 [kN]

Govern Tensile Resistance
$$N_r = min (n_t N_{sr}, N_{rbr}, n_t N_{cpr}, N_{sbgr}) = 310.5 [kN]$$

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

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

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

Resistance

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

ratio = 0.73 > V_u

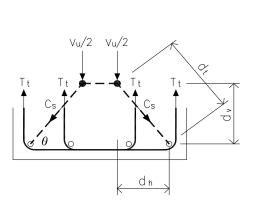
ΟK

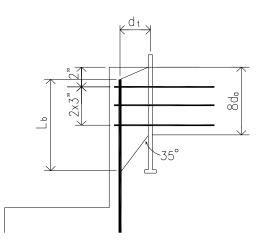
Anchor Reinft Shear Breakout Resistance

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 $\phi_{st} = 0.75$ 9.3.2.6





Strut-and-Tie model geometry

 $d_v = 57$ [mm]

 $d_h = 57$ [mm]

θ = 45

 $d_t = 81$

[mm]

[kN]

Strut compression force

 $C_s = 0.5 V_u / \sin\theta$

= 78.6

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 $I_e = min(8d_a, h_{ef})$

= 254

[mm] D.6.2.2

Anchor bearing area
Anchor bearing resistance

 $A_{brg} = I_e x d_a$ $C_r = n_s x \phi_{st} x f_{ce} x A_{brg}$

= 8065 = 283.8

 $> V_u$

[mm²] [kN]

OK

[mm²]

* Bearing of ver reinft bar

Ver bar bearing resistance

Ver bar bearing area $A_{brg} \ = \ (I_e + 1.5 \ x \ d_t - d_a/2 \ - d_b/2) \ x \ d_b$

= 8664

 $C_r = \phi_{st} \times f_{ce} \times A_{brq}$

= 152.4

.4 [kN]

ratio = 0.52

> C_s

ΟK

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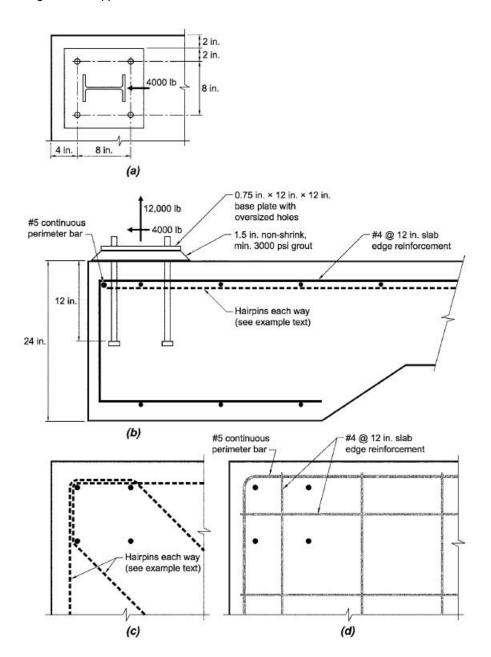
				Code Reference
Tie Reinforcement				
	3 layers of ties (2" from TOC and 2x3" after) a		. , .	
	he tie cannot develop full yield strength f _y . Use	·		
	3.3-04 Annex D Eq. (D-17) as the max force cal	n be developed	at hook	h
* Assume 100% of hor. tie bars can	develop full yield strength.			100 0 04 (50040)
Tatal accept as of boatis has	a a (log) v a (lover)	4		A23.3-04 (R2010)
Total number of hor tie bar	$n = n_{leg} (leg) \times n_{lay} (layer)$	= 4	FI A 17	D 0 0 5 (D 47)
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	$V_{rbr} = 1.0 \times n \times Tr$	= 248.4	[kN]	
Seismic design strength reduction	= x 1.0 not applicable	= 248.4	[kN]	D.4.3.5
	ratio = 0.45	$> V_u$	ок	
Conc. Pryout Shear Resistance				
The pryout failure is only critical for s	short and stiff anchors. It is reasonable to assur	ne that for gene	ral	
cast-in place headed anchors with h	$_{ef}$ > = 12 d_a , the pryout failure will not govern			
	$12d_a = 381$ [mm]	$h_{ef} = 1397$	[mm]	
		> 12d _a	ОК	CSA S16-09
Anchor Rod on Conc Bearing	$B_r = n_s x 1.4 x \phi_c x min(8d_a, h_{ef}) x d_a x f_c'$	= 405.1	[kN]	25.3.3.2
	ratio = 0.27	$< V_u$	ОК	
Govern Shear Resistance	$V_r = min (V_{sr}, V_{rbr}, B_r)$	= 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	ОК	
Ductility Tension	$N_{sr} = 170.0$ [kN]			
	> min (N _{rbr} , N _{cpr} , N _{sbqr})	= 168.2	[kN]	
	4			
	Non-ductile			
Ductility Shear	$V_{sr} = 153.0$ [kN]			
Ductility Shear		= 248.4	[kN]	

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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



 $N_u = 12 \text{ kips (tension)},$

V_u=4 kips,

 $f_c' = 3 \text{ ksi}$

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

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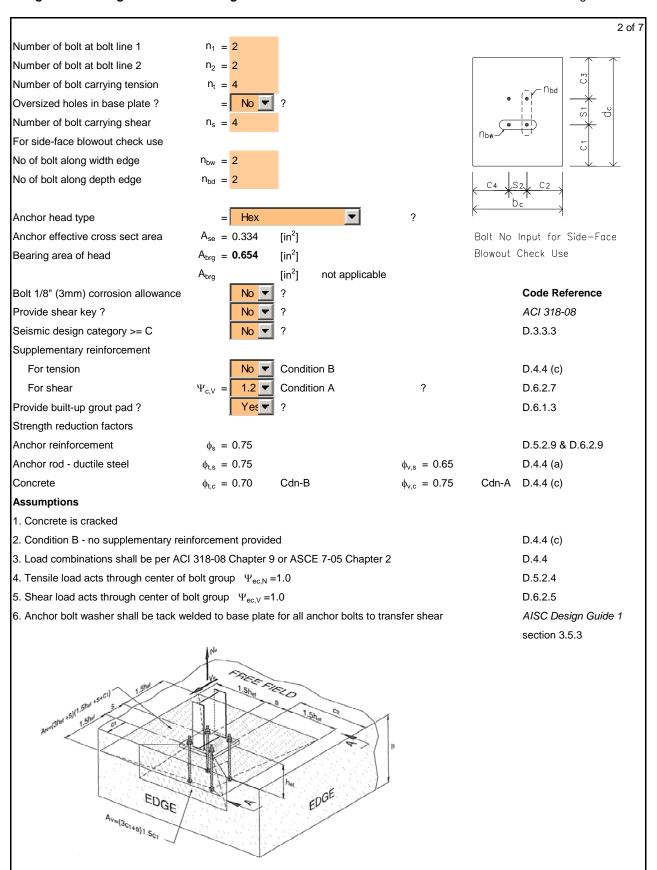
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CONCLUSION						
Abchor Rod Embedment, Spacir	ng and Edg	je Distance			Warn	
Overall			ratio	= 0.83	OK	
Tension						
Anchor Rod Tensile Resistance	•		ratio	= 0.16	ОК	
Conc. Tensile Breakout Resistance	е		ratio	= 0.58	ок	
Anchor Pullout Resistance			ratio	= 0.27	ок	
Side Blowout Resistance			ratio	= 0.23	ок	
Shear						
Anchor Rod Shear Resistance			ratio	= 0.13	ОК	
Conc. Shear Breakout Resistance			ratio	= 0.41	ОК	
Conc. Pryout Shear Resistance			ratio	= 0.10	ОК	
Tension Shear Interaction						
Tension Shear Interaction			ratio	= 0.83	OK	
Ductility						
	Tension	Non-ductile	Shear	Non-duct	tile	
Seismic Design Requirement					OK	D.3.3.4
	SDC< C, A	ACI318-08 D.3.3 duc	tility requirement is N	OT required		
CALCULATION						Code Reference
						ACI 318-08
Anchor Rod Tensile	$\phi_{t,s} N_{sa} =$	$= \phi_{t,s} n_t A_{se} f_{uta}$		= 75.2	[kips]	D.5.1.2 (D-3)
Resistance	ratio =	= 0.16		> N _u	ОК	
Conc. Tensile Breakout Resistar						
	N _b =	$= 24 \lambda \sqrt{f_c'} h_{ef}^{1.5}$ if h	$_{\rm ef}$ < 11" or $h_{\rm ef}$ > 25"	= 55.1	[kips]	D.5.2.2 (D-7)
		16 $\lambda \sqrt{f_c'} h_{ef}^{5/3}$ if 1	$1" \le h_{ef} \le 25"$			D.5.2.2 (D-8)
Projected conc failure area	1.5h _{ef} =	=		= 18.00	[in]	
	A _{Nc} =		$(c_3, 1.5h_{ef})$ min($(c_3, 1.5h_{ef})$]x	= 900.0	[in ²]	
	A _{Nco} =	= 9 h _{ef} ²	(+/ 61/3	= 1296.0	[in ²]	D.5.2.1 (D-6)
		= min (A _{Nc} , n _t A _{Nco})		= 900.0	[in ²]	D.5.2.1
Min edge distance		= min(c ₁ , c ₂ , c ₃ , c ₄)		= 4.0	[in]	
Eccentricity effects		= 1.0 for no eccentric	cload			D.5.2.4
Edge effects		= min[(0.7+0.3c _{min} /1		= 0.77		D.5.2.5
Concrete cracking		= 1.0 for cracked cor				D.5.2.6
Concrete splitting		= 1.0 for cast-in anch				D.5.2.7

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						Code Reference	. 0
						ACI 318-08	
Concrete breakout resistance	$\phi_{t,c} N_{cbg} =$	$= \phi_{t,c} \frac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi$	$_{\rm ed,N}~\Psi_{\rm c,N}~\Psi_{\rm cp,N}~N_{\rm b}$	= 20.5	[kips]	D.5.2.1 (D-5)	
Seismic design strength reduction	=	x 1.0 not application	able	= 20.5	[kips]	D.3.3.3	
	ratio =	= 0.58		$> N_u$	ОК		
Anchor Pullout Resistance							
Single bolt pullout resistance	N _p =	= 8 A _{brg} f _c '		= 15.7	[kips]	D.5.3.4 (D-15)	
	φ _{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 applica	able	= 43.9	[kips]	D.3.3.3	
	ratio =	= 0.27		> N _u	OK		
	Ψ _{c.p} =	= 1 for cracked con	С			D.5.3.6	
			strength is always	Condition B		D.4.4(c)	
Side Blowout Resistance							
Failure Along Pedestal Width Edge	<u> </u>						
Tensile load carried by anchors clo		which may cause s	side-face blowout				
along pedestal width edge	_	= N _u x n _{bw} / n _t		= 6.0	[kips]	RD.5.4.2	
3, 111111111111111111111111111111111111		= min (c ₁ , c ₃)		= 4.0	[in]	-	
Check if side blowout applicable		= 12.0 [in]		- 1.0	[]		
officer if side blowdat applicable		- 12.0 [iii] - 2.5c	side bowout is ap	nlicable		D.5.4.1	
Check if edge anchors work as a		= 8.0 [in]		$= S_2 = 8.0$	[in]	D.3.4.1	
a group or work individually		= 6.0 [] < 6c	edge anchors wo		[]	D.5.4.2	
		$= \phi_{t,c} \left(160 c \sqrt{A_{brg}} \right).$		= 19.8	[kips]	D.5.4.1 (D-17)	
		•	λ √1 c	- 13.0	[κιρο]	D.3.4.1 (D-11)	
Multiple anchors SB resistance	$\phi_{t,c}N_{sbg,w} =$			00.5	Diam's	D 5 4 0 (D 40)	
work as a group - applicable		= $(1+s/6c) \times \phi_{t,c} N_{s}$		= 26.5	[kips]	D.5.4.2 (D-18)	
work individually - not applicable		= n _{bw} x φ _{t,c} N _{sb} x [1+			[kips]	D.5.4.1	
Seismic design strength reduction		x 1.0 not applica	able	= 26.5	[kips]	D.3.3.3	
	ratio =	= 0.23		$> N_{buw}$	OK		
Failure Along Pedestal Depth Edge	=						
Tensile load carried by anchors clo	-	•	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 ap	plicable		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 wo	rk as a group		D.5.4.2	
Single anchor SB resistance	$\phi_{t,c} N_{sb} =$	$= \phi_{\rm t,c} \left(160 \rm c \sqrt{A_{\rm brg}} \right).$	λ √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) x φ _{t,c} N _s	b	= 26.5	[kips]	D.5.4.2 (D-18)	
work individually - not applicable	· =	= n _{bd} x φ _{t,c} N _{sb} x [1+	(c ₁ or c ₃) / c] / 4	= 0.0	[kips]	D.5.4.1	
Seismic design strength reduction		= x 1.0 not applica		= 26.5	[kips]	D.3.3.3	
	ratio =			> N _{bud}	ОК		

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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 $N_r = min [\phi_{t,s} N_{sa}, \phi_{t,c} (N_{cbg}, N_{pn}, N_{sbg})] = 20.5$ [kips]

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

Anchor Rod Shear $\phi_{v,s} V_{sa} = \phi_{v,s} n_s 0.6 A_{se} f_{uta} = 39.1$ [kips] D.6.1.2 (b) (D-20)

Resistance

Reduction due to built-up grout pads = x 0.8 , applicable = 31.3 [kips] D.6.1.3

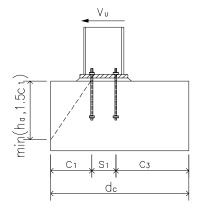
ratio = 0.13 > V_u

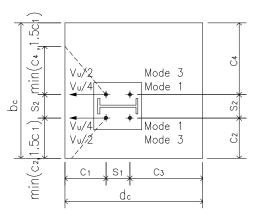
OK

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 $c_1 = ca1$ needs NOT to be adjusted = 4.0 [in] D.6.2.4

 $c_2 = = 4.0$ [in]

 $1.5c_1 = 6.0$ [in]

 $A_{Vc} = [min(c_2, 1.5c_1) + s_2 + min(c_4, 1.5c_1)] x = 108.0 [in^2] D.6.2.1$

 $min(1.5c_1, h_a)$

 $A_{Vco} = 4.5c_1^2$ = 72.0 [in²] D.6.2.1 (D-23)

 $A_{Vc} = min (A_{Vc}, n_1 A_{Vco})$ = 108.0 [in²] D.6.2.1

 $l_e = min(8d_a, h_{ef})$ = 6.0 [in] D.6.2.2

 $V_b = \left[7 \left(\frac{I_e}{d_a} \right)^{0.2} \sqrt{d_a} \right] \lambda \sqrt{f_e'} c_1^{1.5}$ = 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

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Code Reference

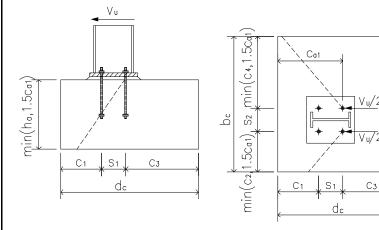
ACI 318-08

Conc shear breakout

resistance
$$V_{cbg1} = \phi_{v,c} \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_b = 4.9$$
 [kips] D.6.2.1 (D-22)

Mode 1 is used for checking $V_{cbg1} = V_{cbg1}$ x 2.0 = 9.8 [kips]

Mode 2 Failure cone at back anchors



Code Reference

ACI 318-08

D.6.2.4

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

Bolt edge distance - adjusted $c_{a1} = ca1$ needs NOT to be adjusted = 12.0 [in] D.6.2.4

$$c_2 = 4.0$$
 [in]

$$1.5c_{a1} = 18.0$$
 [in]

$$A_{Vc} = [min(c_2, 1.5c_{a1}) + s_2 + min(c_4, 1.5c_{a1})] x = 540.0$$
 [in²] D.6.2.1

min(1.5c_{a1}, h_a)

$$A_{Vco} = 4.5c_{a1}^2$$
 = 648.0 [in²] D.6.2.1 (D-23)

$$A_{Vc} = min (A_{Vc}, n_2 A_{Vco})$$
 = 540.0 [in²] D.6.2.1

$$l_e = min(8d_a, h_{ef})$$
 = 6.0 [in] D.6.2.2

$$V_{b} = \left[7 \left(\frac{I_{e}}{d_{a}} \right)^{0.2} \sqrt{d_{a}} \right] \lambda \sqrt{f_{c}^{'}} c_{a1}^{1.5}$$
 = 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 $\Psi_{h,v} = \max[(\operatorname{sqrt}(1.5c_{a1} / h_a), 1.0] = 1.00$ D.6.2.8

Conc shear breakout

resistance
$$V_{cbg2} = \phi_{v,c} \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_b = 12.0$$
 [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**

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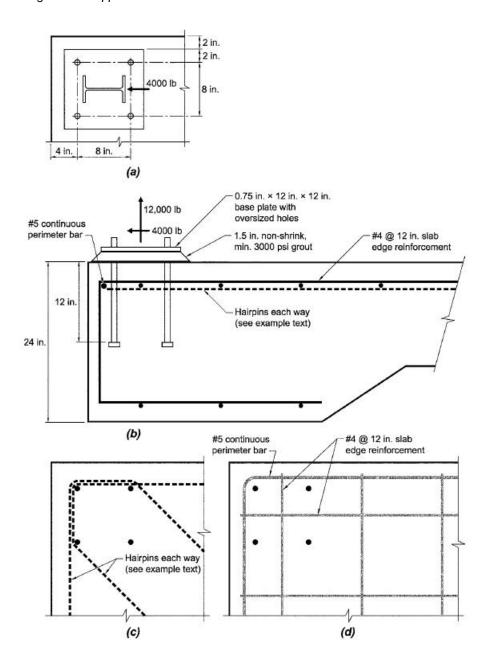
					Code Reference	7
Conc. Pryout Shear Resistance					ACI 318-08	
	k _{cp} =	2.0			D.6.3	
actored shear pryout resistance ϕ	v,c V _{cpg} =	$\phi_{v,c} \; k_{cp} \; N_{cbg}$	= 41.1	[kips]	D.6.3 (D-31)	
	φ _{v,c} =	0.70 pryout strength is alwa	ays Condition B		D.4.4(c)	
Seismic design strength reduction	=	x 1.0 not applicable	= 41.1	[kips]	D.3.3.3	
	ratio =	0.10	> Vu	ОК		
Govern Shear Resistance	V _r =	min [$\phi_{v,s}V_{sa}$, $\phi_{v,c}$ (V_{cbg} , V_{cpg})]	= 9.8	[kips]		
ension 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	
		$N_u/\phi N_n + V_u/\phi V_n$	= 0.99		D.7.3 (D-32)	
	ratio =	0.83	< 1.2	ок		
Ductility Tension						
	$\phi_{t,s} N_{sa} =$	75.2 [kips]				
	>	$\phi_{t,c} \text{ min } (N_{cbg}, \ N_{pn}, \ N_{sbg})$	= 20.5	[kips]		
		Non-ductile				
Ductility Shear						
	$\phi_{v,s} V_{sa} =$	31.3 [kips]				
	>	$\phi_{\text{v,c}} \min \left(V_{\text{cbg}}, V_{\text{cpg}} \right)$	= 9.8	[kips]		
		Non-ductile				

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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



 $N_u = 53.4 \text{ kN (tension)}, V_u=17.8 \text{ kN},$

 $f_{c}' = 20.7 \text{ MPa}$

Anchor bolt d_a=3/4 in ASTM F1554 Grade 55

 $h_{ef} = 305 mm$

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

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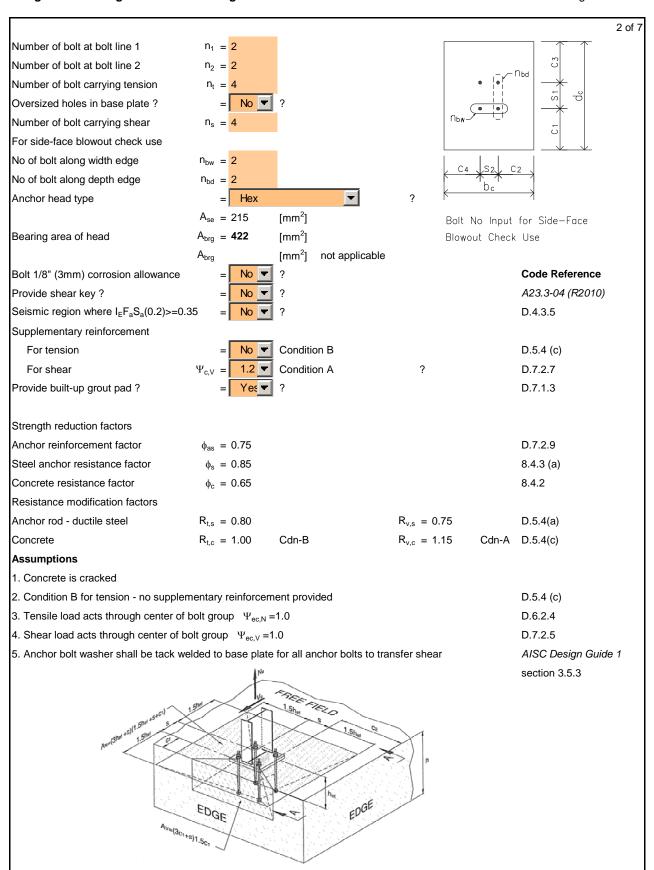


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						3
CONCLUSION						
Abchor Rod Embedment, Spaci	ng and Edo	ge Distance			Warn	
Overall			ratio	= 0.86	ок	
Tension						
Anchor Rod Tensile Resistance			ratio	= 0.18	ОК	
Conc. Tensile Breakout Resistand	e		ratio	= 0.62	ОК	
Anchor Pullout Resistance			ratio	= 0.29	ОК	
Side Blowout Resistance			ratio	= 0.24	ок	
Shear						
Anchor Rod Shear Resistance			ratio	= 0.13	OK	
Conc. Shear Breakout Resistance			ratio	= 0.41	ок	
Conc. Pryout Shear Resistance			ratio	= 0.10	ок	
Anchor Rod on Conc Bearing			ratio	= 0.08	ок	
Ç						
Tension Shear Interaction						
Tension Shear Interaction			ratio	= 0.86	ОК	
Ductility						
	Tension	Non-ductile	Shear	Non-duc	tile	
Seismic Design Requirement					ОК	D.4.3.6
leFaSa(0.2)<0.35, A23.3-04 D.4.3	.3 ductility r	requirement is NOT r	equired			
CALCULATION						Code Reference
						A23.3-04 (R2010)
Anchor Rod Tensile	N _{sr} =	= n _t A _{se} ϕ_s f _{uta} R _{t,s}		= 303.1	[kN]	D.6.1.2 (D-3)
Resistance	ratio =	= 0.18		> N _u	OK	
Conc. Tensile Breakout Resista	nce	10 (() 15 0	(25		
	N _{br} =		$f h_{ef} \le 275 \text{ or } h_{ef} \ge 63$	25		D.6.2.2 (D-7)
		$3.9 \ \phi_{\rm c} \ \sqrt{{\rm f}_{\rm c}^{'}} \ {\rm h}_{\rm ef}^{5/3} \ {\rm R}_{\rm t,c}$	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 ₀	$_{\rm ef}$)+min(c_3 ,1.5 $h_{\rm ef}$)]x	= 5.8E+05	[mm ²]	
		[s ₂ +min(c ₂ ,1.5h _{ef})+	min(c ₄ ,1.5h _{ef})]			
	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	$\Psi_{\text{ec,N}}$:	= 1.0 for no eccentric	load			D.6.2.4
Edge effects		= min[(0.7+0.3c _{min} /1		= 0.77		D.6.2.5
Concrete cracking		= 1.0 for cracked cor				D.6.2.6

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					Code Reference	
					A23.3-04 (R2010)	
Concrete breakout resistance	$N_{cbgr} = \frac{A_{Nc}}{A_{Nco}} \Psi$	$\Psi_{\text{ec},N} \; \Psi_{\text{ed},N} \; \Psi_{\text{c},N} \; \Psi_{\text{cp},N} \; \; N_{\text{br}}$	= 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	ОК		
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	ок		
	$\Psi_{c,p} = 1$ for cr	racked conc			D.6.3.6	
	$R_{t,c} = 1.00$		Condition B		D.5.4(c)	
Side Blowout Resistance						
Failure Along Pedestal Width Edge						
Tensile load carried by anchors clos	e to edge which m	nay cause side-face blowout			ACI318 M-08	
along pedestal width edge	$N_{buw} = N_u \times n_b$	_{ow} / n _t	= 26.7	[kN]	RD.5.4.2	
	c = min(c		= 102	[mm]		
Check if side blowout applicable	$h_{ef} = 305$				A23.3-04 (R2010)	
	> 2.5c		pplicable		D.6.4.1	
Check if edge anchors work as a	$s_{22} = 203$		$= s_2 = 203$	[mm]	2.0	
a group or work individually	< 6c	edge anchors w			D.6.4.2	
Single anchor SB resistance		$\sqrt{A_{brq}} \phi_c \sqrt{f'_c} R_{t,c}$	= 83.0	[kN]	D.6.4.1 (D-18)	
Multiple anchors SB resistance	$N_{\text{sbgr,w}} =$	V-brg rc V-ct,c			- (-,	
work as a group - applicable	= (1+s/6	ic) x Naha	= 110.5	[kN]	D.6.4.2 (D-19)	
work individually - not applicable		$I_{\text{sbr,w}} \times [1+(c_2 \text{ or } c_4)/c]/4$	= 0.0	[kN]	D.6.4.1	
Seismic design strength reduction		not applicable	= 110.5	[kN]	D.4.3.5	
Joeisinic design strength reduction	ratio = 0.24	пот аррисаые	> N _{buw}	OK	D.4.3.3	
Failure Along Pedestal Depth Edge	10110 = 0.24		> · •buw	O.K		
Tensile load carried by anchors clos	e to edge which m	nay cause side-face blowout			ACI318 M-08	
along pedestal depth edge	$N_{bud} = N_u \times n_b$		= 26.7	[kN]	RD.5.4.2	
	c = min(c		= 102	[mm]		
Check if side blowout applicable	$h_{\rm ef} = 305$	[mm]			A23.3-04 (R2010)	
oneen ii eide bienedt approable	> 2.5c	side bowout is a	innlicable		D.6.4.1	
Check if edge anchors work as a	$s_{11} = 203$		$= s_1 = 203$	[mm]		
a group or work individually	< 6c	edge anchors w	•	[]	D.6.4.2	
Single anchor SB resistance		$\sqrt{A_{brg}} \; \phi_{c} \; \sqrt{f'_{c}} \; R_{t,c}$	= 83.0	[kN]	D.6.4.1 (D-18)	
Multiple anchors SB resistance	$N_{\text{sbgr,d}} = 13.30$	V ``brg ♥c V ' c '`t,c	- 00.0	[141.4]	5.0. 4 .1 (D-10)	
·		c) v d N	= 110.5	[FVII	D 6 4 2 (D 10)	
work as a group - applicable		Sc) $x \phi_{t,c} N_{sbr,d}$		[kN]	D.6.4.2 (D-19)	
work individually - not applicable		$l_{\text{sbr,d}} \times [1+(c_1 \text{ or } c_3)/c]/4$	= 0.0	[kN]	D.6.4.1	
Seismic design strength reduction		not applicable	= 110.5	[kN]	D.4.3.5	
	ratio = 0.24		$> N_{bud}$	OK		

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Code Reference

OK

Group side blowout resistance $N_{\text{sbgr}} = \min \left(\frac{N_{\text{sbgr},w}}{n_{\text{bw}}} n_{\text{t}}, \frac{N_{\text{sbgr},d}}{n_{\text{bd}}} n_{\text{t}} \right) = 221.1 \quad \text{[kN]} \quad A23.3-04 \, (R2010)$

Govern Tensile Resistance $N_r = min(N_{sr}, N_{rbr}, N_{cpr}, N_{sbgr}) = 85.5$ [kN]

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

Anchor Rod Shear $V_{sr} = n_s A_{se} \phi_s 0.6 f_{uta} R_{v,s} = 170.5 [kN] D.7.1.2 (b) (D-21)$

Resistance

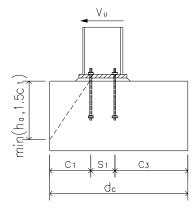
Reduction due to built-up grout pads $= x \cdot 0.8$, applicable = 136.4 [kN] D.7.1.3

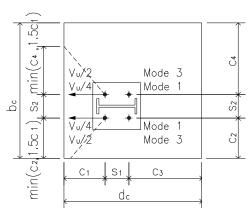
ratio = 0.13 > V_u

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 $c_1 = ca1$ needs NOT to be adjusted = 102 [mm] D.7.2.4

 $c_2 = = 102$ [mm]

 $1.5c_1 = = 153$ [mm]

 $A_{Vc} = [min(c_2, 1.5c_1) + s_2 + min(c_4, 1.5c_1)] x = 7.0E+04 [mm^2] D.7.2.1$

 $min(1.5c_1, h_a)$

 $A_{Vco} = 4.5c_1^2$ = 4.7E+04 [mm²] D.7.2.1 (D-24)

 $A_{VC} = min (A_{VC}, n_1 A_{VCO})$ = 7.0E+04 [mm²] D.7.2.1

 $l_e = min(8d_a, h_{ef})$ = 152 [mm] D.3

 $V_{br} = 0.58 \left(\frac{I_e}{d_a} \right)^{0.2} \sqrt{d_a} \phi_c \sqrt{f_c} c_{a1}^{1.5} R_{v,c} = 13.5$ [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 $\Psi_{\text{ed,v}} = \min[(0.7 + 0.3c_2/1.5c_1), 1.0] = 0.90$ D.7.2.6 Concrete cracking $\Psi_{\text{c,v}} = = 1.20$ D.7.2.7

Member thickness $\Psi_{h,v} = max[(sqrt(1.5c_1 / h_a), 1.0] = 1.00$ D.7.2.8

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Code Reference

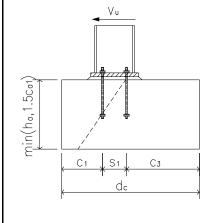
Conc shear breakout

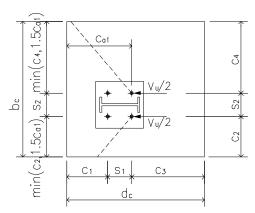
resistance

$$V_{cbgr1} = \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_{br}$$
 = 21.9 [kN] D.7.2.1 (D-23)

 $V_{cbgr1} = V_{cbg1}$ x 2.0 Mode 1 is used for checking [kN]

Mode 2 Failure cone at back anchors





A23.3-04 (R2010)

Bolt edge distance

$$c_{a1} = c_1 + s_1$$
 = 305 [mm]

Limiting ca1 when anchors are influenced by 3 or more edges = No

D.7.2.4 D.7.2.4

Bolt edge distance - adjusted

$$c_{a1}$$
 = ca1 needs NOT to be adjusted = **305**

102

= 152

= 0.77

= 1.20

= 1.00

 $C_2 =$

$$A_{Vc} = [min(c_2, 1.5c_{a1}) + s_2 + min(c_4, 1.5c_{a1})] x = 3.5E+05 [mm^2]$$

D.7.2.1

[mm]

$$min(1.5c_{a1}, h_a)$$

$$A_{Vco} = 4.5c_{a1}^2$$
 = 4.2E+05 [mm²] D.7.2.1 (D-24)

$$A_{Vc} = min (A_{Vc}, n_2 A_{Vco})$$

$$= 3.5E+05 \text{ [mm}^2\text{]} D.7.2.1$$

$$l_e = min(8d_a, h_{ef})$$

$$V_{br} = 0.58 \left(\frac{I_e}{d_a}\right)^{0.2} \sqrt{d_a} \phi_c \sqrt{f_c^{\cdot}} c_{a1}^{1.5} R_{v,c}$$
 = 70.0

Eccentricity effects
$$\Psi_{ec,v} = 1.0$$
 shear acts through center of group

Edge effects

$$\Psi_{\text{ed,v}} = \text{min}[(0.7+0.3c_2/1.5c_{a1}), 1.0]$$

Concrete cracking

D.7.2.2 (D-25)

Member thickness

resistance

$$\Psi_{h,v} = max[(sqrt(1.5c_{a1} / h_a), 1.0]$$

Conc shear breakout

$$V_{cbgr2} = \frac{A_{vc}}{A_{vco}} \Psi_{ec,v} \Psi_{ed,v} \Psi_{c,v} \Psi_{h,v} V_{br}$$

Min shear breakout resistance

$$V_{cbgr} = min (V_{cbgr1}, V_{cbgr2})$$

$$= 43.8$$
 [kN]

OK

Seismic design strength reduction

$$ratio = 0.41$$

$$> V_u$$

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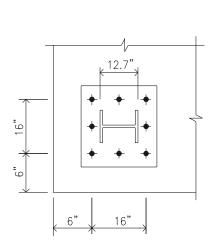
			Code Reference	
			A23.3-04 (R2010)	
$k_{cp} = 2.0$			D.7.3	
$V_{cpgr} = k_{cp} N_{cbgr}$	= 171.0	[kN]	D.7.3 (D-32)	
$R_{v,c} = 1.00$ pryout strength is alway	s Condition B		D.5.4(c)	
= x 1.0 not applicable	= 171.0	[kN]	D.4.3.5	
			2.1.0.0	
	, u			
			CSA S16-09	
$B_r = n_s \times 1.4 \times \phi_c \times min(8d_a, h_{ef}) \times d_a \times f_c$	= 221.9	[kN]	25.3.3.2	
ratio = 0.08	$> V_u$	ОК		
$V_r = min(V_{sr}, V_{cbgr}, V_{cpgr}, B_r)$	= 43.8	[kN]		
			A23.3-04 (R2010)	
V.			D 0 0 0 D 0 0	
	4.00			
		OK	ט.צ.4 (D-35) ■	
1411U = U.0U	< 1.2	UK		
$N_{sr} = 303.1$ [kN]				
	= 85.5	[kN]		
Non-ductile				
$V_{sr} = 136.4$ [kN]				
> min (V _{cbgr} , V _{cpgr} , B _r)	= 43.8	[kN]		
Non-ductile				
	$\begin{split} &V_{cpgr} = k_{cp} N_{cbgr} \\ &R_{v,c} = 1.00 \qquad \text{pryout strength is alway} \\ &= x 1.0 \text{not applicable} \\ &\text{ratio} = 0.10 \\ \\ &B_r = n_s x 1.4 x \varphi_c x \text{min}(8d_a, h_{ef}) x d_a x f_r \\ &\text{ratio} = 0.08 \\ \\ &V_r = \min \left(V_{sr}, V_{cbgr}, V_{cpgr}, B_r \right) \\ &\qquad \qquad $	$\begin{array}{llllllllllllllllllllllllllllllllllll$	$\begin{array}{llllllllllllllllllllllllllllllllllll$	$ \begin{array}{c} k_{cp} = 2.0 \\ V_{cpgr} = k_{cp} N_{cbgr} \\ R_{v,c} = 1.00 \\ \end{array} \begin{array}{c} \text{pryout strength is always Condition B} \\ \end{array} \begin{array}{c} D.7.3 \\ D.7.4 \\ $

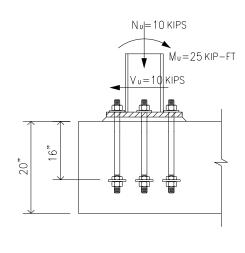
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Example 13: Anchor Bolt + No Anchor Reinft + Tension Shear & Moment + ACI 318-08 Code





 $M_u = 25 \text{ kip-ft}$ $N_u = 10 \text{ kips}$ (Compression) $V_u = 10 \text{ kips}$

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

Provide built-up grout pad

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ANCHOR BOLT DESIGN Combined Tension, Shear and Moment

Anchor bolt design based on

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

PIP STE05121 Anchor Bolt Design Guide-2006

Code Abbreviation

ACI 318-08

PIP STE05121

Assumptions Code Reference

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

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

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

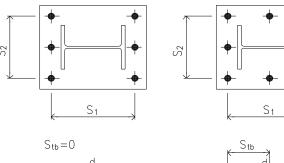
AISC Design Guide 1

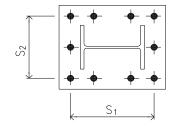
section 3.5.3

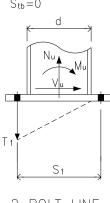
Anchor Bolt Data

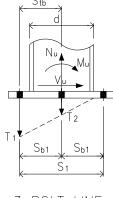
Factored moment $M_u = 25.0$ [kip-ft] = 33.9[kNm] Factored tension /compression $N_u = -10.0$ [kips] in compression = -44.5 [kN] $V_u = 10.0$ Factored shear [kips] = 44.5 [kN]

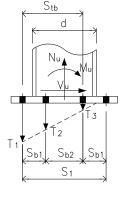
Factored shear for bolt design $V_u = 10.0$ [kips] $V_u = 0$ if shear key is provided











2 BOLT LINE

3 BOLT LINE

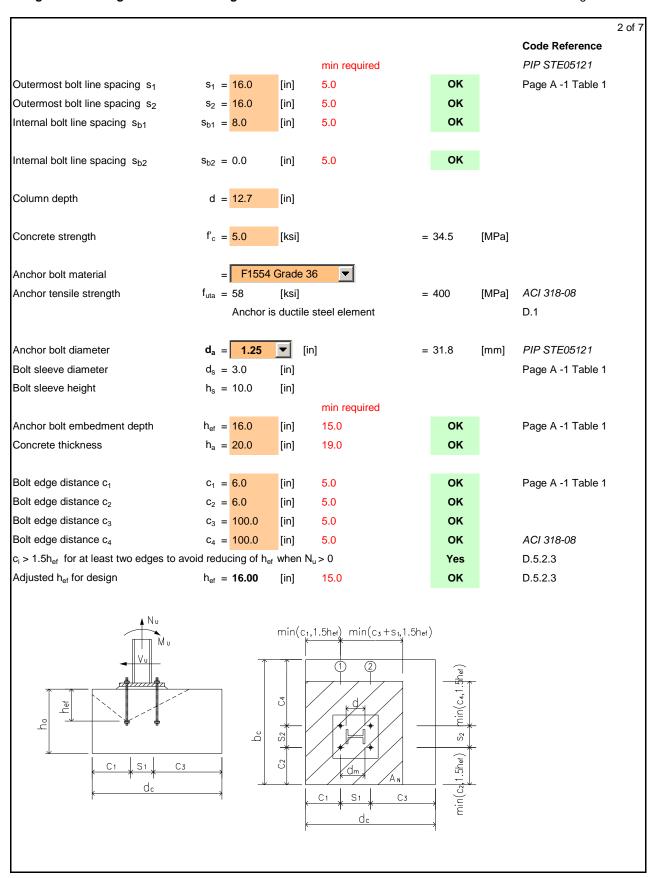
4 BOLT LINE

No of bolt line for resisting moment
No of bolt along outermost bolt line

= 3 Bolt Line = 3

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Number of bolt at bolt line 1	n ₁ =							Code Reference	
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	T	?				
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 -	Conditio	n A				D.4.4 (c)	
For shear	Ψ _{c,V} =		Conditio		?			D.6.2.7	
Provide built-up grout pad?	C, V	Yes -	?					D.6.1.3	
Torrac same ap grout pau i			•					2.66	
Strength reduction factors									
Anchor reinforcement	$\phi_s =$	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		φ _{v,c} =	0.75	Cdn-A	D.4.4 (c)	
CONCLUSION									
Abchor Rod Embedment, Spacing	and Edg	e Distance	е				OK		
Overall					ratio =	0.81	OK		
Гension									
Anchor Rod Tensile Resistance					ratio =	0.12	ок		
Conc. Tensile Breakout Resistance					ratio =	0.39	ОК		
Anchor Pullout Resistance					ratio =	0.08	ОК		
Side Blowout Resistance					ratio =	0.13	ОК		
Shear									
Anchor Rod Shear Resistance					ratio =	0.14	ОК		
Conc. Shear Breakout Resistance					ratio =	0.58	ок		
Conc. Pryout Shear Resistance					ratio =	0.11	ОК		
Fension Shear Interaction									
Tension Shear Interaction					ratio =	0.81	OK		
Ductility								ACI 318-08	
T	ension	Ductile			Shear	Non-duct	tile		
Seismic Design Requirement							ок	D.3.3.4	
	SDC< C A	CI318-08	D 3 3 due	tility requireme	ent is N∩	T required			

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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	$I_{T1} = 3$			
	$T_2 = 2.15$	[kips] No of bolt for T ₂ n	$I_{T2} = 2$			
	$T_3 = 0.00$	[kips] No of bolt for T ₃ n	$I_{T3} = 0$			
Sum of bolt tensile force	$\mathbf{N_u} = \sum \mathbf{n_i} \mathbf{T_i}$		= 18.9	[kips]		
Tensile bolts outer distance s _{tb}	$s_{tb} = 8.0$	[in]				
Eccentricity e' _N distance betwe	een resultant of te	nsile load and centroid of a	nchors			
loaded in tension	$e'_{N} = 1.38$	[in]			Fig. RD.5.2.4 (b)	
Eccentricity modification factor	$\Psi_{ec,N} = \frac{1}{\left(1 + \frac{2i}{3i}\right)}$	$\frac{\dot{\theta_N}}{\dot{\theta}}$	= 0.95		D.5.2.4 (D-9)	
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 = 0.12	·uta	> T ₁	OK	2.62 (2.6)	
	1440 = 0.12		7 1			
Conc. Tensile Breakout Resista						
	$N_b = 24 \lambda v$	$f_c^{'}$ $h_{ef}^{1.5}$ if h_{ef} < 11" or h_{ef} > 25'	= 114.9	[kips]	D.5.2.2 (D-7)	
	16 2 √	$\overline{f_c'} h_{ef}^{5/3} \text{ if } 11" \le h_{ef} \le 25"$			D.5.2.2 (D-8)	
Projected conc failure area	1.5h _{ef} =		= 24.00	[in]		
		$n(c_1, 1.5h_{ef}) + min(c_3, 1.5h_{ef})]x$ $n(c_2, 1.5h_{ef}) + min(c_4, 1.5h_{ef})]$	= 1748.0	[in ²]		
	$A_{Nco} = 9 h_{ef}^{2}$	(2) (1) (4) (1)2	= 2304.0	[in ²]	D.5.2.1 (D-6)	
	$A_{Nc} = min (A$	$n_{t,N_{C}}$, n_{t} $A_{N_{CO}}$)	= 1748.0	[in ²]	D.5.2.1	
Min edge distance	$c_{min} = min(c_1)$		= 6.0	[in]	-	
Eccentricity effects	Ψ _{ec,N} =	, 2, 3, 4,	= 0.95		D.5.2.4 (D-9)	
Edge effects		.7+0.3c _{min} /1.5h _{ef}), 1.0]	= 0.78		D.5.2.5	
Concrete cracking		cracked concrete			D.5.2.6	
Concrete splitting	$\Psi_{cp,N} = 1.0 \text{ for}$				D.5.2.7	
Concrete breakout resistance	$\phi_{t,c} N_{cbg} = \phi_{t,c} \frac{A}{A}$	$\frac{Nc}{Nc}$ $\Psi_{ec,N}$ $\Psi_{ed,N}$ $\Psi_{c,N}$ $\Psi_{cp,N}$ 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)	
	$\phi_{t,c} N_{pn} = \phi_{t,c} \Psi_{c}$	$_{p,p}N_p$	= 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	ОК		
	$\Psi_{c,p} = 1$ for co	acked conc			D.5.3.6	
	$\phi_{t,c} = 0.70$	pullout strength is always C	ondition B		D.4.4(c)	

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Side Blowout Resistance				Code Reference	
Failure Along Pedestal Width Edge	<u>e</u>			ACI 318-08	
Tensile load carried by anchors clo	ose to edge which may cause s	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	$\phi_{t,c} N_{sb} = \phi_{t,c} \left(160 c \sqrt{A_{brg}} \right) \lambda$	$\lambda \sqrt{f'_c}$ = 76.1	[kips]	D.5.4.1 (D-17)	
Multiple anchors SB resistance	$\varphi_{t,c} N_{sbg,w} \; = \;$				
work as a group - applicable	= (1+s/6c) $x \phi_{t,c} N_{st}$	= 110.0	[kips]	D.5.4.2 (D-18)	
work individually - not applicable	$= n_{bw} x \phi_{t,c} N_{sb} x [1+$	$(c_2 \text{ or } c_4)/c]/4 = 0.0$	[kips]	D.5.4.1	
Seismic design strength reduction	= x 1.0 not applica	ble = 110.0	[kips]	D.3.3.3	
	ratio = 0.13	$> N_{buw}$	OK		
Group side blowout resistance	$\phi_{t,c} \; N_{sbg} \; = \; \phi_{t,c} \; \; \frac{N_{sbgr,w}}{n_{T1}} n_t \label{eq:phitotal}$	= 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]		

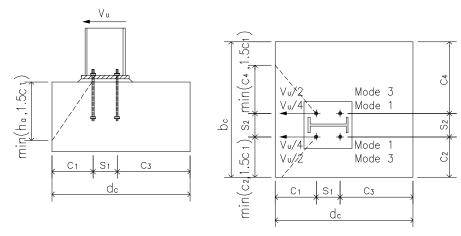
Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear

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



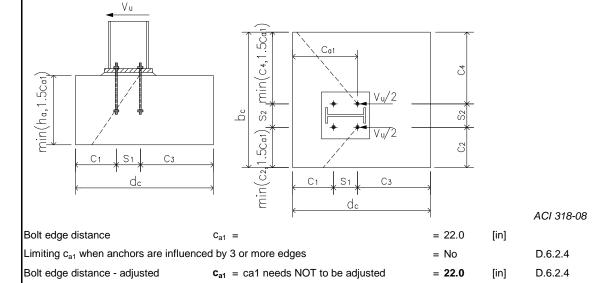
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					6 of 7
				Code Reference	
Bolt edge distance	C _{a1} =	= 6.0	[in]	ACI 318-08	
Limiting c _{a1} when anchors are influenced by 3 or more edges				D.6.2.4	
Bolt edge distance - adjusted	c_{a1} = ca1 needs NOT to be adjusted	= 6.0	[in]	D.6.2.4	
	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	
	$V_b = \left[7 \left(\frac{I_e}{d_a} \right)^{0.2} \sqrt{d_a} \right] \lambda \sqrt{f_c'} c_1^{1.5}$	= 12.3	[kips]	D.6.2.2 (D-24)	
Eccentricity effects	$\Psi_{\text{ec,v}}$ = 1.0 shear acts through center of group	р		D.6.2.5	
Edge effects	$\Psi_{\text{ed,v}} = \text{min}[(0.7+0.3c_2/1.5c_1), 1.0]$	= 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	
Conc shear breakout					
resistance	$V_{cbg1} = \phi_{v,c} \frac{A_{vc}}{A_{vc}} \Psi_{ec,v} \Psi_{ed,v} \Psi_{c,v} \Psi_{h,v} V_b$	= 17.2	[kips]	D.6.2.1 (D-22)	
	Vco			Fig. RD.6.2.1 (b)	
Mode 3 is used for checking	$V_{cbg1} = V_{cbg1} $ x 1.0	= 17.2	[kips]	note	

Mode 2 Failure cone at back anchors



 $C_2 =$

 $1.5c_{a1} =$

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6.0

33.0

[in]

[in]



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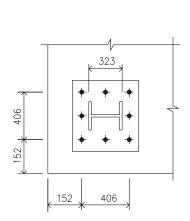
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				Code Reference	7 of
				ACI 318-08	
	$A_{Vc} = [min(c_2, 1.5c_{a1}) + s_2 + min(c_4, 1.5c_{a1})$ $min(1.5c_{a1}, h_a)$] x = 1100.0	[in ²]	D.6.2.1	
	$A_{Vco} = 4.5c_{a1}^{2}$	= 2178.0	[in ²]	D.6.2.1 (D-23)	
	$A_{Vc} = \min (A_{Vc}, n_2 A_{Vco})$	= 1100.0	[in ²]	D.6.2.1	
	$I_e = min(8d_a, h_{ef})$	= 10.0	[in]	D.6.2.2	
	$V_b = \left[7 \left(\frac{I_e}{d_a} \right)^{0.2} \sqrt{d_a} \right] \lambda \sqrt{f_c'} c_{a1}^{1.5}$	= 86.6	[kips]	D.6.2.2 (D-24)	
Eccentricity effects	$\Psi_{\text{ec,v}}$ = 1.0 shear acts through center of gr	roup		D.6.2.5	
Edge effects	$\Psi_{\text{ed,v}} = \text{min}[(0.7+0.3c_2/1.5c_{a1}), 1.0]$	= 0.75		D.6.2.6	
Concrete cracking	$\Psi_{c,v}$ =	= 1.20		D.6.2.7	
Member thickness	$\Psi_{h,v} = max[(sqrt(1.5c_{a1} / h_a), 1.0]$	= 1.28		D.6.2.8	
Conc shear breakout					
resistance	$V_{cbg2} \; = \; \phi_{v,c} \; \; \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V} \; \Psi_{ed,V} \; \Psi_{c,V} \; \Psi_{h,V} \; \; V_{\mathsf{b}}$	= 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	ок		
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	ок		
Govern Shear Resistance	$\mathbf{V_r} = \min \left[\phi_{v,s} V_{sa}, \phi_{v,c} \left(V_{cbg}, V_{cpg} \right) \right]$	= 17.2	[kips]		
Tension Shear Interaction					
Check if $N_u > 0.2 \phi N_n$ and $V_u > 0.2 \phi V_n$	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	ОК		
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				

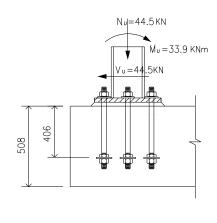
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Example 14: Anchor Bolt + No Anchor Reinft + Tension Shear & Moment + CSA A23.3-04 Code





 $M_u = 33.9 \text{ kNm}$ $N_u = 44.5 \text{ kN}$ (Compression) $V_u = 44.5 \text{ kN}$

Concrete f_c'= 34.5 MPa

Oversized holes in base plate $\label{eq:seismic} \mbox{Seismic design I}_E \, F_a S_a(0.2) < 0.35$

Supplementary reinforcement Tension → Condition A

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

Provide built-up grout pad

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

Anchor bolt design based on

CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D

ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary

PIP STE05121 Anchor Bolt Design Guide-2006

Code Abbreviation

A23.3-04 (R2010)

ACI318 M-08

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
- D.4.1 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
- 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

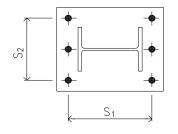
AISC Design Guide 1

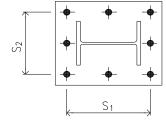
section 3.5.3

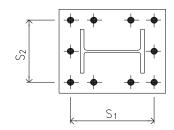
Anchor Bolt Data

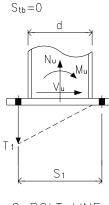
Factored moment	M _u =	33.9	[kNm]		= 25.0	[kip-ft]
Factored tension /compression	$N_u =$	-44.5	[kN]	in compression	= -10.0	[kips]
Factored shear	$V_u =$	44.5	[kN]		= 10.0	[kips]

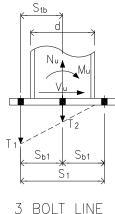
 $V_u = 0$ if shear key is provided Factored shear for bolt design $V_u = 44.5$ [kN]

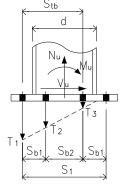












BOLT LINE

2 BOLT LINE

No of bolt line for resisting moment No of bolt along outermost bolt line

3 Bolt Line = 3

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		min required		Code Reference
Outermost bolt line spacing s ₁	$s_1 = 406$	[mm] 127	ок	PIP STE05121
Outermost bolt line spacing s ₂	$s_2 = 406$	[mm] 127	ок	Page A -1 Table 1
nternal bolt line spacing s _{b1}	$s_{b1} = 203$	[mm] 127	OK	
nternal bolt line spacing s _{b2}	$s_{b2} = 0$	[mm] 127	ОК	
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 [MF	Pa] A23.3-04 (R2010)
	Anchor i	s ductile steel element		D.2
Anchor bolt diameter	d _a = 1.25	[in]	= 31.8 [mn	n] PIP STE05121
Bolt sleeve diameter	$d_s = 76$	[mm]		Page A -1 Table 1
Bolt sleeve height	$h_s = 254$	[mm] min required		
\nahar halt amhadmant danth	$h_{ef} = 406$	•	ОК	Dogo A 1 Toble 1
Anchor bolt embedment depth Concrete thickness	$h_a = 508$	[mm] 381 [mm] 482	OK	Page A -1 Table 1
John Cicle Unickness	11 _a = 300	[11111] 402	Oit	
Bolt edge distance c ₁	$c_1 = 152$	[mm] 127	ОК	Page A -1 Table 1
Bolt edge distance c ₂	$c_2 = 152$	[mm] 127	ок	· ·
Bolt edge distance c ₃	$c_3 = 2540$	[mm] 127	ок	
Bolt edge distance c ₄	$c_4 = 2540$	[mm] 127	ок	A23.3-04 (R2010)
$c_i > 1.5h_{ef}$ for at least two edges to avo	oid reducing of h _e	$_{\rm f}$ when $N_{\rm u} > 0$	Yes	D.6.2.3
Adjusted h _{ef} for design	$h_{ef} = 406$	[mm] 381	ок	D.6.2.3
C1 S1 C3	p _o	min(c ₁ ,1.5her) min(c ₃ +s ₁ ,1	min(c2, 1.5hef)	

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No. 1. Constitution of the		0	_						0 1=
Number of bolt at bolt line 1	n ₁ =								3 of 7
Number of bolt at bolt line 2	n ₂ =							Code Reference	
Total no of anchor bolt	n =								
Number of bolt carrying tension	n _t =								
Number of bolt carrying shear	n _s =	4							
Oversized holes in base plate?	=	Yes	?						
Anchor head type	=	Heavy	Hex	T	?				
	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 $I_EF_aS_a(0.2)>=0$	0.35 =	No 🔻	?					D.4.3.5	
Supplementary reinforcement									
For tension	=	Yes	Conditio	n A				D.5.4 (c)	
For shear	$\Psi_{c,V}$ =	1.2	Conditio	n 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		= 1.15	Cdn-A		$R_{v,c} =$	1.15	Cdn-A	D.5.4(c)	
CONCLUSION									
Abchor Rod Embedment, Spacing	and Edg	e Distance	•				ОК		
Overall				•	ratio =	0.81	ОК		
Tension									
Anchor Rod Tensile Resistance					ratio =	0.13	ОК		
Conc. Tensile Breakout Resistance					ratio =	0.39	ок		
Anchor Pullout Resistance					ratio =	80.0	ок		
Side Blowout Resistance					ratio =	0.13	ок		
Shear									
Anchor Rod Shear Resistance					ratio =	0.15	ОК		
Conc. Shear Breakout Resistance					ratio =	0.58	ок		
Conc. Pryout Shear Resistance					ratio =	0.12	ок		
Anchor Rod on Conc Bearing					ratio =	0.04	ок		
Tension Shear Interaction									
Tension Shear Interaction					ratio =	0.81	ОК		
Ductility								A23.3-04 (R2010)	
T	ension	Ductile			Shear	Non-duct	tile		
Seismic Design Requirement							ок	D.4.3.6	
leFaSa(0.2)<0.35, A23.3-04 D.4.3.3	ductility re	equirement	is NOT r	equired					

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							4 c
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	$\mathbf{N_u} = \sum \mathbf{n_i} \mathbf{T_i}$			= 83.9	[kN]		
Tensile bolts outer distance s _{tb}	$s_{tb} = 203$	[mm]					
Eccentricity e' _N distance betwee	en resultant of ter	sile load	and centroid of and	hors			
loaded in tension	$e'_{N} = 35$	[mm]				Figure D.8 (b)	
Eccentricity modification factor	$\Psi_{\text{ec},N} = \frac{1}{\left(1 + \frac{2e}{3h}\right)}$	N ef		= 0.95		D.6.2.4 (D-9)	
Anchor Rod Tensile	$N_{sr} = A_{se} \phi_s f_u$	_{ita} R _{t,s}		= 170.0	[kN]	D.6.1.2 (D-3)	
Resistance	ratio = 0.13			> T ₁	ОК		
Conc. Tensile Breakout Resistand		_					
			if $h_{ef} \leq 275$ or $h_{ef} \geq 62$	25		D.6.2.2 (D-7)	
	3.9 φ _c γ	$f_c^{'}$ $h_{ef}^{5/3}$ 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]		
			+min(c ₃ ,1.5h _{ef})]x +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)	
	$A_{Nc} = min (A_{I})$	Nc, n _t A _{Nco})	= 1.1E+06		D.6.2.1	
Min edge distance	$c_{min} = min(c_1,$			= 152	[mm]		
Eccentricity effects	$\Psi_{ec,N}$ =			= 0.95		D.6.2.4 (D-9)	
Edge effects	$\Psi_{\text{ed,N}} = \text{min}[(0.$	7+0.3c _{min} /	1.5h _{ef}), 1.0]	= 0.78		D.6.2.5	
Concrete cracking	$\Psi_{c,N} = 1.0$ for c					D.6.2.6	
Concrete splitting	$\Psi_{cp,N} = 1.0 \text{ for } c$	cast-in and	hor			D.6.2.7	
Concrete breakout resistance	$N_{cbgr} = \frac{A_{Nc}}{A_{Nco}} \Psi_{c}$	$_{ec,N}\ \Psi_{ed,N}\ \Psi$	$V_{\sf c,N} \; \Psi_{\sf cp,N} \; \; {\sf N}_{\sf br}$	= 213.0	[kN]	D.6.2.1 (D-5)	
Seismic design strength reduction	= x 1.0 r	not applica	ble	= 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 fc' Rt,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 r	not applica	ble	= 261.2	[kN]	D.4.3.5	
	ratio = 0.08			> T ₁	ОК		
	$\Psi_{c,p} = 1$ for cra	acked cond				D.6.3.6	
	$R_{t,c} = 1.00$		strength is always Cor	ndition B		D.5.4(c)	

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Side Blowout Resistance				Code Reference	
Failure Along Pedestal Width Edge					
Tensile load carried by anchors clos	se to edge which may cause si	de-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	$N_{sbr,w} = 13.3c\sqrt{A_{brg}} \phi_c \sqrt{f}$	$R_{t,c} = 339.6$	[kN]	D.6.4.1 (D-18)	
Multiple anchors SB resistance	$N_{sbgr,w} =$				
work as a group - applicable	= (1+s/6c) x N _{sbr,w}	= 490.3	[kN]	D.6.4.2 (D-19)	
work individually - not applicable	$= n_{bw} \times N_{sbr,w} \times [1+(c_2)]$	$(c_2 \text{ or } c_4) / c] / 4 = 0.0$	[kN]	D.6.4.1	
Seismic design strength reduction	= x 1.0 not applicab	e = 490.3	[kN]	D.4.3.5	
	ratio = 0.13	> N _{buw}	ок		
Group side blowout resistance	$N_{sbgr} = \frac{N_{sbgr,w}}{n_{T1}} n_t$	= 817.2	[kN]		
Govern Tensile Resistance	$\mathbf{N_r} = \min (n_t N_{sr}, N_{rbr}, r)$	$N_{t} N_{cpr}, N_{sbgr}$ = 213.0	[kN]		

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

Anchor Rod Shear $V_{sr} = n_s A_{se} \phi_s 0.6 f_{uta} R_{v,s} = 382.5 [kN] D.7.1.2 (b) (D-21)$

Resistance

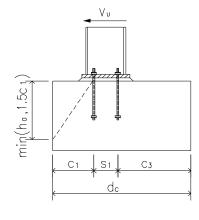
Reduction due to built-up grout pads = x 0.8 , applicable = 306.0 [kN] D.7.1.3

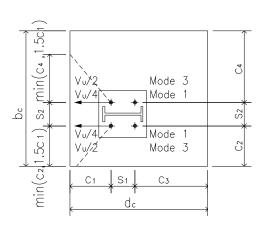
ratio = 0.15 > V_u **OK**

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





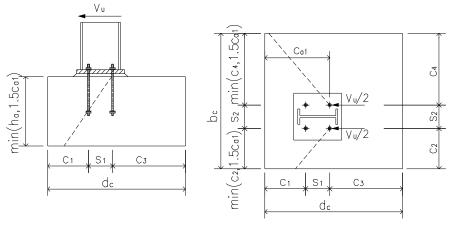
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6 of 7 **Code Reference** Bolt edge distance = 152 A23.3-04 (R2010) [mm] Limiting ca1 when anchors are influenced by 3 or more edges = No D.7.2.4 Bolt edge distance - adjusted **c**₁ = ca1 needs NOT to be adjusted D.7.2.4 = 152 [mm] $C_2 =$ = 152 [mm] $1.5c_1 =$ = 229 [mm] $A_{Vc} = [min(c_2, 1.5c_1) + s_2 + min(c_4, 1.5c_1)] x$ $= 1.8E+05 \text{ [mm}^2\text{]}$ D.7.2.1 $min(1.5c_1, h_a)$ $A_{Vco} = 4.5c_1^2$ $= 1.0E + 05 \text{ [mm}^2\text{]}$ D.7.2.1 (D-24) $A_{Vc} = min (A_{Vc}, n_1 A_{Vco})$ $= 1.8E + 05 \text{ [mm}^2\text{]}$ D.7.2.1 $I_e = min(8d_a, h_{ef})$ = 254 [mm] D.3 $V_{br} = 0.58 \left(\frac{I_e}{d_a} \right)^{0.2} \sqrt{d_a} \phi_c \sqrt{f_c} c_{a1}^{1.5} R_{v,c}$ = 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 $\Psi_{\text{ed,v}} = \text{min}[(0.7+0.3c_2/1.5c_1), 1.0]$ = 0.90Edge effects D.7.2.6 Concrete cracking = 1.20D.7.2.7 $\Psi_{h,v} = max[(sqrt(1.5c_1 / h_a), 1.0]$ Member thickness = 1.00D.7.2.8 Conc shear breakout $V_{\text{cbgr1}} \ = \ \frac{A_{\text{Vc}}}{A_{\text{Vco}}} \Psi_{\text{ec,V}} \ \Psi_{\text{ed,V}} \ \Psi_{\text{c,V}} \ \Psi_{\text{h,V}} \quad V_{\text{br}}$ resistance D.7.2.1 (D-23) $V_{cbgr1} = V_{cbg1}$ x 1.0 Mode 3 is used for checking = 76.4 [kN]

Mode 2 Failure cone at back anchors



A23.3-04 (R2010)

Bolt edge distance = 558 $C_{a1} = C_1 + S_1$ [mm] Limiting ca1 when anchors are influenced by 3 or more edges = No D.7.2.4 Bolt edge distance - adjusted c_{a1} = ca1 needs NOT to be adjusted = 558 D.7.2.4 [mm] $C_2 =$ 152 [mm] $1.5c_{a1} =$ 838 [mm]

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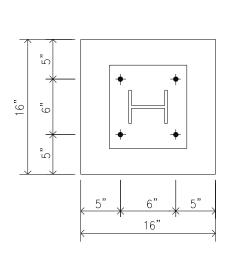
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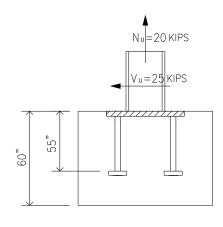
					7 of 7
				Code Reference	7 01 7
	$A_{Vc} = [min(c_2, 1.5c_{a1}) + s_2 + min(c_4, 1.5c_{a1})] x$	= 7.1E+05	[mm ²]	A23.3-04 (R2010)	
	$min(1.5c_{a1}, h_a)$		[]	D.7.2.1	
	$A_{Vco} = 4.5c_{a1}^{2}$	= 1.4E+06	[mm ²]	D.7.2.1 (D-24)	
	$A_{Vc} = \min (A_{Vc}, n_2 A_{Vco})$	= 7.1E+05		D.7.2.1	
	$I_e = min(8d_a, h_{ef})$	= 254	[mm]	D.3	
	$V_{br} = 0.58 \left(\frac{I_e}{d_a}\right)^{0.2} \sqrt{d_a} \ \phi_c \sqrt{f_c} \ c_{a1}^{1.5} R_{v,c}$	= 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	$\Psi_{\text{ed,v}} = \text{min}[(0.7+0.3c_2/1.5c_{a1}), 1.0]$	= 0.75		D.7.2.6	
Concrete cracking	$\Psi_{c,v}$ =	= 1.20		D.7.2.7	
Member thickness	$\Psi_{h,v} = max[(sqrt(1.5c_{a1} / h_a), 1.0]$	= 1.28		D.7.2.8	
Conc shear breakout					
resistance	$V_{\text{cbgr2}} = \frac{A_{\text{Vc}}}{A_{\text{Vco}}} \Psi_{\text{ec,V}} \Psi_{\text{ed,V}} \Psi_{\text{c,V}} \Psi_{\text{h,V}} V_{\text{br}}$	= 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	ОК		
Conc. Pryout Shear Resistance					
	$k_{co} = 2.0$			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 Cor			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	ОК		
		ū		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	ОК		
		-			
Govern Shear Resistance	$V_r = min(V_{sr}, V_{cbgr}, V_{cpgr}, B_r)$	= 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	ОК		
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				

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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 her is required for anchor reinforcement to develop resistance on both sides of the failure plane.

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					1 (
STUD ANCHOR DESIGN C	Combined Tension a	nd Shear			
Anchor bolt design based on					Code Abbreviation
ACI 318-08 Building Code Requirem	ACI 318-08				
PIP STE05121 Anchor Bolt Design (PIP STE05121 Code Reference				
Accumptions	ACI 318-08				
Assumptions 1. Concrete is cracked					ACI 310-00
2. Condition A - supplementary reinfo	arcoment is provided				D.4.4 (c)
Load combinations shall be as per		0 or ASCE 7-05 Chapter	2		D.4.4 (c)
 Anchor reinft strength is used to re 	•	•			D.T.T
ACI318-08 Appendix D clause D.5	•	on 7 onear breakedt streng	jur do por		D.5.2.9 & D.6.2.9
5. For tie reinft, only the top most 2 c		from TOC and 2x3" after) a	are effective		D.0.2.0 Q D.0.2.0
6. Strut-and-Tie model is used to anl					
	,				
nput Data					
		if it's compression			
Factored tension		[kips]	= 89.0	[kN]	
Factored shear	$V_u = 25.0$	[kips]	= 111.2	[kN]	
Concrete strength		[ksi]	= 27.6	[MPa]	
Stud material	= AWS D	1.1 Grade B	▼		
Stud tensile strength		[ksi]	= 448	[MPa]	ACI 318-08
		ctile steel element			D.1
Stud diameter		[in]	= 25.4	[mm]	
Stud shank area		[in ²]	= 507	[mm ²]	
Stud head bearing area	$A_{brg} = 1.29$	[in ²]	= 831	[mm ²]	DID OTFORMO
Otand and advantal	h 55.0	min required	OK		PIP STE05121
Stud embedment depth		[in] 12.0	OK		Page A -1 Table 1
Pedestal height Pedestal width		[in] 58.0	OK		
Pedestal depth		[in] [in]			
=0.5 η ^{ef}	u _c = 10.0	<= min(0.5C ₁ ,0.3C ₂)		<= min	(0.5 C ₁ ,0.3 C ₂)
<=0.5 (ter		(0.501,0.502)			(0.507,0.502)
			>		\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\
50	350	C C C C C C C C C C C C C C C C C C C	35°		C1
$ \begin{array}{c cccc} c_4 & s_2 & c_2 \\ b_c & & & \\ \end{array} $	*	c_2 c_2 c_2	*	C ₂	
Ver. Reinft For Tension	Hor. Ties F	or Shear - 4 Legs	Hor. Ties F	or Shear	· - 2 Legs

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1			min required				2 of
Stud edge distance c ₁	$c_1 = 5.0$	[in]	4.5	ок		Code Reference	
Stud edge distance c ₂	$c_2 = 5.0$	[in]	4.5	ок		PIP STE05121	
Stud edge distance c ₃	$c_3 = 5.0$	[in]	4.5	ок		Page A -1 Table 1	
Stud edge distance c ₄	$c_4 = 5.0$	[in]	4.5	ок		-	
		_					
Outermost stud line spacing s ₁	$s_1 = 6.0$	[in]	4.0	ок		Page A -1 Table 1	
Outermost stud line spacing s ₂	$s_2 = 6.0$	[in]	4.0	ок			
		_				ACI 318-08	
To be considered effective for resisti	ng anchor tension,	ver reinfor	cing bars shall be loc	ated		RD.5.2.9	
within 0.5h _{ef} from the outmost stud's	centerline. In this	design 0.5	h _{ef} value is limited to	8 in.			
			0.5h _{ef}	= 8.0	[in]		
No of ver. rebar that are effective for	resisting anchor te	nsion	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 resisti	ng anchor shear, h	or. reinft sl	hall be located			RD.6.2.9	
within min(0.5c ₁ , 0.3c ₂) from the ou	tmost stud's center	line	$min(0.5c_1, 0.3c_2)$	= 1.5	[in]		
					_		
No of tie <u>leg</u> that are effective to re	sist anchor shear		n _{leg}	= 4	?		
No of tie <u>layer</u> that are effective to re	sist 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 stre	ength calc	100% hor.	. tie bars develop ful	l yield streng	gth 🔽	?	
		_	suggest				
Rebar yield strength	$f_y = 60$	[ksi]	60	= 414	[MPa]		
		_					
Total no of welded stud	n = 4						
Number of stud carrying tension	$n_t = 4$		·				
Number of stud corning sheer			N _{bd}				
Number of stud carrying shear	$n_s = 4$		• (•) Nbd	-			
indiffuel of studicallying streat	$n_s = 4$		• [•]	81 - X - Q			
, ,	n _s = 4		n _{bw} — n _{bd}	-			
For side-face blowout check use No of stud along width edge	n _{bw} = 2	1	n _{bw}	C1 × S1 × Gc			
For side-face blowout check use			Nbw S2, C2	-			
For side-face blowout check use No of stud along width edge	n _{bw} = 2		NbW S2 C2	-			
For side-face blowout check use No of stud along width edge	n _{bw} = 2		Nbw S2 C2	-			
For side-face blowout check use No of stud along width edge	n _{bw} = 2		* * *	5			
For side-face blowout check use No of stud along width edge	n _{bw} = 2		b _c >	Side-Face			
For side-face blowout check use No of stud along width edge	n _{bw} = 2		Bolt No Input for	Side-Face		ACI 318-08	
For side-face blowout check use No of stud along width edge No of stud along depth edge	n _{bw} = 2	?	Bolt No Input for	Side-Face		<i>ACI 318-08</i> D.3.3.3	
For side-face blowout check use No of stud along width edge No of stud along depth edge Seismic design category >= C	$n_{bw} = 2$ $n_{bd} = 2$? ?	Bolt No Input for	Side-Face			
For side-face blowout check use No of stud along width edge No of stud along depth edge Seismic design category >= C Provide built-up grout pad ?	$n_{bw} = 2$ $n_{bd} = 2$ $= Yes$	7	Bolt No Input for	Side-Face		D.3.3.3	
For side-face blowout check use No of stud along width edge No of stud along depth edge Seismic design category >= C Provide built-up grout pad ?	$n_{bw} = 2$ $n_{bd} = 2$ $= Yes$	7	Bolt No Input for	Side-Face		D.3.3.3	
For side-face blowout check use No of stud along width edge No of stud along depth edge Seismic design category >= C Provide built-up grout pad ? Strength reduction factors	$n_{bw} = 2$ $n_{bd} = 2$ $= Yes = No = N$	7	Bolt No Input for Blowout Check Us	Side-Face		D.3.3.3 D.6.1.3	

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CONCLUSION							Code Reference
Abchor Rod Embedment, Spacing	and Edge D	Distance				ОК	ACI 318-08
Min Rquired Anchor Reinft. Develo	_		r	atio =	= 0.25	ок	12.2.1
Overall		,			0.60	ок	
Tension					0.00		
Stud Tensile Resistance			r	atio =	= 0.13	ОК	
Anchor Reinft Tensile Breakout Re	esistance				= 0.09	ОК	
Stud Pullout Resistance	00.010.100				= 0.23	ОК	
Side Blowout Resistance			•		= 0.26	ОК	
Shear				u –	0.20		
Stud Shear Resistance			r	atio -	= 0.19	ок	
Anchor Reinft Shear Breakout Res	sistanca		'	alio -	- 0.15	Oit	
Strut Bearing Strength	Sistarioc		r	atio -	= 0.59	ок	
Tie Reinforcement					= 0.59 = 0.46	OK	
Conc. Pryout Not Govern When h	, _{>=} 12d		ı	สแบ =	- 0.40	OK	
Tension Shear Interaction	ef /= 12ua					OK	
Tension Shear Interaction			-	otio -	0.60	OK	
			ı	alio =	= 0.60	ОК	
Ductility	Tamalan	Non-ductile	CI-		Non-duct		ACI 318-08
Calamia Danima Damainamant	Tension	Non-ductile	Sn	ear	Non-auct		
Seismic Design Requirement	000 0	4.01040 00 D 0 0 F	D.O.O		- C - 1 C	NG	D.3.3.4
OACUU ATION	SDC>= C,	ACI318-08 D.3.3.5	or D.3.3.6 must b	e sati	stied for no	n-auctile	_
CACULATION							Code Reference
							ACI 318-08
Stud Tensile Resistance		$\phi_{t,s} \; n_t \; A_{se} f_{uta}$			= 153.2	[kips]	D.5.1.2 (D-3)
	ratio =	0.13		>	N _u	ОК	
Anchor Reinft Tensile Breakout	Resistance						
Min tension development length	I _d =			=	= 47.4	[in]	12.2.1, 12.2.2, 12.2.4
for ver. #8 bar	-u					[····]	,,
Actual development lenngth	Ι -	h _{ef} - c (2 in) - 8 in x	tan35	_	= 47.4	[in]	
	'a -	Tier O(E III) O III)	tarioo		12.0		12.2.1
.⊑					12.0	OR	12.2.1
p p p p p p p p p p p p p p p p p p p							
35							
							101010 00
							ACI 318-08
		$\phi_s \times f_y \times n_v \times A_s \times (I$	$_{\rm a}$ / $_{\rm d}$, if $_{\rm la}$ < $_{\rm d}$)		= 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	

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								4 o
							Code Reference	
Stud Pullout Resistance							ACI 318-08	
•		$= 8 A_{brg} f$			= 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			$> N_u$	ОК		
	$\Psi_{\text{c,p}}$	= 1 for cr	acked cond				D.5.3.6	
	$\varphi_{t,c}$	= 0.70	pullout s	strength is alwa	ays Condition B		D.4.4(c)	
Side Blowout Resistance								
Failure Along Pedestal Width Edge								
Tensile load carried by anchors close	to edg	e which m	ay cause s	de-face blowo	ut			
along pedestal width edge	N_{buw}	$= N_u \times n_b$	w/nt		= 10.0	[kips]	RD.5.4.2	
	С	= min (c	₁ , c ₃)		= 5.0	[in]		
Check if side blowout applicable	h_{ef}	= 55.0	[in]					
		> 2.5c		side bowout is	s 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	⇒ _{t,c} N _{sb}	$= \phi_{t,c}$ (16	$0 c \sqrt{A_{brg}} \lambda$	-	= 43.1	[kips]	D.5.4.1 (D-17)	
	cN _{sbg,w}		V 9 /	•				
	_		ic) x φ _{t.c} N _{sb}		= 51.7	[kips]	D.5.4.2 (D-18)	
work individually - not applicable		= n _{bw} x φ	_{t.c} N _{sb} x [1+(c ₂ or c ₄)/c]/4	= 0.0	[kips]	D.5.4.1	
Seismic design strength reduction			applicable		= 38.8	[kips]	D.3.3.3	
	ratio	= 0.26			> N _{buw}	ОК		
Failure Along Pedestal Depth Edge					22			
Tensile load carried by anchors close	to edg	e which m	ay cause si	de-face blowo	ut			
along pedestal depth edge	•	$= N_u \times n_b$	•		= 10.0	[kips]	RD.5.4.2	
		= min (c			= 5.0	[in]		
Check if side blowout applicable		= 55.0						
		> 2.5c	r1	side bowout is	s applicable		D.5.4.1	
Check if edge anchors work as a		= 6.0	[in]		$s = s_1 = 6.0$	[in]		
a group or work individually		< 6c	17	edge anchors	work as a group	F-1-1	D.5.4.2	
• •			$0 c \sqrt{A_{brg}} \lambda$	•	= 43.1	[kips]	D.5.4.1 (D-17)	
	cN _{sbg,d}		oo _γ Λ _{brg}) λ	√' c	- 40.1	լաբօյ	5.0. 4 .1 (D-11)	
work as a group - applicable $\psi_{t,i}$	_		ic) x φ _{t,c} N _{sb}		= 51.7	[kips]	D.5.4.2 (D-18)	
work individually - not applicable				c ₁ or c ₃) / c] / 4			D.5.4.2 (D-16)	
Seismic design strength reduction						[kips]		
Seismic design strength reduction	un4:-		applicable		= 38.8	[kips]	D.3.3.3	
	ratio	= 0.26			$> N_{bud}$	OK		
Group side blowout resistance ϕ_t	$_{\rm t,c}$ $N_{\rm sbg}$	= $\phi_{\rm t,c}$ mi	$n \left(-\frac{N_{\text{sbg,w}}}{n_{\text{bw}}} n_{\text{obs}} \right)$, $\frac{N_{sbg,d}}{n_{bd}}n_t$	= 77.5	[kips]		
Govern Tensile Resistance	N	= d. min	ı (N _s , N _{rb} , 1	V. N. Y	= 77.5	[kips]		
CO.O.II TORIORO NEGIGIARIO	ı v r	→ Ψt,C IIIII	' (' 'S, ' 'I'D, '	·cp, ···sbg /	- 11.5	[wps]		

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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)

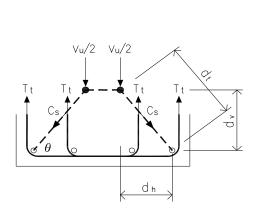
Reduction due to built-up grout pads = x 1.0, not applicable = 132.7 [kips] D.6.1.3

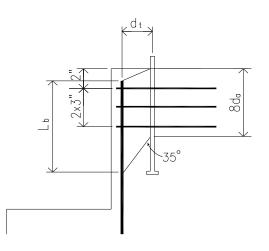
ratio = 0.19 > V_u **OK**

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 $\phi_{st} = 0.75$ 9.3.2.6





Strut-and-Tie model geometry

$$d_v = 2.250$$
 [in]

$$d_h = 2.250$$
 [in]

[in]

OK

$$\theta = 45$$

$$d_t = 3.182$$

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 $I_e = min(8d_a, h_{ef})$ = 8.0 [in] D.6.2.2

Anchor bearing area $A_{brg} \, = \, I_e \, x \, d_a \qquad \qquad = \, 8.0 \qquad \text{[in2]}$

Anchor bearing resistance $C_r = n_s x \phi_{st} x f_{ce} x A_{brg}$ = 81.6 [kips]

 $> V_u$

* Bearing of ver reinft bar

 $\text{Ver bar bearing area} \qquad \qquad A_{brg} \,=\, (I_e + 1.5 \,\, x \,\, d_t - d_a/2 \,\, - d_b/2) \,\, x \,\, d_b \qquad \qquad = \,\, 11.8 \qquad \text{[in2]}$

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

ratio = 0.59 > C_s **OK**

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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
$$n = n_{leg} (leg) \times n_{lay} (layer) = 8$$

ACI 318-08

Pull out resistance at hook
$$T_h = \phi_{l,c} 0.9 f_c' e_h d_a = 3.0$$
 [kips] D.5.3.5 (D-16)

$$e_h = 4.5 d_b$$
 = 2.250 [in]

Single tie bar tension resistance
$$T_r = \phi_s x f_y x A_s = 9.0$$
 [kips]

Total tie bar tension resistance
$$V_{rb} = 1.0 \text{ x n x Tr}$$
 = 72.0 [kips]

Seismic design strength reduction =
$$x \cdot 0.75$$
 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
$$V_r = min (\phi_{v,s} V_{sa}, V_{rb}) = 54.0$$
 [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
$$N_u / N_r + V_u / V_r = 0.72 \qquad \text{D.7.3 (D-32)}$$
 ratio = 0.60 < 1.2 **OK**

Ductility Tension

$$\begin{array}{lll} \phi_{t,s} \; N_{sa} \; = \; 153.2 & \text{[kips]} \\ \\ \; > \; \min \left[\; N_{rb} \; , \; \phi_{t,c} \left(\; N_{pn}, \; N_{sbg} \; \right) \; \right] & = \; 77.5 & \text{[kips]} \\ \\ \hline & \quad \text{Non-ductile} \end{array}$$

Ductility Shear

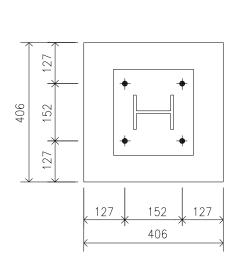
$$\phi_{t,s} \; N_{sa} \; = \; 132.7 \qquad \text{[kips]}$$

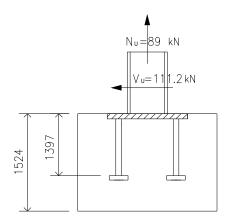
$$\; > \; V_{rb} \qquad \qquad = \; 54.0 \qquad \text{[kips]}$$
 Non-ductile

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





 $N_u = 89 \text{ kN (Tension)}$ $V_u = 111.2 \text{ kN}$

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 $h_{ef} = 1397 mm$ $h_a = 1524 mm$

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 her is required for anchor reinforcement to develop resistance on both sides of the failure plane.

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Ver. Reinft For Tension

Design of Anchorage to Concrete Using ACI 318-08 & CSA-A23.3-04 Code

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						1 o
	ombined T	ension a	and Shear			
Anchor bolt design based on						Code Abbreviation
CSA-A23.3-04 (R2010) Design of Co						A23.3-04 (R2010)
ACI 318M-08 Metric Building Code F	Requirement	ts for Stru	ictural Concrete and Commer	ntary		ACI318 M-08
PIP STE05121 Anchor Bolt Design (Guide-2006					PIP STE05121
Assumptions						Code Reference
Concrete is cracked						A23.3-04 (R2010)
2. Condition A - supplementary reinfo	orcement is	provided				D.5.4 (c)
3. Anchor reinft strength is used to re	eplace conci	rete tensi	on / shear breakout strength	as per		ACI318 M-08
ACI318 M-08 Appendix D clause [0.5.2.9 and	D.6.2.9				D.5.2.9 & D.6.2.9
I. For tie reinft, only the top most 2 o	or 3 layers of	f ties (50	mm from TOC and 2x75mm a	after) are effec	tive	
5. Strut-and-Tie model is used to anl	yze the she	ar transfe	er and to design the required t	ie reinft		
Input Data						
			if it's compression			
Factored tension	$N_u = 8$		[kN]	= 20.0	[kips]	
Factored shear	$V_u = 1$	111.2	[kN]	= 25.0	[kips]	
Concrete strength	f' _c = 2		[MPa]	= 4.0	[ksi]	
Stud material	=	AWS D	1.1 Grade B ▼			
Stud tensile strength	$f_{uta} = 6$	65	[ksi]	= 448	[MPa]	A23.3-04 (R2010)
	5		ctile 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	$A_{brg} = 1$	1.29	[in ²]	= 831	$[mm^2]$	
			min required			PIP STE05121
Anchor bolt embedment depth	$h_{ef} = 1$	1397	[mm] 305	ок		Page A -1 Table 1
Pedestal height	h = 1	1524	[mm] 1473	ОК		
Pedestal width	$b_c = 4$	106	[mm]			
Pedestal depth	$d_c = 4$	106	[mm]			
<=0.5 h ef			<= min(0.5 C ₁ , 0.3 C ₂)		<= min	(0.5 C ₁ , 0.3 C ₂)
* 1						
				>		C3
))	35°	C1 S1 Q Q Q	35°		C1
$ \begin{array}{c c} c_4 & s_2 & c_2 \\ b_c & & \\ \end{array} $		*	S ₂ C ₂ b _c	C ₄ s	*	
Vor Beinft For Tonsion			For Shear Allege	Hor Ties F	01	

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Hor. Ties For Shear - 2 Legs

Hor. Ties For Shear - 4 Legs



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			min required			2 of 6
Stud odgo distance c	0 407	[mm]	min required	OK	Code Refe	
Stud edge distance c ₁	$c_1 = 127$	[mm]	115	OK		
Stud edge distance c ₂	$c_2 = 127$	[mm]	115	OK	PIP STE05	
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	ОК	Page A -1	Гable 1
Outermost stud line spacing s ₂	$S_2 = 152$	[mm]	102	ок	ŭ	
3 12	-				ACI318 M-0	08
To be considered effective for resisting	anchor tensic	on, ver reinfo	orcing bars shall be locate	ed	RD.5.2.9	
within 0.5h _{ef} from the outmost anchor's	centerline. In	this design	0.5h _{ef} value is limited to	200mm.		
			0.5h _{ef} =	200	[mm]	
No of your value that are offertive for re	oiotina on chor	· tonoion	n _v =	0	l	
No of ver. rebar that are effective for re		tension			r 21	
Ver. bar size	$d_b = 25$		single bar area A _s =	: 500	[mm ²]	
To be considered effective for resisting	anchor shear	hor reinfts	shall be located		RD.6.2.9	
within min($0.5c_1$, $0.3c_2$) from the outm			min($0.5c_1$, $0.3c_2$) =	: 38	[mm]	
			(6.66], 6.662)		[]	
No of tie <u>leg</u> that are effective to resis	st anchor shea	ır	n _{leg} =	4	?	
No of tie <u>layer</u> that are effective to resis	st anchor shea	ır	n _{lay} =	2 🔻	?	
Hor. bar size	$d_b = 15$	T	single bar area A _s =	200	[mm ²]	
For anchor reinft shear breakout streng	th calc	100% hor	. tie bars develop full y	ield streng	th ?	
			suggest		_	
Rebar yield strength	$f_y = 414$	[MPa]		: 60.0	[ksi]	
Tatal as of walded about						
Total no of welded stud	n = 4		,_ ,_ Nbd	C3		
No of stud carrying tension	$n_t = 4$		• [•]	-		
No of stud carrying shear	$n_s = 4$			 		
For side-face blowout check use			llpm	2		
No of stud along width edge	$n_{bw} = 2$					
No of stud along depth edge	$n_{bd} = 2$		C4 S2 C2	,		
3	bu		be			
			N	1	A23.3-04 (F	R2010)
Seismic region where $I_EF_aS_a(0.2)>=0.3$	5 = Yes	₹ ?	Bolt No Input for		e D.4.3.5	/
Provide built-up grout pad ?	= No	?	Blowout Check U	se	D.7.1.3	
Strength reduction factors	- 143	<u> </u>			2	
Anchor reinforcement factor	$\phi_{as} = 0.75$				D.7.2.9	
Steel anchor resistance factor	$\phi_{as} = 0.75$ $\phi_{s} = 0.85$				8.4.3 (a)	
Concrete resistance factor	$\phi_{\rm s} = 0.65$ $\phi_{\rm c} = 0.65$				8.4.2	
Control to Todisiano Ciacioi	Ψ _C = 0.05				0.4.2	
Resistance modification factors						
Anchor rod - ductile steel	$R_{t,s} = 0.80$		R _{v.s} =	0.75	D.5.4(a)	
Concrete - condition A	$R_{t,c} = 1.15$: 1.15	D.5.4(c)	

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CONCLUSION						Code Reference	
Abchor Rod Embedment, Spacing	and Edge Distance				ок	A23.3-04 (R2010)	
Min Rquired Anchor Reinft. Develo	pment Length		ratio = 0	0.25	ок	12.2.1	
Overall			ratio = (0.60	ок		
Tension							
Stud Tensile Resistance			ratio = 0	0.14	ок		
Anchor Reinft Tensile Breakout Re	sistance		ratio = 0	0.10	ок		
Stud Pullout Resistance			ratio = 0	0.25	ок		
Side Blowout Resistance			ratio = 0	0.26	ок		
Shear							
Stud Shear Resistance			ratio = 0	0.19	ОК		
Anchor Reinft Shear Breakout Resi	stance						
Strut Bearing Strength			ratio = 0	0.60	ок		
Tie Reinforcement			ratio = 0	0.30	ок		
Conc. Pryout Not Govern When her	>= 12d _a				ок		
Stud on Conc Bearing			ratio = 0	0.21	ок		
Tension Shear Interaction							
Tension Shear Interaction			ratio = 0	0.46	ок		
Ductility	Tension Non-de	uctile	Shear <mark>I</mark>	Non-ducti	le	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 b	e satisfied	for non	-ductile design	
CACULATION						Code Reference	
						A23.3-04 (R2010)	
Stud Tensile Resistance	$N_{sr} = n_t A_{se} \phi$	$o_s f_{uta} R_{t,s}$	= 6	617.7	kN]	D.6.1.2 (D-3)	
	ratio = 0.14		> 1	N _u	ОК		
Anchor Reinft Tensile Breakout F	Resistance						
Min tension development length	$I_d =$		= 8	387	[mm]	12.2.3	
for ver. 25M bar							
Actual development lenngth	$I_a = h_{ef} - c$ (50mm) - 200mm x ta	n35 = 1	1207	[mm]		
<0.5h _{ef}			> 3	300	OK	12.2.1	
200							
Def Def							
35°							
	$N_{rbr} = \phi_{as} x f_{v}$	$x n_v x A_s x (l_a / l_d, if l_a)$	< I _d) = 1	1242.0	[kN]	12.2.5	
Seismic design strength reduction		applicable			kN]	D.4.3.5	
1	ratio = 0.10		> 1		ок		

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				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	$> N_u$	ОК		
	$\Psi_{c,p} = 1$ 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 clos	e 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 a	pplicable		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 we	ork as a group		D.6.4.2	
Single anchor SB resistance	$N_{\text{sbr,w}} = 13.3 \text{c} \sqrt{A_{\text{brg}}} \phi_{\text{c}} \sqrt{f'_{\text{c}}} R_{\text{t,c}}$	= 191.3	[kN]	D.6.4.1 (D-18)	
Multiple anchors SB resistance	$N_{\text{sbgr,w}} =$				
work as a group - applicable	$= (1+s/6c) \times N_{sbr,w}$	= 229.4	[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	= 172.1	[kN]	D.4.3.5	
	ratio = 0.26	> N _{buw}	ОК		
Failure Along Pedestal Depth Edge					
Tensile load carried by anchors clos	e to edge which may cause side-face blowout			ACI318 M-08	
along pedestal depth edge	$N_{\text{bud}} = N_{\text{u}} \times n_{\text{bd}} / n_{\text{t}}$	= 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 a	pplicable		D.6.4.1	
Check if edge anchors work as a		$= s_1 = 152$	[mm]		
a group or work individually	< 6c edge anchors we		į	D.6.4.2	
Single anchor SB resistance	$N_{\rm sbr,d} = 13.3 \text{c} \sqrt{A_{\rm brg}} \phi_{\rm c} \sqrt{f'_{\rm c}} R_{\rm t,c}$	= 191.3	[kN]	D.6.4.1 (D-18)	
Multiple anchors SB resistance	$N_{\text{sbgr,d}} = 13.36 \sqrt{N_{\text{brg}}} \ \psi_{\text{c}} \ \sqrt{V_{\text{c}}} \ N_{\text{t,c}}$	101.0	[1]	(5 .0)	
work as a group - applicable	= $(1+s/6c) \times \phi_{t,c} N_{sbt,d}$	= 229.4	[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	= 172.1	[kN]	D.4.3.5	
Colonia decign energan readenen	ratio = 0.26	> N _{bud}	OK	2.1.0.0	
	14110 - 0.20	> · · · bud	OR		
Group side blowout resistance	$N_{sbgr} = min \left(\begin{array}{c} N_{sbgr,w} \\ n_{bw} \end{array} n_{t}, \begin{array}{c} N_{sbgr,d} \\ n_{bd} \end{array} n_{t} \right)$	= 344.1	[kN]		
Govern Tensile Resistance	$\mathbf{N_r} = \min(N_{sr}, N_{rbr}, N_{cpr}, N_{sbgr})$	= 344.1	[kN]		

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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

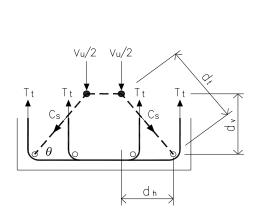
Anchor Reinft Shear Breakout Resistance

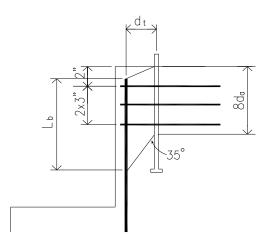
ACI318 M-08

ΟK

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

STM strength reduction factor $\phi_{st} = 0.75$ 9.3.2.6





Strut-and-Tie model geometry

 $d_v = 57$ [mm]

 $d_h = 57$

[mm]

 $\theta = 45$

 $d_t = 81$

[mm]

[kN]

Strut compression force $C_s = 0.5 V_u / \sin \theta$

= 78.6

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 $I_e = min(8d_a, h_{ef})$

= 203

 $> V_u$

[mm] D.6.2.2

Anchor bearing area

 $A_{brg} = I_e x d_a$

= 5161 = 363.3

[mm²] [kN]

Anchor bearing resistance

 $C_r = n_s x \phi_{st} x f_{ce} x A_{brg}$

OK

* Bearing of ver reinft bar

Ver bar bearing area $A_{brg} \ = \ (I_e + 1.5 \ x \ d_t - d_a/2 \ - d_b/2) \ x \ d_b$

= 7473

 $[mm^2]$

Ver bar bearing resistance

 $C_r = \phi_{st} \times f_{ce} \times A_{brq}$

= 131.5

[kN]

ratio = 0.60

 $> C_s$

ΟK

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6 of 6 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_v . 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 $T_h = 0.9 \phi_c f_c' e_h d_b R_{t.c}$ D.6.3.5 (D-17) = 16.3[kN] $e_h = 4.5 d_h$ = 68 [mm] $T_r = \phi_{as} x f_v x A_s$ Single tie bar tension resistance = 62.1 [kN] $V_{rbr} = 1.0 \times n \times Tr$ Total tie bar tension resistance = 496.8[kN] Seismic design strength reduction = x 0.75 applicable = 372.6[kN] D.4.3.5 ratio = 0.30> V_{II} 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 $B_r = n_s x 1.4 x \phi_c x min(8d_a, h_{ef}) x d_a x f_c'$ = 518.5[kN] 25.3.3.2 $> V_u$ ratio = 0.21OK Govern Shear Resistance $\mathbf{V_r} = \min \left(V_{sr}, V_{rbr}, B_r \right)$ = 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 $N_u/N_r + V_u/V_r$ = 0.56D.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

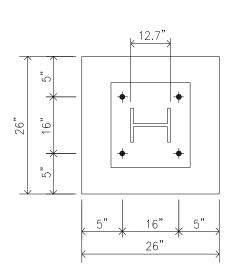
Ductility Shear

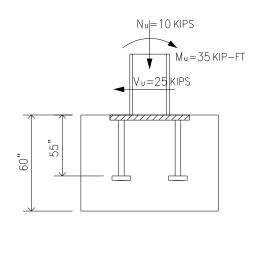
$$V_{sr} = 579.1$$
 [kN]
> min (V_{rbr} , B_{r}) = 372.6 [kN]

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





 $M_u = 35 \text{ kip-ft}$ $N_u = 10 \text{ kips}$ (Compression) $V_u = 25 \text{ kips}$

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 her is required for anchor reinforcement to develop resistance on both sides of the failure plane.

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

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

ACI 318-08 PIP STE05121 Code Reference

Assumptions

- 1. Concrete is cracked
- 2. Condition A supplementary reinforcement is provided
- 3. Load combinations shall be as per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2
- 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
- 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
- 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

Code Abbreviation

D.4.4 (c)

D.4.4

D.3.1

ACI 318-08

D.5.2.9 & D.6.2.9

Anchor Stud Data

Factored shear

Factored moment Factored tension /compression $M_u = 35.0$ $N_u = -10.0$

 $V_u = 25.0$

[kip-ft]

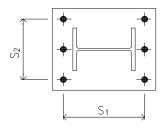
[kips] in compression

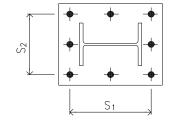
[kips]

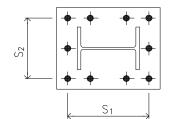
[kNm]

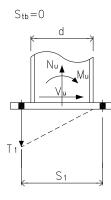
= -44.5 [kN] [kN] = 111.2

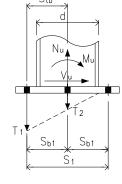
= 47.5

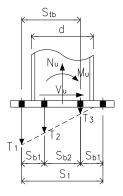








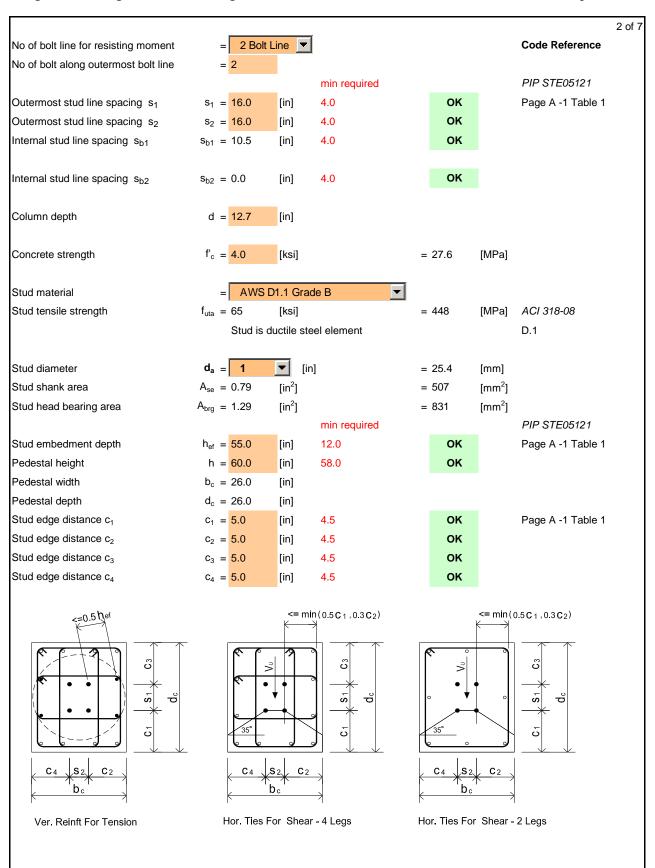




2 BOLT LINE 3 BOLT LINE BOLT LINF

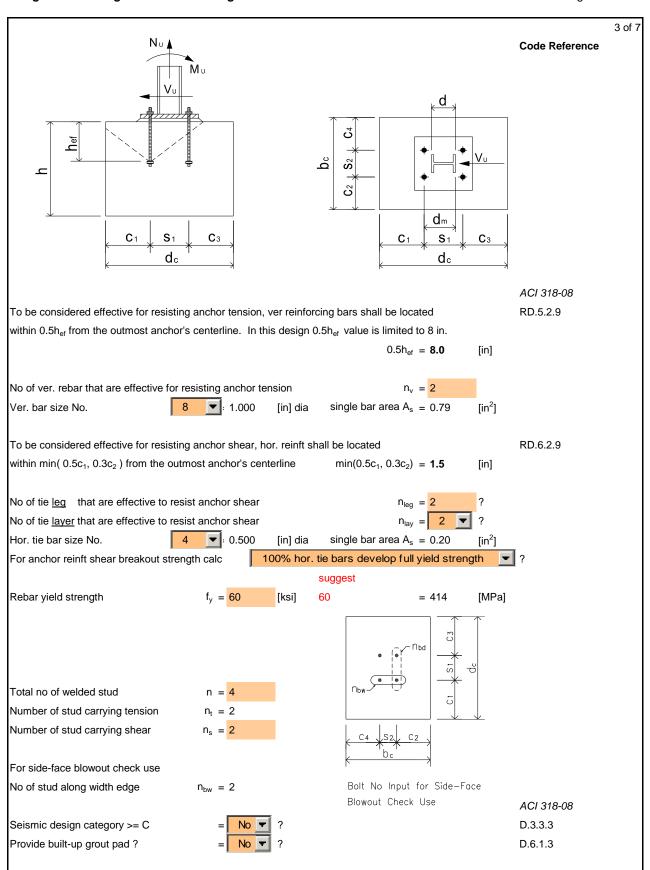
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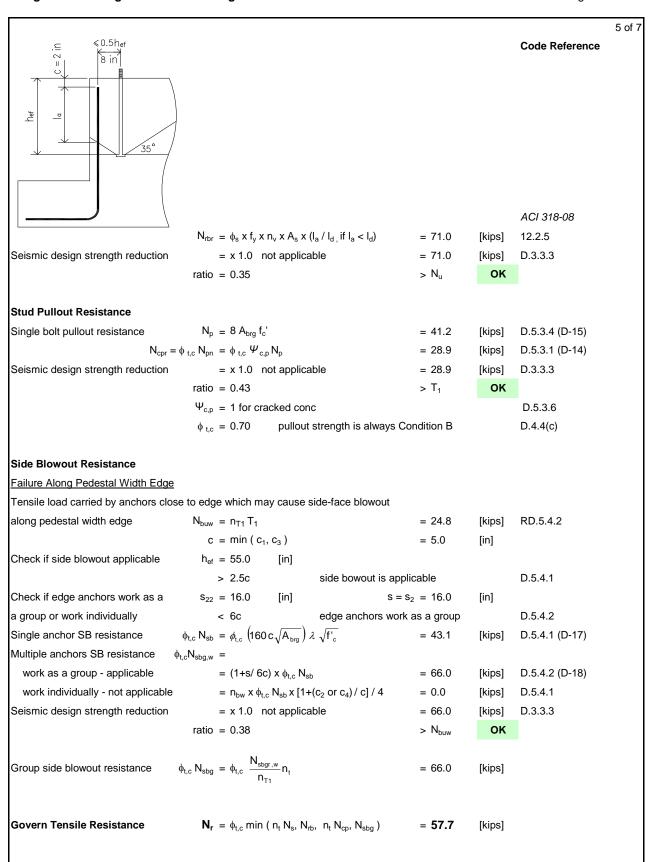


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								4 of
							Code Reference	
Strength reduction factors							ACI 318-08	
Anchor reinforcement	φ _s =	= 0.75					D.5.2.9 & D.6.2.9	
Anchor rod - ductile steel	φ _{t,s} =	= 0.75		$\phi_{v,s}$	= 0.65		D.4.4(a)	
Concrete - condition A	φ _{t,c} =	= 0.75		$\phi_{v,c}$	= 0.75		D.4.4(c)	
CONCLUSION								
Abchor Rod Embedment, Spacing	g and Edge	Distance				ОК		
Min Rquired Anchor Reinft. Devel	opment Len	gth		ratio	= 0.25	ОК	12.2.1	
Overall				ratio	= 0.94	ОК		
Tension Tension								
Stud Tensile Resistance				ratio	= 0.32	ОК		
Anchor Reinft Tensile Breakout R	esistance			ratio	= 0.35	ОК		
Stud Pullout Resistance				ratio	= 0.43	ОК		
Side Blowout Resistance	_			ratio	= 0.38	ОК		
Shear								
Stud Shear Resistance				ratio	= 0.38	ОК		
Anchor Reinft Shear Breakout Re	sistance							
Strut Bearing Strength				ratio	= 0.59	ОК		
Tie Reinforcement				ratio	= 0.69	ОК		
Conc. Pryout Not Govern When h	_{ef} >= 12d _a					ОК		
Tension Shear Interaction								
Tension Shear Interaction	_			ratio	= 0.94	ОК		
Ductility								
	Tension	Non-du	ctile	Shear	Non-duc	tile		
Seismic Design Requirement						OK	D.3.3.4	
	SDC< C, A	ACI318-08	3 D.3.3 du	ctility requirement is N	OT required			
CACULATION								
Stud Tensile Force							ACI 318-08	
Single stud tensile force		= 12.42	[kips]	No of stud for $T_1 n_{T1}$				
		= 0.00	[kips]	No of stud for $T_2 n_{T2}$				
		= 0.00	[kips]	No of stud for $T_3 n_{T3}$				
Sum of bolt tensile force	N _u =	$= \sum n_i T_i$			= 24.8	[kips]		
Stud Tensile Resistance	$\phi_{t,s} N_{sa} =$		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	I _d =	=			= 47.4	[in]	12.2.1, 12.2.2, 12.	2.4
or ver. #8 bar								
Actual development lenngth	l _a =	= h _{ef} - c (2	2 in) - 8 in	x tan35	= 47.4	[in]		
					> 12.0	ок	12.2.1	

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Code Reference

ACI 318-08

ΟK

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)

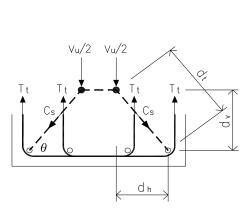
D.6.1.3 Reduction due to built-up grout pads = x 1.0, not applicable = 66.4 [kips]

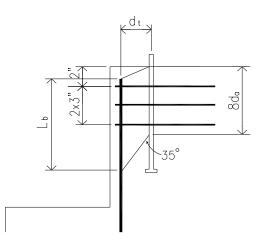
> ratio = 0.38 $> V_u$

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 $\phi_{\rm st} = 0.75$ 9.3.2.6





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

 $d_h = 2.250$ [in] $d_t = 3.182$

 $\theta = 45$

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

ACI 318-08

[in]

Strut Bearing Strength

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

Bearing of anchor bolt

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

 $A_{brg} = I_e \times d_a$ = 8.0 [in²] Anchor bearing area

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

> $> V_u$ OK

Bearing of ver reinft bar

 $A_{brg} = (I_e + 1.5 \times d_t - d_a/2 - d_b/2) \times d_b$ Ver bar bearing area = 11.8 [in²]

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

ratio = 0.59 $> C_s$ oĸ

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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_v . Use the pullout resistance in tension of a single hooked bolt as per ACl318-08 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
$$n = n_{leg} (leg) \times n_{lay} (layer) = 4$$

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
$$T_r = \phi_s x f_y x A_s$$
 = 9.0 [kips]

Total tie bar tension resistance
$$V_{rb} = 1.0 \text{ x n x Tr}$$
 = 36.0 [kips]

Seismic design strength reduction
$$= x \cdot 1.0$$
 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
$$V_r = min (\phi_{v,s}V_{sa}, V_{rb})$$
 = 36.0 [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 $N_u/N_r + V_u/V_r$ = 1.12 D.7.3 (D-32)

$$N_{U}/N_{r} + V_{U}/V_{r} = 1.12 \qquad D.7$$

Ductility Tension
$$\phi_{t,s} N_{sa} = 38.3$$
 [kips] $> \phi_{t,c} \min (N_{rb}, N_{pp}, N_{sbg})$ = 28.9 [kips]

$$> \phi_{t,c} \min (N_{rb}, N_{pn}, N_{sbg}) = 28.9$$
 [kips]

Non-ductile

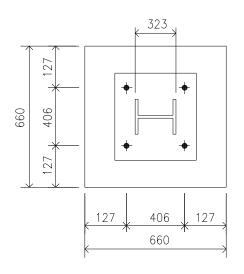
Ductility Shear
$$\phi_{t,s} \; N_{sa} \; = \; 66.4 \qquad \text{[kips]}$$

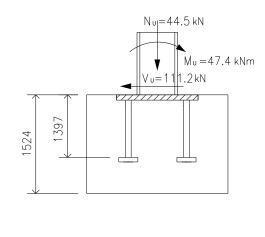
$$> \; V_{rb} \qquad \qquad = \; 36.0 \qquad \text{[kips]}$$
 Non-ductile

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





 $M_u = 47.4 \text{ kNm}$ $N_u = 44.5 \text{ kN (Compression)}$ $V_u = 111.2 \text{ kN}$

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 $h_{ef} = 1397$ mm $h_a = 1524$ mm

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.

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

Anchor bolt design based on

CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D

ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary

PIP STE05121 Anchor Bolt Design Guide-2006

Code Abbreviation

A23.3-04 (R2010)

PIP STE05121

Code Reference

Assumptions A23.3-04 (R2010)

- 1. Concrete is cracked
- 2. Condition A supplementary reinforcement is provided
- 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
- 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
- 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
- 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

ACI318 M-08

D.5.4 (c) ACI318 M-08

D.5.2.9 & D.6.2.9

A23.3-04 (R2010)

D.4.1

Anchor Stud Data

Factored moment

Factored tension /compression

Factored shear

 $M_u = 47.4$

 $V_u = 111.2$

 $N_u = -44.5$ [kN]

[kNm]

[kN]

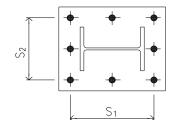
in compression

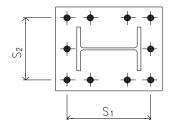
= -10.0

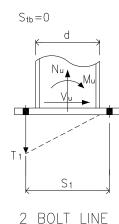
[kips] = 25.0[kips]

[kip-ft]

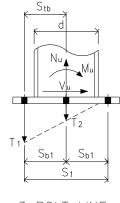
= 35.0

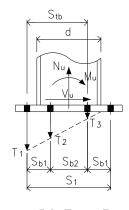






S₁





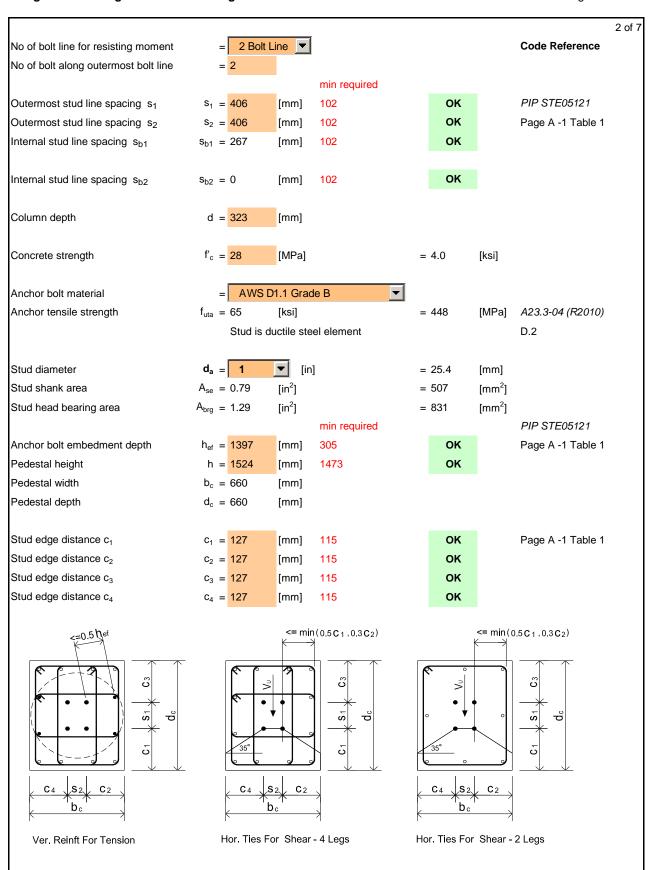
3 BOLT LINE

4 BOLT LINE

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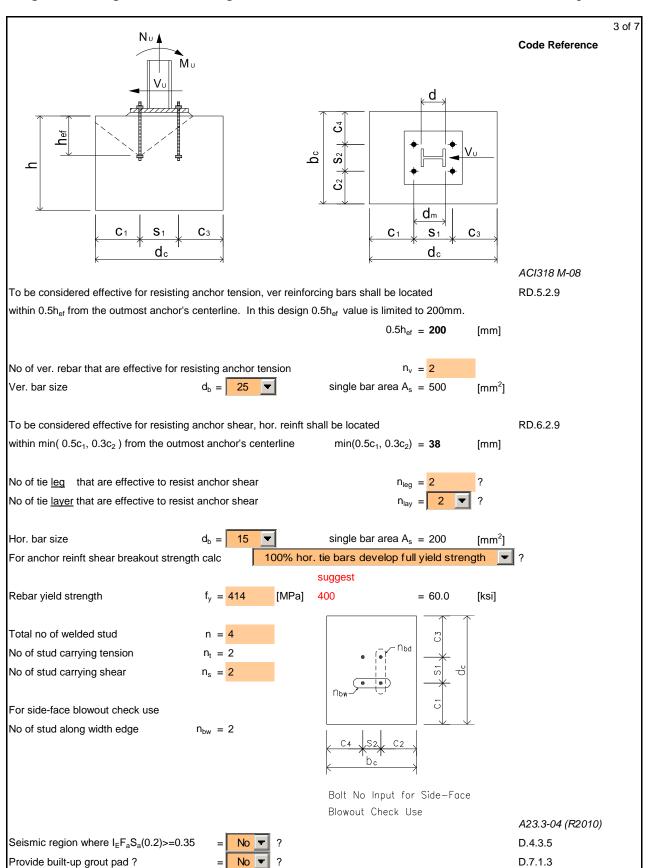
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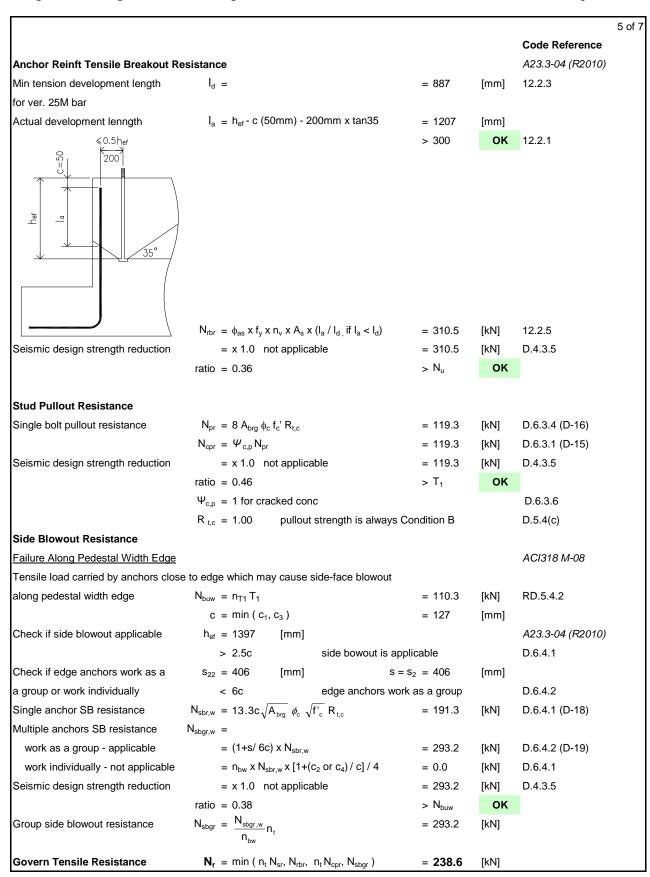
Dongxiao Wu P. Eng.

							Code Referer
Strength reduction factors							A23.3-04 (R2
Anchor reinforcement factor	ϕ_{as} =	0.75					D.7.2.9
Steel anchor resistance factor	$\phi_s =$	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	g and Edge D	istance				ОК	
Min Rquired Anchor Reinft. Devel	opment Leng	th		ratio	= 0.25	ок	12.2.1
Overall				ratio	= 0.76	ок	
Tension							
Stud Tensile Resistance				ratio	= 0.36	ОК	
Anchor Reinft Tensile Breakout R	esistance			ratio	= 0.36	ок	
Stud Pullout Resistance				ratio	= 0.46	ок	
Side Blowout Resistance				ratio	= 0.38	ок	
Shear							
Stud Shear Resistance				ratio	= 0.38	ок	
Anchor Reinft Shear Breakout Re	sistance						
Strut Bearing Strength				ratio	= 0.60	ок	
Tie Reinforcement				ratio	= 0.45	ок	
Conc. Pryout Not Govern When h	n _{ef} >= 12d _a					ок	
Stud on Conc Bearing				ratio	= 0.43	ок	
Tension Shear Interaction							
Tension Shear Interaction				ratio	= 0.76	ок	
Ductility							
	Tension	Non-du	ctile	Shear	Non-du	ctile	
Seismic Design Requirement						ок	D.4.3.6
leFaSa(0.2)<0.35, A23.3-04 D.4.3	3.3 ductility re	quiremer	nt 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 $T_3 n_{T3}$			
		$\Sigma \; n_i T_i$			= 110.3	[kN]	
Sum of stud tensile force							
Sum of stud tensile force Stud Tensile Resistance	N _{sr} =	$A_{se} \phi_s f_u$	_{ita} R _{t,s}		= 154.4	[kN]	D.6.1.2 (D-3)

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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

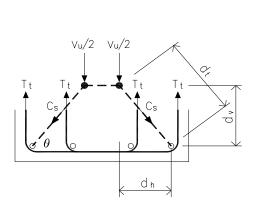
Anchor Reinft Shear Breakout Resistance

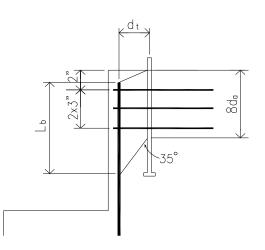
ACI318 M-08

ΟK

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

STM strength reduction factor $\phi_{\rm st} = 0.75$ 9.3.2.6





Strut-and-Tie model geometry

 $d_v = 57$ [mm]

 $d_h = 57$

θ = 45

 $d_t = 81$

[mm] [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 $I_e = min(8d_a, h_{ef})$ = 203 [mm] D.6.2.2

Anchor bearing area $A_{brg} = I_e x d_a = 5161 [mm^2]$

Anchor bearing resistance $C_r = n_s x \phi_{st} x f_{ce} x A_{brg}$ = 181.6 [kN]

 $> V_u$

ок

* Bearing of ver reinft bar

 $\text{Ver bar bearing area} \qquad \qquad A_{\text{brg}} \,=\, (I_{\text{e}} + 1.5 \, \text{x} \, d_{\text{t}} - d_{\text{a}}/2 \, - d_{\text{b}}/2) \, \text{x} \, d_{\text{b}} \qquad \qquad = \, 7473 \qquad [\text{mm}^2]$

Ver bar bearing resistance $C_r = \phi_{st} x f_{ce} x A_{brg} = 131.5$ [kN]

ratio = 0.60 > C_s **OK**

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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.

= 248.4

> V_{...}

[kN]

OK

D.4.3.5

Seismic design strength reduction = x 1.0 not applicable ratio = 0.45

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 $h_{\rm ef} > = 12d_{\rm a}$, the pryout failure will not govern

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)

Ductility Tension
$$N_{sr} = 154.4$$
 [kN] $> min (N_{rbr}, N_{cpr}, N_{sbgr}) = 119.3$ [kN]

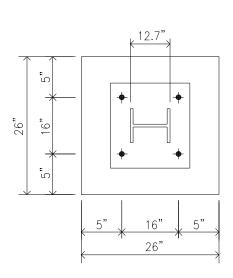
Ductility Shear $V_{sr} = 289.5 \quad [kN]$ $> min (V_{rbr}, B_r) = 248.4 \quad [kN]$

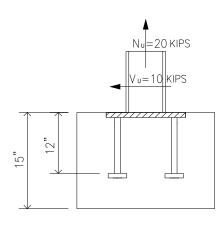
Non-ductile

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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 \rightarrow 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.

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STUD ANCHOR DESIGN	Combined Tension	and Shear				1
Anchor bolt design based on	Combined Tension	i and Silear				Code Abbreviation
ACI 318-08 Building Code Requi	rements for Structural	Concrete an	d Commentary Appen	dix D		ACI 318-08
PIP STE05121 Anchor Bolt Desig			, , ,			PIP STE05121
·	•					
nput Data	set N _u =	0 if it's com	pression			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
		ductile steel				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]	
			nin required			PIP STE05121
Stud embedment depth	h _{ef} = 12.0		2.0	OK		Page A -1 Table 1
Concrete thickness	$h_a = 15.0$	[in] 1	5.0	OK		
Stud edge distance c₁	$c_1 = 5.0$	[in] 4	l.5	ОК		Page A -1 Table 1
Stud edge distance c ₂	$c_2 = 5.0$		1.5	ок		3
Stud edge distance c ₃	$c_3 = 5.0$		1.5	ок		
Stud edge distance c ₄	$c_4 = 5.0$		l.5	ок		ACI 318-08
c _i > 1.5h _{ef} for at least two edges		ef when N _u >	0	No		D.5.2.3
Adjusted h _{ef} for design	$h_{ef} = 5.33$		2.0	Warn		D.5.2.3
Outermost stud line spacing s ₁	$s_1 = 16.0$	[in] 4	1.0	ок		PIP STE05121
Outermost stud line spacing s ₂	$s_2 = 16.0$	[in] 4	1.0	ок		Page A -1 Table 1
C1 S1	C3	**	1, 1.5hef) S1 min(c	C3, 1.5hef)	min(c ₂ , 1.5het) s2 min(c ₄ , 1.5het)	

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2 of 6 **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 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 No ▼ ? Seismic design category >= C D.3.3.3 Supplementary reinforcement Condition A D.4.4 (c) For tension Condition A For shear D.6.2.7 No -Provide built-up grout pad? D.6.1.3 Strength reduction factors Anchor reinforcement $\phi_s = 0.75$ 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) $\phi_{v,c} = 0.75$ Concrete $\phi_{t,c} = 0.75$ Cdn-A 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 FREE FIELD EDGE $\mathsf{A}_{\mathsf{V} \approx (3_{\mathsf{C}_1 + 3})_{\mathsf{1}.5_{\mathsf{C}_1}}}$

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CONCLUSION						
Ababar Dad Embadment Specie	og ond Edo	o Distance			Warn	
Abchor Rod Embedment, Spacin	ig and Edg	je Distance	ratio	= 1.00	OK	
Overall			ralio	= 1.00	UK	
Tension						
Stud Tensile Resistance			ratio	= 0.13	ок	
Conc. Tensile Breakout Resistance	Э		ratio	= 0.57	ок	
Stud Pullout Resistance			ratio	= 0.15	ок	
Side Blowout Resistance			ratio	= 0.00	ок	
Shear						
Stud Shear Resistance			ratio	= 0.15	ок	
Conc. Shear Breakout Resistance			ratio	= 0.62	ок	
Conc. Pryout Shear Resistance			ratio	= 0.15	ОК	
Tension Shear Interaction						
Tension Shear Interaction			ratio	= 1.00	ок	
Tension enear interaction			idio	- 1.00	O.K	
Ductility						
	Tension	Non-ductile	Shear	Non-du	ctile	
Seismic Design Requirement					ок	D.3.3.4
	SDC< C, A	ACI318-08 D.3.3 duc	tility requirement is N	OT required	I	
CALCULATION						Code Reference
						ACI 318-08
Stud Tensile Resistance	φ _{t,s} N _{sa} =	= φ _{t,s} n _t A _{se} f _{uta}		= 153.2	[kips]	D.5.1.2 (D-3)
	ratio =	= 0.13		> N _u	ок	
Conc. Tensile Breakout Resistan		_				
	N _b =	$_{\rm c}$ 24 λ $\sqrt{\rm f_c^{'}}$ $\rm h_{ef}^{1.5}$ if $\rm h_{ef}$	$_{\text{ef}}$ < 11" or h_{ef} > 25"	= 19.8	[kips]	D.5.2.2 (D-7)
		16 $\lambda \sqrt{f_c^{'}} h_{ef}^{5/3}$ if 1	$1" \le h_{ef} \le 25"$			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 _e	$_{ef}$)+min(c_3 ,1.5 h_{ef})]x	= 676.0	[in ²]	
		[s ₂ +min(c ₂ ,1.5h _{ef})+	min(c ₄ ,1.5h _{ef})]			
	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	$\Psi_{\text{ec,N}}$ =	= 1.0 for no eccentric	load			D.5.2.4
Edge effects	$\Psi_{\text{ed,N}}$ =	min[(0.7+0.3c _{min} /1	.5h _{ef}), 1.0]	= 0.89		D.5.2.5
Concrete cracking		1.0 for cracked cor				D.5.2.6
Concrete splitting	$\Psi_{cp,N}$ =	1.0 for cast-in anch	nor			D.5.2.7

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						Code Reference	
						ACI 318-08	
Concrete breakout resistance	$\phi_{t,c} \; N_{cbg}$	$= \phi_{t,c} \frac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi_{ec}$	$_{\mathrm{ed,N}}~\Psi_{\mathrm{c,N}}~\Psi_{\mathrm{cp,N}}~N_{\mathrm{b}}$	= 34.9	[kips]	D.5.2.1 (D-5)	
Seismic design strength reduction		= x 1.0 not applica	ble	= 34.9	[kips]	D.3.3.3	
	ratio	= 0.57		> N _u	ОК		
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 applica	ble	= 129.9	[kips]	D.3.3.3	
	ratio	= 0.15		> N _u	ОК		
	Ψ_{cn}	= 1 for cracked cond	C			D.5.3.6	
			strength is always Co	ndition B		D.4.4(c)	
Side Blowout Resistance							
Failure Along Pedestal Width Edge	!						
Tensile load carried by anchors clo	-	e which mav cause s	side-face blowout				
along pedestal width edge	-	$= N_u \times n_{bw} / n_t$		= 10.0	[kips]	RD.5.4.2	
anong poucotal main ougo		$= \min (c_1, c_3)$		= 5.0	[in]		
Check if side blowout applicable		= 12.0 [in]		- 3.0	נייין		
officer if side blowout applicable		< 2.5c	side bowout is NOT	annlicable		D.5.4.1	
Check if edge anchors work as a		= 0.0 [in]		$applicable$ $_2 = 0.0$	[in]	D.3.4.1	
a group or work individually		< 6c	side bowout is NOT		[]	D.5.4.2	
		$= \phi_{t,c} \left(160 c \sqrt{A_{brg}} \right) \lambda$		= 0.0	[kinc]	D.5.4.1 (D-17)	
		•	ι VI c	= 0.0	[kips]	D.3.4.1 (D-17)	
	$\phi_{t,c}N_{sbg,w}$			0.0	Dia al	D = 4.0 (D 40)	
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		$= n_{bw} x \phi_{t,c} N_{sb} x [1+$			[kips]	D.5.4.1	
Seismic design strength reduction		= x 1.0 not applica	ble	= 0.0	[kips]	D.3.3.3	
		= 0.00		$< N_{buw}$	OK		
Failure Along Pedestal Depth Edge							
Tensile load carried by anchors clo			side-face blowout				
along pedestal depth edge		$= N_u \times n_{bd} / n_t$		= 10.0	[kips]	RD.5.4.2	
	С	$= \min (c_2, c_4)$		= 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	$\phi_{t,c} \; N_{sb}$	$= \phi_{\rm t,c} \left(160 \rm c \sqrt{A_{\rm brg}} \right) \lambda$	l √f' _c	= 0.0	[kips]	D.5.4.1 (D-17)	
Multiple anchors SB resistance	$\phi_{t,c} N_{\text{sbg,d}}$	• •					
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		$= n_{bd} x \phi_{t,c} N_{sb} x [1+$		= 0.0	[kips]	D.5.4.1	
Seismic design strength reduction		= x 1.0 not applica		= 0.0	[kips]	D.3.3.3	
	ratio	= 0.00		< N _{bud}	ОК		

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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 $\mathbf{N_r} = \min \left[\phi_{t,s} \ N_{sa}, \phi_{t,c} \ (N_{cbg}, \ N_{pn}, \ N_{sbg}) \ \right] = 34.9 \quad \text{[kips]}$

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

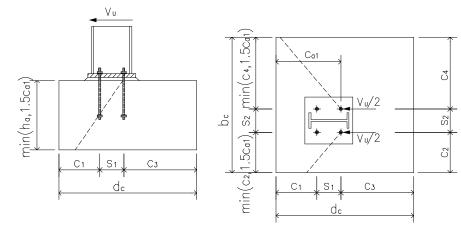
ratio = 0.15 > V_u **OK**

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 $c_{a1} = ca1$ needs to be adjusted = 10.0 [in] D.6.2.4

 $c_2 = 5.0$ [in]

 $1.5c_{a1} = 15.0$ [in]

 $A_{Vc} = [min(c_2, 1.5c_{a1}) + s_2 + min(c_4, 1.5c_{a1})] x = 390.0$ [in²] D.6.2.1

 $min(1.5c_{a1}, h_a)$

 $A_{Vco} = 4.5c_{a1}^2$ = 450.0 [in²] D.6.2.1 (D-23)

 $A_{Vc} = min (A_{Vc}, n_2 A_{Vco})$ = 390.0 [in²] D.6.2.1

 $I_e = min(8d_a, h_{ef})$ = 8.0 [in] D.6.2.2

 $V_{b} = \left[8 \left(\frac{I_{e}}{d_{a}} \right)^{0.2} \sqrt{d_{a}} \right] \lambda \sqrt{f_{c}^{'}} c_{a1}^{1.5}$ = 25.7 [kips] D.6.2.3 (D-25)

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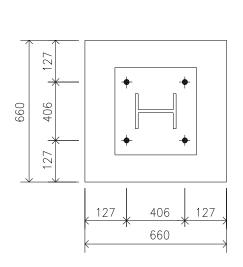
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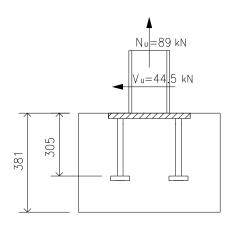
			Code Reference
			ACI 318-08
$\Psi_{\text{ec,v}}$ = 1.0 shear acts through center of φ			D.6.2.5
eu,v L	= 0.80		D.6.2.6
$\Psi_{c,v}$ =	= 1.20		D.6.2.7
$\Psi_{h,v} = max[(sqrt(1.5c_{a1} / h_a), 1.0]$	= 1.00		D.6.2.8
Δ			
$V_{cbg2} = \phi_{v,c} \frac{W_{c}}{A_{Vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_{b}$	= 16.1	[kips]	D.6.2.1 (D-22)
= x 1.0 not applicable	= 16.1	[kips]	D.3.3.3
ratio = 0.62	$> V_u$	ОК	
$k_{cp} = 2.0$			D.6.3
$\phi_{v,c} V_{cpg} = \phi_{v,c} k_{cp} N_{cbg}$	= 65.1	[kips]	D.6.3 (D-31)
	s Condition B		D.4.4(c)
4.0. mat applicable	05.4	Dain al	D 0 0 0
			D.3.3.3
ratio = 0.15	> V _u	OK	
$V_r = min [\phi_{v,s}V_{sa}, \phi_{v,c}(V_{cbg}, V_{cpg})]$	= 16.1	[kips]	
'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	ок	
$\phi_{t,s} N_{sa} = 153.2$ [kips]			
$> \phi_{t,c} \min (N_{cbg}, N_{pn}, N_{sbg})$	= 34.9	[kips]	
Non-ductile			
$\phi_{v,s} V_{sa} = 66.4$ [kips]			
$\phi_{v,c}$ min (V_{cbg} , V_{cpg})	= 16.1	[kips]	
	$\begin{split} \Psi_{\text{h,v}} &= \text{max}[\;(\text{sqrt}(1.5\text{c}_{\text{a1}}/\text{h}_{\text{a}}),1.0] \\ V_{\text{cbg2}} &= \phi_{\text{v,c}} \frac{A_{\text{Vc}}}{A_{\text{Vco}}} \Psi_{\text{ec,v}} \Psi_{\text{ed,v}} \Psi_{\text{c,v}} \Psi_{\text{h,v}} V_{\text{h,v}} \\ &= \text{x}1.0 \text{not applicable} \\ \text{ratio} &= 0.62 \\ \\ K_{\text{cp}} &= 2.0 \\ \phi_{\text{V,c}} V_{\text{cpg}} &= \phi_{\text{v,c}} K_{\text{cp}} N_{\text{cbg}} \\ \phi_{\text{v,c}} &= 0.70 \qquad \text{pryout strength is alway} \\ &= \text{x}1.0 \text{not applicable} \\ \text{ratio} &= 0.15 \\ \\ \textbf{V}_{\textbf{r}} &= \min\left[\phi_{\text{v,s}} V_{\text{sa}},\phi_{\text{v,c}} (V_{\text{cbg}},V_{\text{cpg}})\right] \\ \\ \text{n} \qquad	$\begin{split} \Psi_{\text{h,v}} &= \text{max} [\left(\text{sqrt} (1.5 c_{\text{a1}} / h_{\text{a}} \right), 1.0] \\ V_{\text{cbg2}} &= \phi_{\text{v,c}} \frac{A_{\text{vc}}}{A_{\text{vco}}} \Psi_{\text{ec,v}} \Psi_{\text{ed,v}} \Psi_{\text{c,v}} \Psi_{\text{h,v}} V_{\text{b}} \\ &= 16.1 \\ \\ &= x 1.0 \text{not applicable} \\ \text{ratio} &= 0.62 \\ \end{split} \qquad	$\begin{array}{lll} \Psi_{\text{h,v}} = \text{max} [(\text{sqrt} (1.5c_{\text{a1}} / h_{\text{a}}) , 1.0] & = 1.00 \\ \\ V_{\text{cbg2}} = \phi_{\text{v,c}} \frac{A_{\text{Vc}}}{A_{\text{Vco}}} \Psi_{\text{ec,v}} \Psi_{\text{ed,v}} \Psi_{\text{c,v}} \Psi_{\text{h,v}} V_{\text{b}} & = 16.1 & \text{[kips]} \\ \\ = x 1.0 \text{not applicable} & = 16.1 & \text{[kips]} \\ \text{ratio} = 0.62 & > V_{\text{u}} & \text{OK} \\ \\ k_{\text{cp}} = 2.0 & \\ \phi_{\text{v,c}} V_{\text{cpg}} = \phi_{\text{v,c}} K_{\text{cp}} N_{\text{cbg}} & = 65.1 & \text{[kips]} \\ \phi_{\text{v,c}} = 0.70 & \text{pryout strength is always Condition B} \\ \\ = x 1.0 & \text{not applicable} & = 65.1 & \text{[kips]} \\ \text{ratio} = 0.15 & > V_{\text{u}} & \text{OK} \\ \\ \textbf{V}_{\textbf{r}} = \text{min} [\phi_{\text{v,s}} V_{\text{sa}}, \phi_{\text{v,c}} (V_{\text{cbg}}, V_{\text{cpg}})] & = \textbf{16.1} & \text{[kips]} \\ \\ \text{n} & \text{Yes} & \\ & N_{\text{u}} / N_{\text{r}} + V_{\text{u}} / V_{\text{r}} & = 1.20 \\ \\ \text{ratio} = 1.00 & < 1.2 & \text{OK} \\ \\ \phi_{\text{t,s}} N_{\text{sa}} = 153.2 & \text{[kips]} \\ & > \phi_{\text{t,c}} \text{min} (N_{\text{cbg}}, N_{\text{pn}}, N_{\text{sbg}}) & = 34.9 & \text{[kips]} \\ \\ \hline & \text{Non-ductile} \\ \\ \hline \end{array}$

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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 $h_{ef} = 305$ mm $h_a = 381$ mm

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

Supplementary reinforcement Tension → Condition A

Shear \rightarrow 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.

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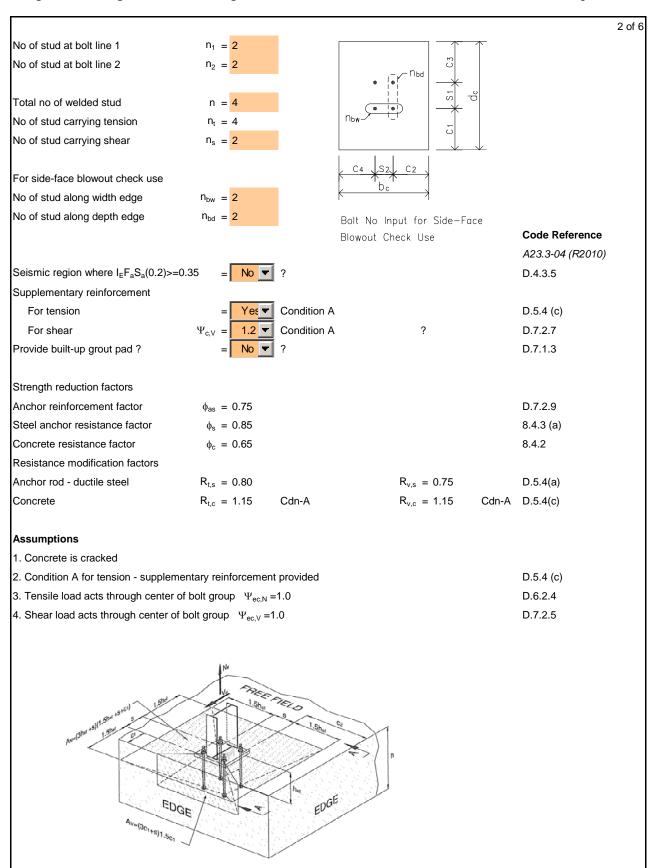
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STUD ANCHOR DESIGN	Combined Tensio	n and Shoar			1 of
Anchor bolt design based on	Combined Tensio	n and Shear			Code Abbreviation
CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D					A23.3-04 (R2010)
ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary					ACI318 M-08
PIP STE05121 Anchor Bolt Design Guide-2006					PIP STE05121
nput Data	set N _u :	= 0 if it's compression			Code Reference
actored 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		D1.1 Grade B	▼		
Anchor tensile strength	f _{uta} = 65	[ksi]	= 448	[MPa]	A23.3-04 (R2010)
-		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	ок		Page A -1 Table 1
Concrete thickness	$h_a = 381$	[mm] 381	ок		
Stud edge distance c₁	c ₁ = 127	[mm] 115	ОК		Page A -1 Table 1
Stud edge distance c ₂	$c_2 = 127$	[mm] 115	ок		rage / Trable r
Stud edge distance c ₃	$c_3 = 127$	[mm] 115	ок		
Stud edge distance c ₄	$c_4 = 127$	[mm] 115	ок		A23.3-04 (R2010)
$c_i > 1.5h_{ef}$ for at least two edges to avoid reducing of h_{ef} when $N_u > 0$					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	ОК		PIP STE05121
Outermost stud line spacing s ₂	$s_2 = 406$	[mm] 102	ок		Page A -1 Table 1
Vu Ni	J -	min(c ₁ ,1.5h _e) s ₁	2) min(c ₃ ,1.5h _{ef})	ءا)	
C1 S1 dc	C3 -	C1 S1	A _N	$\min(c_{2,1.5}$ hef) s2 $\min(c_{4,1.5}$ hef)	

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CONCLUSION						
Abchor Rod Embedment, Spaci	ng and Edç	ge Distance			Warn	
Overall			ratio	= 1.01	NG	
Tension						
Stud Tensile Resistance			ratio	= 0.14	ОК	
Conc. Tensile Breakout Resistanc	е		ratio	= 0.58	ок	
Stud Pullout Resistance			ratio	= 0.17	ок	
Side Blowout Resistance			ratio	= 0.00	ок	
Shear						
Stud Shear Resistance			ratio	= 0.15	ок	
Conc. Shear Breakout Resistance				= 0.63	ок	
Conc. Pryout Shear Resistance				= 0.03	ок	
Stud on Conc Bearing				= 0.17	ок	
Stud on Conc Bearing			iano	- 0.13	OK	
Tension Shear Interaction						
Tension Shear Interaction			ratio	= 1.01	NG	
Ductility						
	Tension	Non-ductile	Shear	Non-duc	tile	
Seismic Design Requirement					OK	D.4.3.6
IeFaSa(0.2)<0.35, A23.3-04 D.4.3	.3 ductility r	equirement is NOT r	equired			
CALCULATION						Code Reference
						A23.3-04 (R2010)
Stud Tensile Resistance	N _{sr} =	= n _t A _{se} φ _s f _{uta} R _{t.s}		= 617.7	[kN]	D.6.1.2 (D-3)
	ratio =	,		> N _u	ОК	, ,
Conc. Tensile Breakout Resista		= 10 $\phi_{\rm c} \sqrt{f_{\rm c}} h_{\rm ef}^{1.5} R_{\rm t,c}$ i	f $h_{ef} \le 275$ or $h_{ef} \ge 6$	25		D.6.2.2 (D-7)
	Di	3.9 $\phi_{\rm c} \sqrt{{\rm f}_{\rm c}^{'}} {\rm h}_{\rm ef}^{5/3} {\rm R}_{\rm t,c}$				D.6.2.2 (D-8)
		φ _c γ'c l'ef l't,c	Tro Tref Tozo	= 65.5	[kN]	2.6.2.2 (2-6)
Projected conc failure area	1.5h _{ef} =	_		= 203	[mm]	
i rojecteu conchanure area			_{ef})+min(c ₃ ,1.5h _{ef})]x	= 203 = 4.4E+05	[mm] [mm²]	
	A _{Nc} =	$[s_1+min(c_1,1.5n_0)+$		= 4.46+05	[mm]	
	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	$\Psi_{\text{ed},N}$ =	= min[(0.7+0.3c _{min} /1	.5h _{ef}), 1.0]	= 0.89		D.6.2.5
Concrete cracking	Ψ _{c,N} =	= 1.0 for cracked cor	ncrete			D.6.2.6
Concrete splitting	$\Psi_{cp,N}$ =	= 1.0 for cast-in anch	nor			D.6.2.7

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					On the Bufferson on	4 of
					Code Reference	
	A _{NC})T()T()T(N			A23.3-04 (R2010)	
Concrete breakout resistance	$N_{cbgr} = \frac{Nc}{A_{Nco}} \Psi$	$_{\text{ec,N}}~\Psi_{\text{ed,N}}~\Psi_{\text{c,N}}~\Psi_{\text{cp,N}}~N_{\text{br}}$	= 153.7	[kN]	D.6.2.1 (D-5)	
Seismic design strength reduction	= x 1.0 r	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 fc' Rt,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 r	not applicable	= 536.0	[kN]	D.4.3.5	
	ratio = 0.17		> N _u	ОК		
	$\Psi_{c,p} = 1$ for cra	acked conc			D.6.3.6	
	$R_{t,c} = 1.00$		Condition B		D.5.4(c)	
Side Blowout Resistance						
Failure Along Pedestal Width Edge						
Tensile load carried by anchors clos	e to edge which m	av cause side-face blowout			ACI318 M-08	
along pedestal width edge	$N_{buw} = N_u \times n_{bv}$		= 44.5	[kN]	RD.5.4.2	
g	$c = min(c_1)$		= 127	[mm]		
Check if side blowout applicable		[mm]	- 121	[]	A23.3-04 (R2010)	
Check it side blowedt applicable	< 2.5c	side bowout is N	OT applicable		D.6.4.1	
Check if edge anchors work as a	$s_{22} = 0$		$= s_2 = 0$	[mm]	D.0.4.1	
a group or work individually	< 6c	side bowout is N		[]	D.6.4.2	
Single anchor SB resistance		$\sqrt{A_{\text{brq}}} \phi_{\text{c}} \sqrt{f'_{\text{c}}} R_{\text{t,c}}$	= 0.0	[kN]	D.6.4.1 (D-18)	
_	•	$/A_{brg} \; \varphi_{c} \; \sqrt{I_{c}} \; K_{t,c}$	= 0.0	[KIN]	D.0.4.1 (D-16)	
Multiple anchors SB resistance	$N_{\text{sbgr,w}} =$	a) v NI	0.0	(I.A.II	D C 4 0 (D 40)	
work as a group - not applicable	= (1+s/6	,,	= 0.0	[kN]	D.6.4.2 (D-19)	
work individually - not applicable		$_{\text{sbr,w}} \times [1+(c_2 \text{ or } c_4)/c]/4$	= 0.0	[kN]	D.6.4.1	
Seismic design strength reduction		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 clos	_				ACI318 M-08	
along pedestal depth edge	$N_{bud} = N_u \times n_{bo}$	ı / 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 N	OT 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 N	OT applicable		D.6.4.2	
Single anchor SB resistance	$N_{sbr,d} = 13.3c_{x}$	$\overline{A_{brg}} \; \phi_{c} \; \sqrt{f'_{c}} \; R_{t,c}$	= 0.0	[kN]	D.6.4.1 (D-18)	
Multiple anchors SB resistance	N _{sbgr,d} =	y 				
work as a group - not applicable	•	c) x $\phi_{t,c}$ $N_{sbr,d}$	= 0.0	[kN]	D.6.4.2 (D-19)	
÷		$_{\text{sbr,d}} \times [1+(c_1 \text{ or } c_3)/c]/4$	= 0.0	[kN]	D.6.4.1	
work individually - not applicable	$= \Pi_{hd} \times \Pi_{s}$	SDF.G X [1 1 (O) O1 O3// O1/ 1				
work individually - not applicable Seismic design strength reduction		not applicable	= 0.0	[kN]	D.4.3.5	

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Code Reference

Group side blowout resistance $N_{\text{sbgr}} = \min \left(\frac{N_{\text{sbgr},w}}{n_{\text{bw}}} n_{\text{t}}, \frac{N_{\text{sbgr},d}}{n_{\text{bd}}} n_{\text{t}} \right) = 0.0$ [kN] A23.3-04 (R2010)

Govern Tensile Resistance $N_r = min (N_{sr}, N_{rbr}, N_{cpr}, N_{sbqr}) = 153.7 [kN]$

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

Conc. Shear Breakout Resistance

Only Case 2 needs to be considered when anchors are rigidly connected to the attachment

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

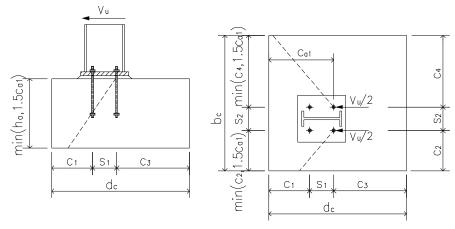
ACI318 M-08

Fig. RD.6.2.1(b) notes

in Case 2

OK

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 $c_{a1} = ca1$ needs to be adjusted = 254 [mm] D.7.2.4

 $c_2 = 127$ [mm]

 $1.5c_{a1} = 381$ [mm]

 $A_{Vc} = [min(c_2, 1.5c_{a1}) + s_2 + min(c_4, 1.5c_{a1})] x = 2.5E+05 [mm^2] D.7.2.1$

 $min(1.5c_{a1}, h_a)$

 $A_{Vco} = 4.5c_{a1}^2$ = 2.9E+05 [mm²] D.7.2.1 (D-24)

 $A_{Vc} = min (A_{Vc}, n_2 A_{Vco})$ = 2.5E+05 [mm²] D.7.2.1

 $I_e = min(8d_a, h_{ef})$ = 203 [mm] D.3

 $V_{br} = 0.66 \left(\frac{I_e}{d_a} \right)^{0.2} \sqrt{d_a} \phi_c \sqrt{f_c} c_{a1}^{1.5} R_{v,c} = 84.9$ [kN] D.7.2.3 (D-26)

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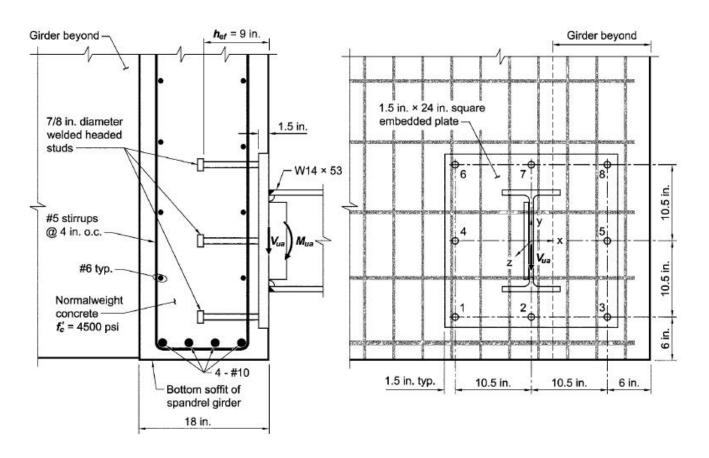
				6
				Code Reference
				A23.3-04 (R2010)
Eccentricity effects	$\Psi_{\rm ec,v} = 1.0$ shear acts through center of gro	oup		D.7.2.5
Edge effects	$\Psi_{\text{ed,v}} = \min[(0.7+0.3c_2/1.5c_{a1}), 1.0]$	= 0.80		D.7.2.6
Concrete cracking	$\Psi_{c,v}$ =	= 1.20		D.7.2.7
Member thickness	$\Psi_{h,v} = max[(sqrt(1.5c_{a1} / h_a), 1.0]$	= 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	> Vu	ОК	
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 C	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	$> V_u$	ОК	
Stud on Cone Bearing	D n v 4 4 v 1 v min/0d h) v d v f !	204.2	[LAI]	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	ОК	
Govern Shear Resistance	$V_r = min(V_{sr}, V_{cbgr}, V_{cpgr}, B_r)$	= 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]			
		= 70.6	[kN]	
	> min (V _{cbgr} , V _{cpgr} , B _r)	- 70.0		

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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



 $M_u = 30 \text{ kip-ft}$ $N_u = 0 \text{ kips}$, $V_u=20 \text{ kips}$, $f_c' = 4.5 \text{ ksi}$

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

Supplementary reinforcement Tension \rightarrow Condition B Shear \rightarrow 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 $A_{Nc} = 1215 \text{ in}^2$, different from $A_{Nc} = 1519 \text{ in}^2$, 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 $c_{a2,1}$ <1.5 c_{a1} and $c_{a2,2}$ <1.5 c_{a1} . Refer to ACI 318-11 Fig. RD.6.2.4 for more details.

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STUD ANCHOR DESIGN Combined Tension, Shear and Moment

Anchor bolt design based on

ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D PIP STE05121 Anchor Bolt Design Guide-2006

Code Abbreviation
ACI 318-08

PIP STE05121

Assumptions

- 1. Concrete is cracked
- 2. Condition B no supplementary reinforcement provided
- 3. Load combinations shall be per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2
- 4. Shear load acts through center of bolt group $\Psi_{ec,V} = 1.0$
- 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
- 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

Code Reference

ACI 318-08

D.4.4 (c)

D.4.4

D.6.2.5

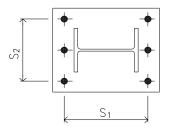
D.3.1

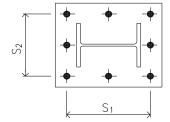
Anchor Stud Data

Factored moment $M_u = 30.0$ [kip-ft] Factored tension /compression $N_u = 0.0$ [kips] Factored shear $V_u = 20.0$ [kips]

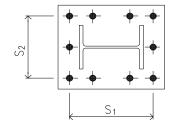
= 40.7 [kNm] = 0.0 [kN]

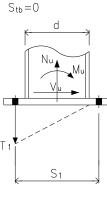
= 89.0 [kN]

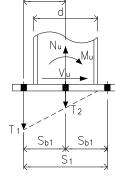


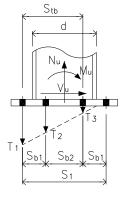


 S_{tb}









2 BOLT LINE

3 BOLT LINE

4 BOLT LINE

No of bolt line for resisting moment

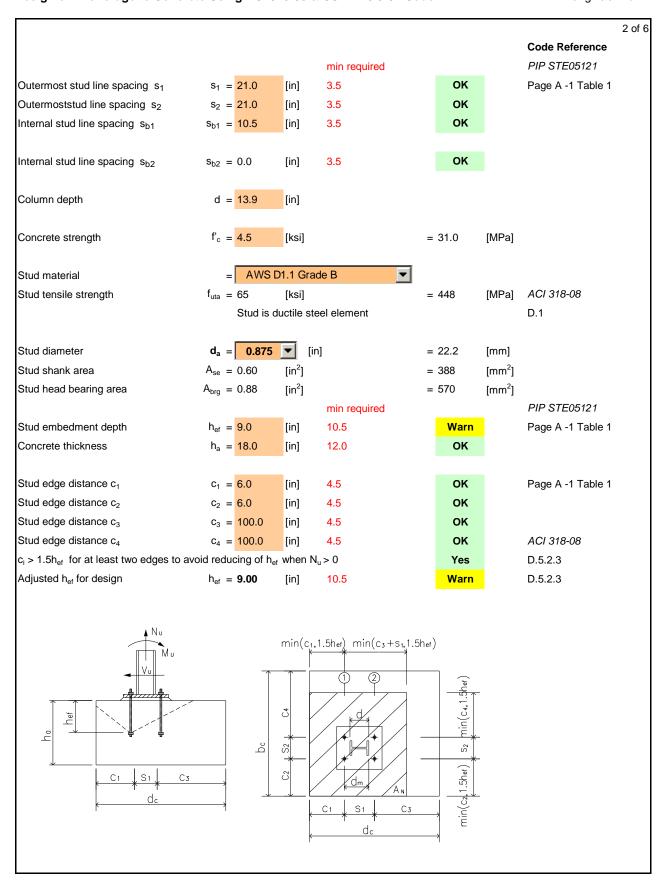
= 3 Bolt Line 🔻

No of bolt along outermost bolt line

= 3

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								3
							0 - 1 - D - (
		_					Code Reference	
Number of stud at bolt line 1	n ₁ =							
Number of stud at bolt line 2	$n_2 =$							
Total no of welded stud	n =							
Number of stud carrying tension	n _t =							
Number of stud carrying shear	n _s =	3						
			Ι.				ACI 318-08	
Seismic design category >= C	=	No 🗾	?				D.3.3.3	
Supplementary reinforcement			ı					
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		$\phi_{\text{v,s}}$	= 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, Spacie	ng and Edg	ıe Distanc	e			Warn		
Abchor Rod Embedment, Spacii	ng and Edg	e Distanc	e	ratio	= 0.95	Warn OK		
Abchor Rod Embedment, Spacii Overall	ng and Edg	e Distanc	е	ratio	= 0.95			
Abchor Rod Embedment, Spacii Overall Tension	ng and Edg	e Distanc	е			ОК		
Abchor Rod Embedment, Spacion Overall Fension Stud Tensile Resistance		e Distanc	е	ratio	= 0.21	ок		
Abchor Rod Embedment, Spacin Overall Tension Stud Tensile Resistance Conc. Tensile Breakout Resistanc		e Distanc	e	ratio ratio	= 0.21 = 0.64	ок ок ок		
Abchor Rod Embedment, Spacin Overall Fension Stud Tensile Resistance Conc. Tensile Breakout Resistance Stud Pullout Resistance		e Distanc	e	ratio ratio ratio	= 0.21 = 0.64 = 0.28	ок ок ок		
Abchor Rod Embedment, Spacin Overall Fension Stud Tensile Resistance Conc. Tensile Breakout Resistance Stud Pullout Resistance		e Distanc	e	ratio ratio ratio	= 0.21 = 0.64	ок ок ок		
Abchor Rod Embedment, Spacin Overall Fension Stud Tensile Resistance Conc. Tensile Breakout Resistanc Stud Pullout Resistance Side Blowout Resistance		e Distanc	e	ratio ratio ratio	= 0.21 = 0.64 = 0.28	ок ок ок		
Abchor Rod Embedment, Spacin Overall Fension Stud Tensile Resistance Conc. Tensile Breakout Resistance Stud Pullout Resistance Side Blowout Resistance		e Distanc	e	ratio ratio ratio ratio	= 0.21 = 0.64 = 0.28	ок ок ок		
Abchor Rod Embedment, Spacin Overall Tension Stud Tensile Resistance Conc. Tensile Breakout Resistance Stud Pullout Resistance Side Blowout Resistance Shear Stud Shear Resistance	е	e Distanc	е	ratio ratio ratio ratio	= 0.21 = 0.64 = 0.28 = 0.00	ок ок ок ок		
Abchor Rod Embedment, Spacin Overall Tension Stud Tensile Resistance Conc. Tensile Breakout Resistance Stud Pullout Resistance Side Blowout Resistance Shear Stud Shear Resistance Conc. Shear Breakout Resistance	е	e Distanc	e	ratio ratio ratio ratio ratio	= 0.21 = 0.64 = 0.28 = 0.00	ок ок ок ок		
Abchor Rod Embedment, Spacin Overall Tension Stud Tensile Resistance Conc. Tensile Breakout Resistance Stud Pullout Resistance Side Blowout Resistance Shear Stud Shear Resistance Conc. Shear Breakout Resistance Conc. Pryout Shear Resistance	е	e Distanc	e	ratio ratio ratio ratio ratio	= 0.21 = 0.64 = 0.28 = 0.00 = 0.26 = 0.50	OK OK OK OK		
Abchor Rod Embedment, Spacin Overall Tension Stud Tensile Resistance Conc. Tensile Breakout Resistance Stud Pullout Resistance Side Blowout Resistance Shear Stud Shear Resistance Conc. Shear Breakout Resistance Conc. Pryout Shear Resistance	е	e Distanc	e	ratio ratio ratio ratio ratio ratio	= 0.21 = 0.64 = 0.28 = 0.00 = 0.26 = 0.50	OK OK OK OK		
Abchor Rod Embedment, Spacin Overall Tension Stud Tensile Resistance Conc. Tensile Breakout Resistance Stud Pullout Resistance Side Blowout Resistance Shear Stud Shear Resistance Conc. Shear Breakout Resistance Conc. Pryout Shear Resistance Tension Shear Interaction	е	e Distanc	e	ratio ratio ratio ratio ratio ratio	= 0.21 = 0.64 = 0.28 = 0.00 = 0.26 = 0.50 = 0.27	ок ок ок ок ок	ACI 318-08	
Abchor Rod Embedment, Spacin Overall Tension Stud Tensile Resistance Conc. Tensile Breakout Resistance Stud Pullout Resistance Side Blowout Resistance Shear Stud Shear Resistance Conc. Shear Breakout Resistance Conc. Pryout Shear Resistance Tension Shear Interaction	е			ratio ratio ratio ratio ratio ratio	= 0.21 = 0.64 = 0.28 = 0.00 = 0.26 = 0.50 = 0.27	ок ок ок ок ок ок	ACI 318-08	
CONCLUSION Abchor Rod Embedment, Spacin Overall Tension Stud Tensile Resistance Conc. Tensile Breakout Resistance Stud Pullout Resistance Side Blowout Resistance Shear Stud Shear Resistance Conc. Shear Breakout Resistance Conc. Pryout Shear Resistance Tension Shear Interaction Tension Shear Interaction Ductility Seismic Design Requirement	е	e Distanc		ratio ratio ratio ratio ratio ratio	= 0.21 = 0.64 = 0.28 = 0.00 = 0.26 = 0.50 = 0.27	ок ок ок ок ок ок	ACI 318-08 D.3.3.4	

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CALCULATION			Code Re	4 c ference
Anchor Stud Tensile Force				
Single bolt tensile force	$T_1 = 6.22$ [kips] No of bolt for	$T_1 n_{T1} = 3$		
olligio boli toriolio foroc	$T_2 = 2.48$ [kips] No of bolt for			
	$T_3 = 0.00$ [kips] No of bolt for			
Sum of bolt tensile force	$\mathbf{N_u} = \sum \mathbf{n_{Ti}} \mathbf{T_i}$		[kips]	
	2.1111		[60]	
Tensile bolts outer distance s _{tb}	$s_{tb} = 10.5$ [in]			
Eccentricity e' _N distance betweer	resultant of tensile load and centroid of ancho	ors	ACI 318-0	08
loaded in tension	$e'_{N} = 2.00$ [in]		Fig. RD.5	.2.4 (b)
Eccentricity modification factor	$\Psi_{\text{ec,N}} = \frac{1}{\left(1 + \frac{2e_{N}^{'}}{3h_{\text{ef}}}\right)}$	= 0.87	D.5.2.4 (I	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 (,
	14.10 = 0.21	2 1		
Conc. Tensile Breakout Resistan				
	N_b = 24 λ $\sqrt{f_c^{'}}$ $h_{ef}^{1.5}$ 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" $\leq h_{ef} \leq 25$ "		D.5.2.2 (I	D-8)
	, 0 s. s.			
Projected conc failure area	$1.5h_{ef} =$	= 13.50	[in]	
	$A_{Nc} = [s_{tb} + min(c_1, 1.5h_{ef}) + min(c_3, 1.5h_{ef})]$	x = 1215.0	[in ²]	
	$[s_2+min(c_2,1.5h_{ef})+min(c_4,1.5h_{ef})]$			
	$A_{Nco} = 9 h_{ef}^2$	= 729.0	[in ²] D.5.2.1 (I	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 (E	D-9)
Edge effects	$\Psi_{\text{ed,N}} = \text{min}[(0.7+0.3c_{\text{min}}/1.5h_{\text{ef}}), 1.0]$	= 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	$\phi_{t,c} N_{cbg} = \phi_{t,c} \frac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_{cd}$	= 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	
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	ок	
	$\Psi_{c,p} = 1$ for cracked conc		D.5.3.6	
	$\phi_{t,c} = 0.70$ pullout strength is alway	vs Condition B	D.4.4(c)	

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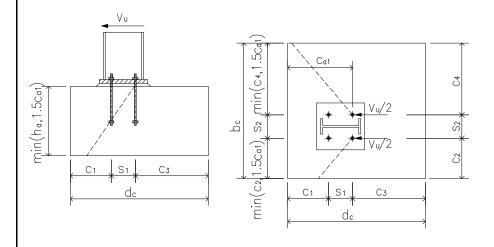
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Side Blowout Resistance						Code Reference	
Failure Along Pedestal Width Edge						ACI 318-08	
Tensile load carried by anchors close	to edge which n	nay cause s	ide-face blowout				
along pedestal width edge	$N_{buw} = n_{T1} T_1$			= 18.7	[kips]	RD.5.4.2	
	c = min(c)	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			side bowout is NOT	applicable		D.5.4.2	
Single anchor SB resistance ϕ	$\rho_{t,c} N_{sb} = \phi_{t,c} (16$	$0 c \sqrt{A_{brg}} \lambda$	$\sqrt{f'_c}$	= 0.0	[kips]	D.5.4.1 (D-17)	
Multiple anchors SB resistance $\phi_{t,c}$	$_{c}N_{sbg,w} =$						
work as a group - not applicable	= (1+s/6)	Sc) $x \phi_{t,c} N_{sb}$		= 0.0	[kips]	D.5.4.2 (D-18)	
work individually - not applicable	$= n_{bw} x \phi$	$P_{t,c} N_{sb} x [1+($	c ₂ or c ₄)/c]/4	= 0.0	[kips]	D.5.4.1	
Seismic design strength reduction	= x 1.0	not applicat	ole	= 0.0	[kips]	D.3.3.3	
	ratio = 0.00			$< N_{buw}$	OK		
Group side blowout resistance $\phi_{t,}$	$_{t,c} N_{sbg} = \phi_{t,c} \frac{N_s}{r}$	$\frac{\log_{t,w}}{n_{t}}$ n_{t}		= 0.0	[kips]		
		'T1					
Govern Tensile Resistance	$N_r = min[\phi]$	$\phi_{t,s} n_t N_{sa}, \phi_{t,o}$	$(N_{cbg}, n_t N_{pn}, N_{sbg})]$	= 36.8	[kips]		
Stud Shear Resistance ϕ	$_{v,s} V_{sa} = \phi_{v,s} n_{s}$	$A_{se} f_{uta}$		= 76.2	[kips]	D.6.1.2 (a) (D-19)	
Reduction due to built-up grout pads	= x 1.0,	not applicat	ole	= 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



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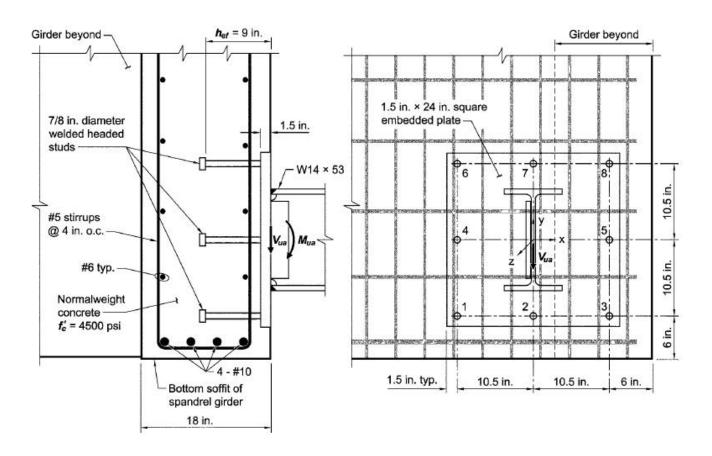
						6 0
					Code Reference	
Bolt edge distance	C _{a1}	=	= 27.0	[in]	ACI 318-08	
Limiting ca1 when anchors are influe	enced by	3 or more edges	= No		D.6.2.4	
Bolt edge distance - adjusted	C _{a1}	= ca1 needs NOT to be adjusted	= 27.0	[in]	D.6.2.4	
	c_2	=	6.0	[in]		
	1.5c _{a1}	=	40.5	[in]		
	A_{Vc}	= $[\min(c_2, 1.5c_{a1}) + s_2 + \min(c_4, 1.5c_{a1})] \times \min(1.5c_{a1}, h_a)$	= 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	
	I _e	= min(8d _a , h _{ef})	= 7.0	[in]	D.6.2.2	
	V_b	$= \left[8 \left(\frac{I_e}{d_a} \right)^{0.2} \sqrt{d_a} \right] \lambda \sqrt{f_c'} c_{a1}^{1.5}$	= 106.7	[kips]	D.6.2.3 (D-25)	
Eccentricity effects	$\psi_{\text{ec},v}$	= 1.0 shear acts through center of group)		D.6.2.5	
Edge effects	$\Psi_{\text{ed},v}$	$= min[(0.7+0.3c_2/1.5c_{a1}), 1.0]$	= 0.74		D.6.2.6	
Concrete cracking	$\Psi_{\text{c},\text{v}}$	=	= 1.20		D.6.2.7	
Member thickness	$\Psi_{\text{h,v}}$	$= max[(sqrt(1.5c_{a1} / h_a), 1.0]$	= 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			= 73.6	[kips]	D.6.3 (D-31)	
, ,		= 0.70 pryout strength is always Cor	ndition B		D.4.4(c)	
Seismic design strength reduction	, ,,-	= x 1.0 not applicable	= 73.6	[kips]	D.3.3.3	
o o	ratio	= 0.27	> V _u	ОК		
Govern Shear Resistance	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 $V_u > 0.2\phi N_n$	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	ОК		
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]				
		> φ _{v,c} min (V _{cbg} , V _{cpg})	= 39.7	[kips]		
		Non-ductile				

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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



$$\begin{split} &M_u=40.7 \text{ kNm} \qquad N_u=0 \text{ kN}, \qquad V_u=89 \text{ kN}, \qquad &f_{c}{'}=31 \text{ MPa} \\ &\text{Anchor stud d}_a=7/8 \text{ in} \qquad &h_{ef}=229 \text{mm} \qquad &h_a=457 \text{mm} \end{split}$$

Supplementary reinforcement Tension \rightarrow Condition B Shear \rightarrow 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 $A_{Nc} = 1215 \text{ in}^2$, different from $A_{Nc} = 1519 \text{ in}^2$, 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 $c_{a2,1}$ <1.5 c_{a1} and $c_{a2,2}$ <1.5 c_{a1} . Refer to ACI 318-11 Fig. RD.6.2.4 for more details.

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

Anchor bolt design based on

CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D

ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary

PIP STE05121 Anchor Bolt Design Guide-2006

Code Abbreviation

A23.3-04 (R2010) ACI318 M-08

PIP STE05121

Assumptions

- 1. Concrete is cracked
- 2. Condition B for tension no supplementary reinforcement provided
- 3. Shear load acts through center of bolt group $\Psi_{ec,V}$ =1.0
- 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
- 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

Code Reference

A23.3-04 (R2010)

D.5.4 (c)

D.7.2.5

D.4.1

Anchor Stud Data

Factored shear

Factored moment Factored tension /compression

 $M_u = 40.7$ $N_u = 0.0$

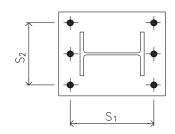
 $V_u = 89.0$

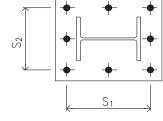
[kNm] [kN] [kN]

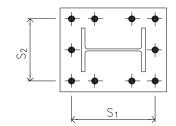
= 30.0

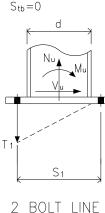
[kip-ft]

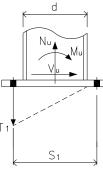
= 0.0= 20.0 [kips] [kips]



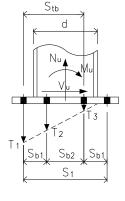








 S_{tb} T 1 S_{b1}



3 BOLT LINE

BOLT LINE

No of bolt line for resisting moment

3 Bolt Line

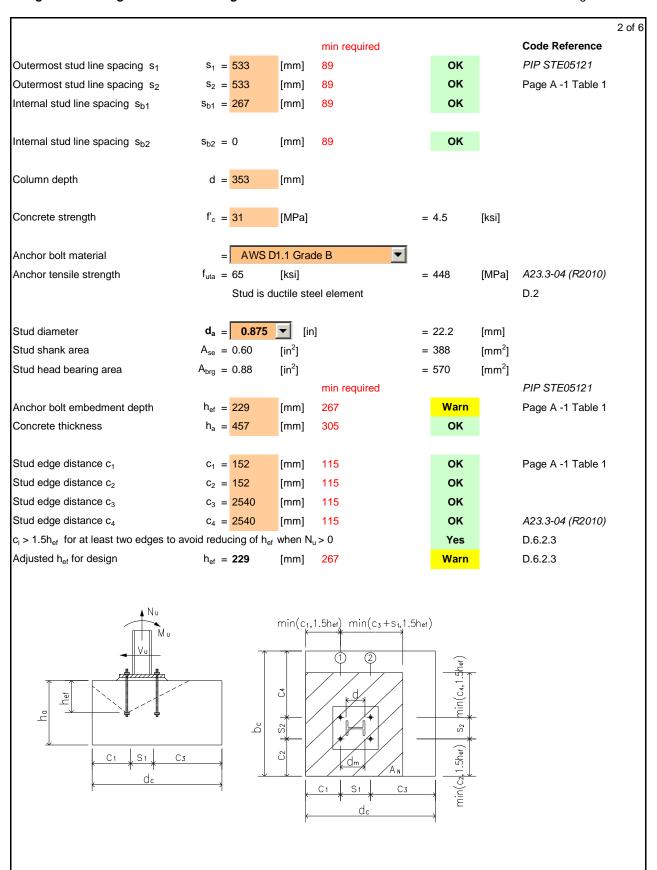
No of bolt along outermost bolt line

= 3

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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 =								
No of stud carrying tension	n _t =	= 5							
No of stud carrying shear	n _s =								
Seismic region where $I_EF_aS_a(0.2)$ >		No =	?					D.4.3.5	
Supplementary reinforcement									
For tension	=	No =	Condition B	,				D.5.4 (c)	
For shear	Ψ _{c,V} =	1.2 🔻	Condition A		?			D.7.2.7	
Provide built-up grout pad ?	=		?					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	фс =	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, Spacia	ng and Edg	e Distance	•				Warn		
Overall					ratio =	1 00	NG	ı	
Tension						1100			
Stud Tensile Resistance					ratio =	0.23	ОК		
Conc. Tensile Breakout Resistanc	;e				ratio =	0.69	ОК		
Stud Pullout Resistance					ratio =		ОК		
Side Blowout Resistance					ratio =		ок		
Shear									
Stud Shear Resistance					ratio =	0.27	ок		
Conc. Shear Breakout Resistance					ratio =		ок		
Conc. Pryout Shear Resistance					ratio =		ок		
Stud on Conc Bearing					ratio =		ок		
Stud on Conc Bearing					TallO =	0.21	OR		
Tension Shear Interaction								_	
Tension Shear Interaction					ratio =	1.00	NG		
Ductility								A23.3-04 (R2010)	
	Tension	Non-duct	tile	S	hear	Non-duc	tile		
Seismic Design Requirement							ок	D.4.3.6	
leFaSa(0.2)<0.35, A23.3-04 D.4.3	.3 ductility r	equirement	is NOT requ	uired					

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							4 o
CALCULATION						Code Reference	
Anchor Tensile Force						A23.3-04 (R2010)	
Single stud tensile force	$T_1 = 27.7$	[kN]	No of stud for $T_1 n_{T_1}$	1 = 3			
	$T_2 = 11.0$	[kN]	No of stud for T ₂ n _{T2}	2 = 2			
	$T_3 = 0.0$	[kN]	No of stud for T ₃ n _{T3}	3 = 0			
Sum of stud tensile force	$\mathbf{N_u} = \sum \mathbf{n_i} \mathbf{T_i}$			= 105.1	[kN]		
Tensile studs outer distance s _{tb}	$s_{tb} = 267$	[mm]					
Eccentricity e' _N distance betwee	en resultant of ter	sile load	and centroid of stu	ds			
loaded in tension	$e'_{N} = 51$	[mm]				Figure D.8 (b)	
Eccentricity modification factor	$\Psi_{\text{ec,N}} = \frac{1}{\left(1 + \frac{2e}{3h_0}\right)}$	N)		= 0.87		D.6.2.4 (D-9)	
Stud Tensile Resistance	$N_{sr} = A_{se} \phi_s f_u$	_{ta} R _{t,s}		= 118.2	[kN]	D.6.1.2 (D-3)	
	ratio = 0.23	,,,		> T ₁	ОК		
Conc. Tensile Breakout Resistand							
	$N_{\rm br} = 10 \phi_{\rm c} \sqrt{100}$	$h_{c}^{1.5} R_{t,c}$	if $h_{ef} \leq 275$ or $h_{ef} \geq 6$	25		D.6.2.2 (D-7)	
	$3.9 \phi_{c}$	f_ h _{ef} R	_{t.c} if 275 < h _{ef} < 625			D.6.2.2 (D-8)	
		6	ι,ο	= 125.4	[kN]		
Projected conc failure area	1.5h _{ef} =			= 344	[mm]		
)+min(c ₃ ,1.5h _{ef})]x +min(c ₄ ,1.5h _{ef})]	= 7.8E+05	[mm ²]		
	$A_{Nco} = 9 h_{ef}^{2}$	(-2) - 61)	(-4) - 6//1	= 4.7E+05	[mm ²]	D.6.2.1 (D-6)	
	$A_{Nc} = min(A_{I})$	n₊ A _{Noc}	.)	= 7.8E+05		D.6.2.1	
Min edge distance	$c_{min} = min(c_1,$			= 152	[mm]	2.0.2	
Eccentricity effects	$\Psi_{\text{ec,N}} =$	-2, -3, -4	,	= 0.87	[]	D.6.2.4 (D-9)	
Edge effects	$\Psi_{\text{ed,N}} = \text{min}[(0.$	7+0.3cmin/	/1.5h _~ (), 1.0 l	= 0.83		D.6.2.5	
Concrete cracking	$\Psi_{c.N} = 1.0 \text{ for } c$			- 0.00		D.6.2.6	
Concrete splitting	$\Psi_{cp,N} = 1.0 \text{ for } c$					D.6.2.7	
ochoroto opiniing	- ср,м					2.0.2	
Concrete breakout resistance	$N_{cbgr} = \frac{A_{Nc}}{A_{Nco}} \Psi_{e}$	c,N Ψ _{ed,N} Y	$\Psi_{c,N} \; \Psi_{cp,N} \; \; N_{br}$	= 151.2	[kN]	D.6.2.1 (D-5)	
Seismic design strength reduction	= x 1.0 r	ot applica	able	= 151.2	[kN]	D.4.3.5	
	ratio = 0.69			> N _u	ОК		
Stud Pullout Resistance							
Single bolt pullout resistance	$N_{pr} = 8 A_{brg} \phi_0$	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 r		able	= 91.9	[kN]	D.4.3.5	
	ratio = 0.30			> T ₁	ОК		
	$\Psi_{c,p} = 1$ for cra	acked con	С			D.6.3.6	
	$R_{t,c} = 1.00$		strength is always Co	ndition B		D.5.4(c)	

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Side Blowout Resistance						Code Reference	
Failure Along Pedestal Width Edge							
Tensile load carried by anchors close	e to edge which	may cause	side-face blowou	ut		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	$N_{sbr,w} = 13.30$	$c\sqrt{A_{brg}} \; \phi_{c} \; \sqrt{A_{brg}} \; \phi_{c} \; \phi_{$	f'_{c} $R_{t,c}$	= 0.0	[kN]	D.6.4.1 (D-18)	
Multiple anchors SB resistance	$N_{\text{sbgr,w}}$ =						
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+($(c_2 \text{ or } c_4)/c]/4$	= 0.0	[kN]	D.6.4.1	
Seismic design strength reduction	= x 1.0	not applica	able	= 0.0	[kN]	D.4.3.5	
	ratio = 0.00			$< N_{buw}$	ок		
Group side blowout resistance	$N_{sbgr} = \frac{N_{sbgr,v}}{n_{T1}}$	n _t		= 0.0	[kN]		
Govern Tensile Resistance			$n_t N_{cpr}, N_{sbgr}$)	= 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 applica	able	= 332.5	[kN]	D.7.1.3	
	ratio = 0.27			$> V_u$	ок		

Conc. Shear Breakout Resistance

Only Case 2 needs to be considered when anchors are rigidly connected to the attachment

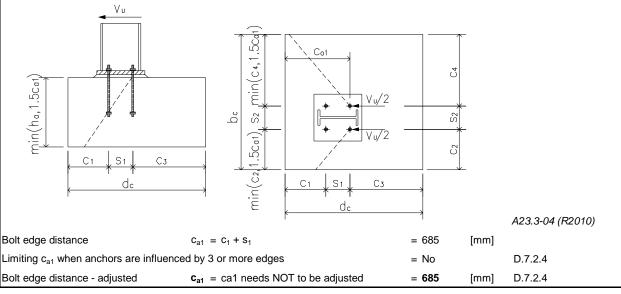
This applies to welded stud case so only Mode 2 is considered for shear checking

ACI318 M-08

Fig. RD.6.2.1(b) notes

in Case 2

Mode 2 Failure cone at back anchors



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	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		D.7.2.1	
	$l_e = min(8d_a, h_{ef})$	= 178	[mm]	D.3	
	$V_{br} \ = \ 0.66 \bigg(\frac{I_e}{d_a}\bigg)^{0.2} \sqrt{d_a} \ \phi_c \sqrt{f_c^{'}} \ c_{a1}^{1.5} R_{v,c}$	= 352.2	[kN]	D.7.2.3 (D-26)	
Eccentricity effects	$\Psi_{ec,v}$ = 1.0 shear acts through center of ground	ap		D.7.2.5	
Edge effects	$\Psi_{\text{ed,v}} = \text{min}[(0.7+0.3c_2/1.5c_{a1}), 1.0]$	= 0.74		D.7.2.6	
Concrete cracking	$\Psi_{c,v}$ =	= 1.20		D.7.2.7	
Member thickness	$\Psi_{h,v} = max[(sqrt(1.5c_{a1} / h_a), 1.0]$	= 1.50		D.7.2.8	
Conc shear breakout	•				
resistance	$V_{\text{cbgr}} = \frac{A_{\text{Vc}}}{A_{\text{Vco}}} \Psi_{\text{ec,V}} \ \Psi_{\text{ed,V}} \ \Psi_{\text{c,V}} \ \Psi_{\text{h,V}} \ V_{\text{br}}$	= 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	> Vu	ОК		
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 C	ondition B		D.5.4(c)	
Seismic design strength reduction	= x 1.0 not applicable	= 302.4	[kN]	D.4.3.5	
	ratio = 0.29	$> V_u$	ОК		
				CSA S16-09	
Stud on Conc Bearing	$B_r = n_s x 1.4 x \phi_c x min(8d_a, h_{ef}) x d_a x f_c'$	= 334.4	[kN]	25.3.3.2	
	ratio = 0.27	$> V_u$	ОК		
Govern Shear Resistance	$V_r = min(V_{sr}, V_{cbgr}, V_{cpgr}, B_r)$	= 174.8	[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	
TOTIOUR II INU ZO.Z IN AIIU VU ZO.Z V	$N_u/N_r + V_u/V_r$	= 1.20		D.8.4 (D-35)	
	ratio = 1.00	> 1.20	NG	D.0.4 (D-33)	
Ductility Tension	– 1.00	- 1.4	110		
buculty rension	$N_{sr} = 118.2$ [kN]				
	$ > \min(N_{cbgr}, N_{cpr}, N_{sbgr}) $	= 91.9	[kN]		
	Non-ductile	- 51.5	[KIV]		
Ductility Shear	Non-adelie				
	$V_{sr} = 332.5$ [kN]				
	> min (V _{cbgr} , V _{cpgr} , B _r)	= 174.8	[kN]		
	Non-ductile		r1		

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Example 41: Shear Lug Design ACI 349-06 Code

SHEAR LUG / SHEAR KEY DESIGN						
Shear Lug / Shear Key design based			_	_		Code Abbreviation
ACI 349-06 Code Requirements for N				Commentary		ACI 349-06
AISC Design Guide 1: Base Plate and		-	Edition			AISC Design Guide 1
AISC 360-05 Specification for Structu	ıral Steel Build	lings				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	= E70	XX				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
o o o o o o o o o o o o o o o o o o o	V _u	min(e,h _o) + d _e	min(e, h _o) d	V _u √45: t		D D
$e = 0.5(d_c - t) t$ d_c	<u> </u>		h _o ·	-pedestal	height]

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Code Reference

ACI 349-06

CONCLUSION

OVERALL

ratio = 0.94OK

ratio = 0.41Concrete Bearing

ΟK D.4.6.2

Shear Toward Free Edge

ratio = 0.81

ΟK D.11.2

Shear Key Section Flexure & Shear Check

Shear Key To Base Plate Fillet Weld

ratio = 0.94ratio = 0.69

OK ΟK

CALCULATION

Concrete Bearing

$$A_b = w d_e = w (d-g)$$

$$= 48.42$$
 [in²]

$$V_b = 1.3 \phi f_c' A_b$$

ratio =
$$0.41$$

$$> V_{ux}$$

 $\phi = 0.65$

for anchor controlled by concrete bearing

D.4.4 (d)

Shear Toward Free Edge

$$e = 0.5x(d_c - t)$$

$$e = min(e, h_a)$$

$$A_{eff} = [e + (d-g)] x b_c - wx(d-g)$$

[in]

$$\Phi V_n = 4\phi \sqrt{f_c} \quad A_{eff}$$
ratio = 0.81

=
$$92.2$$
 > V_u

 $\phi = 0.80$

D.4.4 (f)

D.11.2

Shear Key Section Flexure & Shear Check

Shear Key Plate Sect

This case does not apply

$$Z = w \times t^2 / 4$$

 $M_{ux} = V_{ux} x [0.5x(d-g) + g]$

$$[in^3]$$

 $\phi M_n = 0.9 \times Z \times F_n$ ratio = 0.00

141.9

[kip-in] OK

$$\phi V_n = 0.9 \times A_w \times 0.6 F_y$$

Shear

$$\phi V_n = 0.9 \times A_w \times 0.6F$$

Shear Key Pipe Sect

This case does not apply

ratio = 0.00

$$M_{ux} = V_{ux} x [0.5x(d-g) + g]$$

Z =

$$= 0.00$$

 $\phi M_n = 0.9 \times Z \times F_v$

ratio = 0.00

[in³]

ratio = 0.00

[kip-in] oĸ

 $< M_{ux}$ = 0.000

[in²]

 $\phi V_n = 0.9 \times A_w \times 0.6 F_y$

= 0.0 $< V_{ux}$ [kips] OK

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				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	$\phi M_n = 0.9 \times Z \times F_y$	= 0.0	[kip-in]	ADIE
	ratio = 0.00	< M _{ux}	OK	ADLE
Shear	$A_{w} = \phi V_{n} = 0.9 \times A_{w} \times 0.6 F_{v}$	= 0.000 = 0.0	[in ²]	
	$\psi V_n = 0.9 \times A_w \times 0.0 \Gamma_y$ ratio = 0.00	= 0.0 < V _{ux}	[kips]	
	Talio = 0.00	V ux	OR	
Shear Key W Sect	This case applies			
Flexure strong axis	$M_{ux} = V_{ux} \times [0.5x(d-g) + g]$	= 375.0	[kip-in]	
v	$Z_x =$	= 39.80	[in ³]	
	$\phi M_{nx} = 0.9 \times Z_x \times F_y$	= 1791.0	[kip-in]	
	ratio = 0.21	> M _{ux}	ОК	
	$M_{uy} = V_{uy} x [0.5x(d-g) + g]$	= 250.0	[kip-in]	
	$Z_y =$	= 18.50	[in ³]	
Flexure weak axis	$\phi M_{ny} = 0.9 \times Z_y \times F_y$	= 832.5	[kip-in]	
	ratio = 0.30	$> M_{uy}$	ОК	
	$b_f = 8.07 \qquad [in]$	d = 8.25	[in]	
	$t_w = 0.360$ [in]	$t_f = 0.560$	[in]	
Shear strong axis	$A_w = t_w \times d$	= 2.97	[in ²]	
	$\phi V_{nx} = 0.9 \times A_w \times 0.6 F_y$	= 80.2	[kips]	
	ratio = 0.94	> V _{ux}	OK	
Chaar waak ayia	$A_w = 2 \times t_f \times b_f$	= 9.04	r: . 21	
Shear weak axis	$A_{w} = 2 \times V_{f} \times D_{f}$ $\phi V_{ny} = 0.9 \times A_{w} \times 0.6F_{y}$	= 9.04 = 244.0	[in ²] [kips]	
	$\varphi V_{ny} = 0.3 \times A_w \times 0.01 \text{ y}$ $\text{ratio} = 0.20$	V_{uy}	OK [kiba]	
	14110 - 0.20	≥ • uy	OIL	
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
	$\phi = 0.75$			
Weld metal shear strength	$\phi r_{n1} = \phi (0.707 \times A_m) \times F_w$	= 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	$\phi r_{n2} = min [1.0(0.6F_y t) , 0.75(0.6F_u t)]$	= 0.00		Eq J4-3 & Eq J4-4

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Code Reference

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

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

= 375.0

Shear Key Plate

This case does not apply

$$s = t + (1/3)A_m \times 2$$

[kip-in]

 $f_t = M_{ux} / (s x w)$

 $= V_{ux} / (w \times 2)$

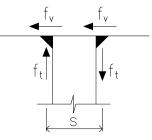
ratio = 0.00

= 0.00= 0.00= 0.00

 $< \phi r_n$

[kips/in] [kips/in] [kips/in]

OK



Force on Shear Key Plate Weld

Shear Key Pipe Sect

This case does not apply

Weld ring diameter

 $f_t = M_{ux} / (\pi D^2 / 4)$

= 8.07 = 0.00= 0.00= 0.00 $< \phi r_n$

[in] [kips/in] [kips/in] [kips/in] ΟK

Shear Key HSS Sect

This case does not apply

ratio = 0.00

Weld box width/depth

$$b = 8.07$$
 [in]

$$d = 0.00$$
 [in]

 $f_t = M_{ux} / (bd + d^2/3)$ $= V_{ux} / (2xd)$ $f_r = \sqrt{f_c^2 + f_v^2}$

= 0.00[kips/in] = 0.00[kips/in] = 0.00[kips/in]

OK

Shear Key W Sect

This case applies

ratio = 0.00

b = 8.07[in] $f_t = M_{ux}/(bxd)$ $f_v = V_{ux} / (2xd)$ $f_r = \sqrt{f_c^2 + f_v^2}$

d = 8.25= 5.63

 $< \phi r_n$

= 4.55= 7.24

[kips/in] [kips/in]

[kips/in]

[in]

ratio = 0.69

 $< \phi r_n$

= 5.76

OK

Weak Axis

Strong Axis

 $f_t = M_{uy} / [(1xb^2/6) x 4]$

 $f_v = V_{uy} / (4xb)$ $f_r = \sqrt{f_c^2 + f_v^2}$

= 1.55 = 5.96

[kips/in] [kips/in] [kips/in]

ratio = 0.57

οк $< \phi r_n$

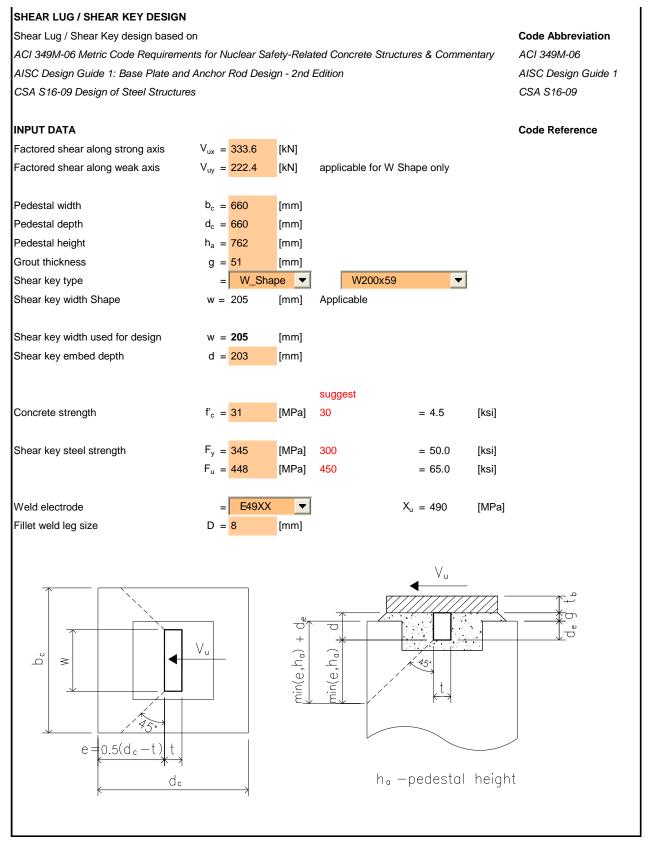
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Example 42: Shear Lug Design ACI 349M-06 Code



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Code Reference ACI 349M-06

OK

CONCLUSION

OVERALL

ratio = 0.94

ratio = 0.41ΟK D.4.6.2 Concrete Bearing ΟK D.11.2 Shear Toward Free Edge ratio = 0.81

Shear Key Section Flexure & Shear Check ratio = 0.94OK Shear Key To Base Plate Fillet Weld ratio = 0.79ΟK

CALCULATION

Concrete Bearing

 $A_b = w d_e = w (d-g)$ = 31242[mm²] $V_b = 1.3 \phi f_c' A_b$ = 818.4 [kN] D.4.6.2 ratio = 0.41OK $> V_{ux}$ $\phi = 0.65$ for anchor controlled by concrete bearing D.4.4 (d)

Shear Toward Free Edge

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

Shear Key Section Flexure & Shear Check

Shear Key Plate Sect

This case does not apply

 $M_{ux} = V_{ux} x [0.5x(d-g) + g]$ = 42.4[kNm] $Z = w \times t^2 / 4$ = 52.5 $[x10^{3}mm^{3}$ $\phi M_n = 0.9 \times Z \times F_n$ = 16.3[kNm] ratio = 0.00OK $\phi V_n = 0.9 \times A_w \times 0.6 F_y$ = 1222.1 Shear [kN]

ratio = 0.00 $> V_{ux}$ OK

Shear Key Pipe Sect This case does not apply

 $M_{ux} = V_{ux} x [0.5x(d-g) + g]$ = 42.4[kNm] Z = = 0.0 $[x10^3 mm^3]$ $\phi M_n = 0.9 \times Z \times F_v$ = 0.0[kNm]

ratio = 0.00 $< M_{ux}$ oĸ = 0 $[mm^2]$

 $\phi V_n = 0.9 \times A_w \times 0.6 F_y$ = 0.0[kN] ratio = 0.00 $< V_{ux}$ OK

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			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	$\phi M_n = 0.9 \times Z \times F_y$	= 0.0	[kNm]
	ratio = 0.00	< M _{ux}	OK
Shear	A _w =	= 0	[mm ²]
	$\phi V_n = 0.9 \times A_w \times 0.6 F_y$	= 0.0	[kN]
	ratio = 0.00	$< V_{ux}$	ОК
Shear Key W Sect	This case applies		
Flexure strong axis	$M_{ux} = V_{ux} \times [0.5x(d-g) + g]$	= 42.4	[kNm]
ŭ	$Z_x =$	= 653	[x10 ³ mm ³]
	$\phi M_{nx} = 0.9 \times Z_x \times F_y$	= 202.8	[kNm]
	ratio = 0.21	> M _{ux}	ОК
	$M_{uy} = V_{uy} x [0.5x(d-g) + g]$	= 28.2	[kNm]
	$Z_y =$	= 303	[x10 ³ mm ³]
Flexure weak axis	$\phi M_{ny} = 0.9 \times Z_y \times F_y$	= 94.1	[kNm]
	ratio = 0.30	$> M_{uy}$	ОК
	b _f = 205.0 [mm]	d = 210.0	[mm]
	t _w = 9.1 [mm]	$t_f = 14.2$	[mm]
Shear strong axis	$A_w = t_w \times d$	= 1911	[mm ²]
	$\phi V_{nx} = 0.9 \times A_w \times 0.6 F_y$	= 356.0	[kN]
	ratio = 0.94	> V _{ux}	ОК
Shear weak axis	$A_w = 2 \times t_f \times b_f$	= 5822	[mm²]
onda waak azaa	$\phi V_{ny} = 0.9 \times A_w \times 0.6 F_y$	= 1084.6	[kN]
	ratio = 0.21	> V _{uy}	ОК
Shear Key To Base Plate Base metal resistance	Fillet Weld $A_{m} = D \times 1 mm$	= 8.00	[mm ²]
Dase metal resistance	$v_{rm} = 0.67 \phi_w A_m F_u$	= 1.61	[kN/mm] 13.13.2.2
	$\phi_{\rm w} = 0.67$	_ 1.01	13.1 (h)
	TW 0.0.		
Weld metal resistance	$A_w = 0.707 \times D \times 1mm$	= 5.66	[mm ²]
Fillet weld resistance - she	ar $\theta =$	= 90	
	$V_{rw} = 0.67 \varphi_w A_w X_u (1 + 0.5 sin\theta ^1.5)$	= 1.87	[kN/mm] 13.13.2.2
	$v_r = min(v_{rm}, v_{rw})$	= 1.61	[kN/mm]

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4 of 4 **Code Reference** $M_{ux} = V_{ux} x [0.5x(d-g) + g]$ Factored moment to base plate = 42.4 [kNm] $M_{uy} = V_{uy} x [0.5x(d-g) + g]$ = 28.2 [kNm] **Shear Key Plate** This case does not apply $s = t + (1/3)D \times 2$ = 37.3[mm] $f_t = M_{ux} / (s x w)$ = 0.00[kN/mm] $= V_{ux} / (w \times 2)$ = 0.00[kN/mm] = 0.00[kN/mm] ratio = 0.00OK $< \phi r_n$ Force on Shear Key Plate Weld **Shear Key Pipe Sect** This case does not apply Weld ring diameter = 205.0 [mm] $f_t = M_{ux} / (\pi D^2 / 4)$ = 0.00[kN/mm] = 0.00[kN/mm] = 0.00[kN/mm] ratio = 0.00 ΟK $< \phi r_n$ **Shear Key HSS Sect** This case does not apply Weld box width/depth b = 205.0d = 205.0[in] [mm] $f_t = M_{ux} / (bd + d^2/3)$ = 0.00[kN/mm] $= V_{ux} / (2xd)$ = 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 = 205.0d = 210.0[in] [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 = 0.79OK $< \phi r_n$ $f_t = M_{uy} / [(1xb^2/6) x 4]$ Weak Axis = 1.01 [kN/mm] $f_v = V_{uy} / (4xb)$ = 0.27 [kN/mm]

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= 1.04

 $< \phi r_n$

[kN/mm] οк

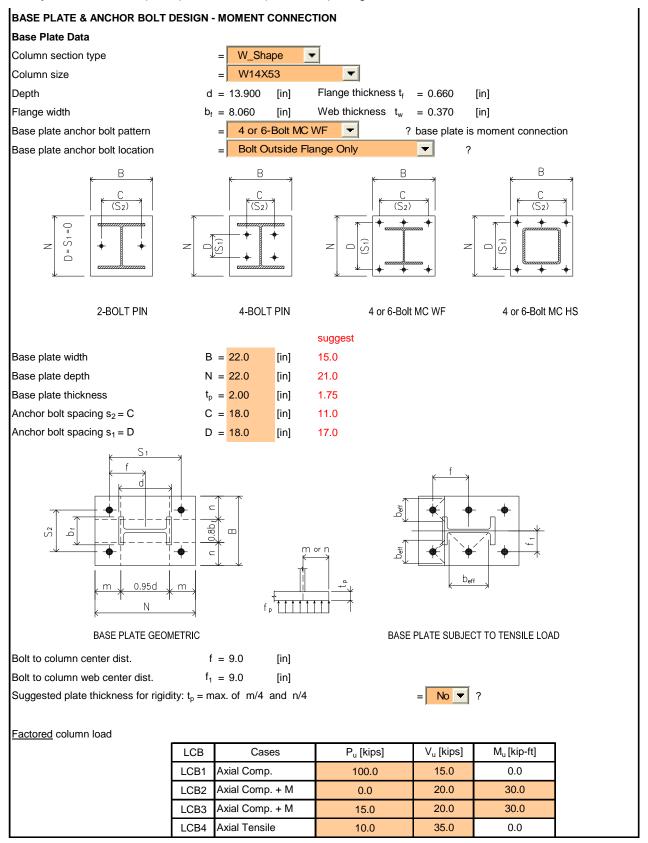
 $f_r = \sqrt{f_c^2 + f_v^2}$

ratio = 0.65



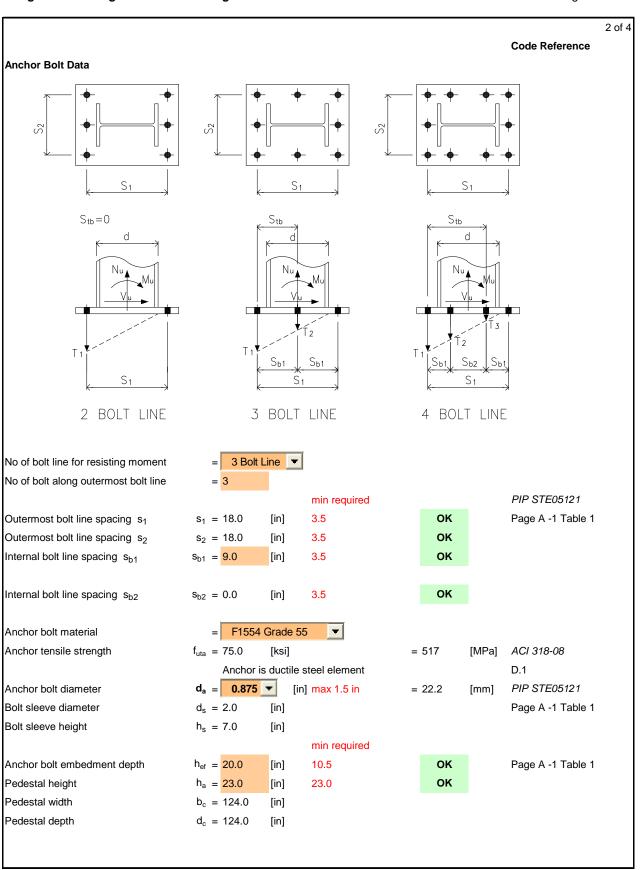
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Example 51: Base Plate (LRFD) & Anchor Bolt (ACI 318-08) Design With Anchor Reinforcement



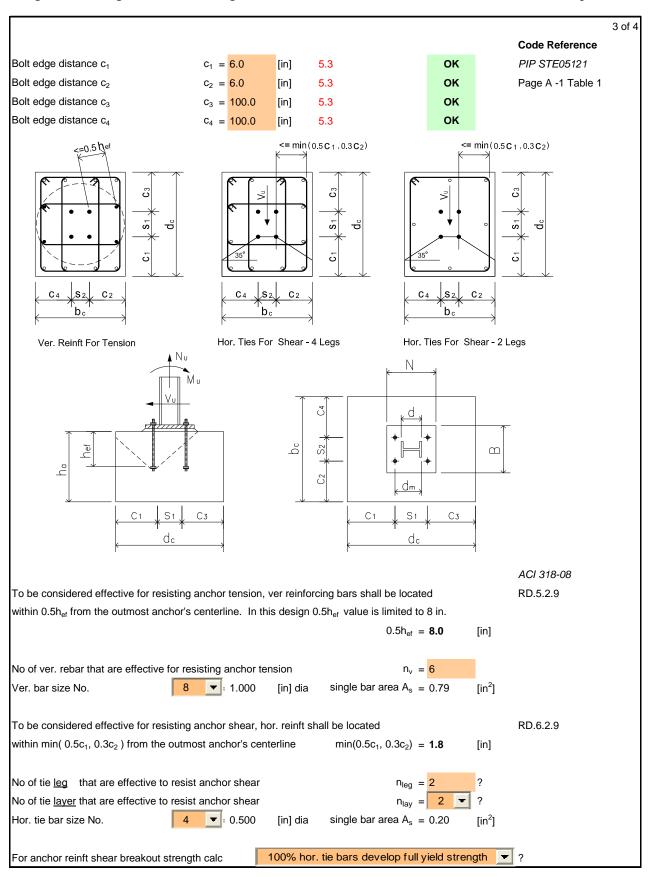
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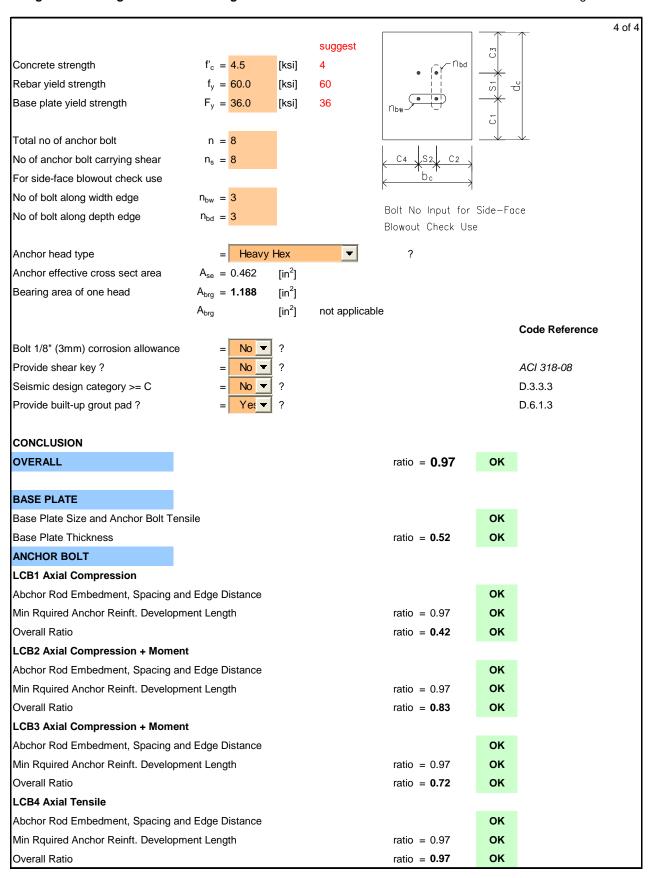
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BASE PLATE DESIGN

Base plate design based on

AISC Design Guide 1: Base Plate and Anchor Rod Design 2nd Edition

ACI 318-08 Building Code Requirements for Structural Concrete and Commentary

Code Abbreviation

AISC Design Guide 1

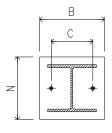
ACI 318-08

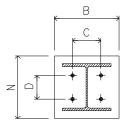
DESIGN DATA

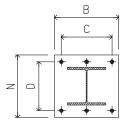
Column section type W_Shape
Column size W14X53

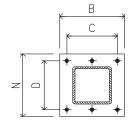
Depth d = 13.900 [in] Flange thickness $t_f = 0.660$ [in] Flange width $b_f = 8.060$ [in] Web thickness $t_w = 0.370$ [in]

Base plate anchor bolt pattern 4 or 6-Bolt MC WF base plate is moment connection









2-BOLT PIN

4-BOLT PIN

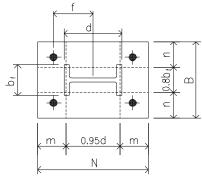
4 or 6-Bolt MC WF

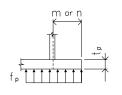
4 or 6-Bolt MC HS

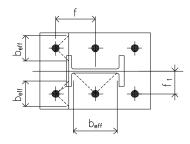
suggest

Base plate width B = 22.0[in] 15.0 N = 22.0Base plate depth 21.0 [in] Base plate thickness $t_{\rm p} = 2.000$ [in] 1.8 C = 18.0Anchor bolt spacing [in] 11.0 Anchor bolt spacing D = 18.017.0 [in]

Anchor bolt diameter d = 0.875 [in] max 1.5 in







BASE PLATE GEOMETRIC

BASE PLATE SUBJECT TO TENSILE LOAD

Pedestal width $b_c = 124.0$ [in] >= 28.5 in Pedestal depth $d_c = 124.0$ [in] >= 28.5 in

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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					

suggest max plate thickness 1.75 in

Suggested plate thickness for rigidity: $t_p = \text{max. of } m/4 \text{ and } n/4 = \text{No}$

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 $A_{se} = 0.462$ [in²]

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 $\phi_c = 0.65$ 9.3.2.4

Base plate bending $\phi_b = 0.90$

CONCLUSION

[Base Plate Size and Anchor Bolt Tensile Is Adequate]

oĸ

[The Base Plate Thickness Is Adequate] ratio= 0.52

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DESIGN CHECK				3 of 6
For base plate subject to large mom	ent			Code Reference ACI 318-08
Anchor rod tensile resistance	$T_r = \phi_{t,s} n_t A_{se} f_{uta}$ $\phi_{t,s} = 0.75$ for ductile steel element			
W Shapes	m = (N - 0.95d) / 2 $n = (B - 0.8b_f) / 2$	= 4.40 = 7.78	[in] [in]	AISC Design Guide 1 3.1.2 on Page 15
HSS Rectangle Shapes	m = (N - 0.95d) / 2 $n = (B - 0.95b_f) / 2$	= 4.40 = 7.17	[in]	3.1.3 on Page 16
HSS Round Shapes	m = (N - 0.8d) / 2 $n = (B - 0.8b_f) / 2$	= 5.44 = 5.44	[in]	3.1.3 on Page 16
m value used for design	m =	= 4.40	[in]	
n value used for design	n =	= 7.78	[in]	
Suggested plate thickness for rigidity	y: $t_p = max$. of m/4 and n/4	= 1.94	[in]	
Base plate area	$A_1 = B \times N$	= 484.0	[in ²]	
Pedestal area	$A_2 = b_c x d_c$	= 15376.0	[in ²]	
		0.000		ACI 318-08
	$k = min [sqrt(A_2/A_1), 2]$ $\phi_c P_n = \phi_c 0.85 f_c' A_1 k$	= 2.000 = 2406.7	[kips]	10.14.1 10.14.1
	$\psi_{cl} = \psi_{c} 0.03 I_{c} A_{1} K$	- 2400.7 > P _u	OK	10.14.1
LCB1: Axial Compressive		, . u		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
	$\lambda = \min(\frac{2\sqrt{X}}{1+\sqrt{1-X}}, 1)$	= 0.2		
	$\lambda n' = \lambda \operatorname{sqrt}(\operatorname{dxb}_f) / 4$	= 0.53	[in]	
For W shape	$L = max (m, n, \lambda n')$	= 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]	
Base Plate B x N OK				

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LCB2: Axial Compression + Mon	nent				Code Reference
	$e = M_u / P_u$ $f_{p(max)} = \phi_c 0.85 f_c' Q_{max} = f_{p(max)} x B$ $e_{crit} = N/2 - P_u / (3)$	2q _{max})	$M_u = 30.0$ $= 3600.00$ $= 4.97$ $= 109.40$ $= 11.00$	[kip-ft] [in] [ksi] [kips/in] [in]	
	e > e _{crit} l	Large moment case appli	eu		
Small moment case Bearing length	This case does not ap $Y = N - 2e$ $q = P_u / Y$	oply	= 0.00	[in]	AISC Design Guide 1
Verify linear bearing pressure If Y>=m	$f_p = P_u / BY$ $m = max(m, n)$ $t_{req1} = 1.49m \text{ sqrt}$	(f_p/F_y)	= 0.00 < q _{max} = 0.00 = 7.78 = 0.00	OK [ksi] [in]	Eq. 3.3.14a-1
If Y <m< td=""><td>$t_{\text{req2}} = 2.11 \sqrt{\frac{f_p Y \left(n\right)}{F}}$</td><td>$\frac{n-\frac{Y}{2}}{y}$</td><td>= 0.00</td><td>[in]</td><td>Eq. 3.3.15a-1</td></m<>	$t_{\text{req2}} = 2.11 \sqrt{\frac{f_p Y \left(n\right)}{F}}$	$\frac{n-\frac{Y}{2}}{y}$	= 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	$var_1 = (f + N/2)^2$ $var_2 = 2P_u (e+f) /$ $var_1 > var_2$	q _{max}	= 400 = 7	[in ²] [in ²] OK	
Bearing length	$Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\frac{N}{2}}$	$\left(f + \frac{N}{2}\right)^2 - \frac{2P_u(e+f)}{q_{max}}$	= 0.17	[in]	Eq. 3.4.3
Anchor rod tension force	$T_{u} = q_{max}Y - P_{u}$ ratio = 0.14		= 18.0 < T _r	[kips]	Eq. 3.4.2
At anchor rod tension interface					
	$x = f - d/2 + t_f /$ $t_{req-t} = 2.11 \sqrt{\frac{T_u \times x}{BF_y}}$		= 2.38 = 0.49	[in]	Eq. 3.4.6 Eq. 3.4.7a
At conc. bearing interface					
If Y>=m	m = max(m, n) $t_{req-b} = 1.49m sqrt$	$(f_{p(max)}/F_y)$	= 7.78 = 0.00	[in] [in]	Eq. 3.3.14a-2
If Y <m< td=""><td>$t_{\text{req-b}} = 2.11 \sqrt{\frac{f_{p(\text{max})}}{f_{p(\text{max})}}}$</td><td>$\frac{Y\left(m-\frac{Y}{2}\right)}{F_{y}}$</td><td>= 0.89</td><td>[in]</td><td>Eq. 3.3.15a-2</td></m<>	$t_{\text{req-b}} = 2.11 \sqrt{\frac{f_{p(\text{max})}}{f_{p(\text{max})}}}$	$\frac{Y\left(m-\frac{Y}{2}\right)}{F_{y}}$	= 0.89	[in]	Eq. 3.3.15a-2
Base Plate B x N OK	$t_{min} = max (t_{req-t},$	$t_{\text{req-b}}$)	= 0.89	[in]	

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LCB3: Axial Compression + Mon	nent					Code Reference
	P _u =	15.0 [kips]	$M_u = 30.0$	[kip-ft]	
	e =	M_u / P_u		= 24.00	[in]	
	f _{p(max)} =	φ _c 0.85 f _c '	k	= 4.97	[ksi]	
	$q_{max} =$	f _{p(max)} x B		= 109.40	[kips/in]	
	e _{crit} =	N/2 - P _u / (2	2q _{max})	= 10.93	[in]	
	e >	e _{crit} I	_arge moment case ap	pplied		
Small moment case	This case d	loes not ap	oply			AISC Design Guide 1
Bearing length	Y =	N - 2e		= 0.00	[in]	
Verify linear bearing pressure	q =	P_u/Y		= 0.00	[kips/in]	
THS SE	f _p = m = t _{red1} =	P _u / BY max(m , n) 1.49m sqrt	(fp/Fy)	< q _{max} = 0.00 = 7.78 = 0.00	OK [ksi] [in]	Eq. 3.3.14a-1
11 12-111				- 0.00	[]	_q. 0.0.1 ld 1
If Y <m< td=""><td>$t_{req2} = 2$</td><td>$2.11\sqrt{\frac{f_p Y(n)}{F}}$</td><td>$\left(\frac{11-\frac{1}{2}}{2}\right)$</td><td>= 0.00</td><td>[in]</td><td>Eq. 3.3.15a-1</td></m<>	$t_{req2} = 2$	$2.11\sqrt{\frac{f_p Y(n)}{F}}$	$\left(\frac{11-\frac{1}{2}}{2}\right)$	= 0.00	[in]	Eq. 3.3.15a-1
	t _{min} =	max (t _{req1} ,	t _{req2})	= 0.00	[in]	
Large moment case	This case a	pplies				
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			ОК	
Bearing length	Y =	$\left(f + \frac{N}{2}\right) \pm \sqrt{2}$	$\left(f + \frac{N}{2}\right)^2 - \frac{2P_u(e+f)}{q_{max}}$	= 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	ОК	
At anchor rod tension interface						
7 K dilonor log tonoidir interidee	x =	f - d/2 + t _f /	2	= 2.38	[in]	Eq. 3.4.6
	$t_{req-t} =$	$2.11\sqrt{\frac{T_u}{BF_y}}$		= 0.36	[in]	Eq. 3.4.7a
At conc. bearing interface						
	m =	max(m , n)		= 7.78	[in]	
If Y>=m	•		(f _{p(max)} / F _y)	= 0.00	[in]	Eq. 3.3.14a-2
If Y <m< td=""><td>$t_{req-b} = 2$</td><td>$2.11\sqrt{\frac{f_{p(max)}}{f_{p(max)}}}$</td><td>$\frac{Y\left(m-\frac{Y}{2}\right)}{F_{y}}$</td><td>= 1.04</td><td>[in]</td><td>Eq. 3.3.15a-2</td></m<>	$t_{req-b} = 2$	$2.11\sqrt{\frac{f_{p(max)}}{f_{p(max)}}}$	$\frac{Y\left(m-\frac{Y}{2}\right)}{F_{y}}$	= 1.04	[in]	Eq. 3.3.15a-2
	t _{min} =	max (t _{req-t} ,	t _{req-b})	= 1.04	[in]	
Base Plate B x N OK						

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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)

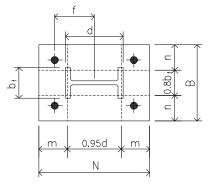
ratio = 0.05 > P_u **OK**

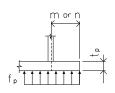
Bolt pattern Bolt Outside Flange Only

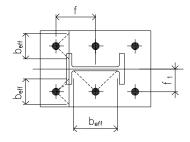
Total No of anchor bolt n = 8

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

 $\label{eq:moment_moment} \begin{tabular}{ll} Moment lever arm & a = 2.38 & [in] \\ Moment to column flange & M_u = 0.25 & [kip-ft] \\ \end{tabular}$

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

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

Bending to Column Web

 $\label{eq:moment_decomposition} \begin{tabular}{lll} Moment lever arm & a = 8.82 & [in] \\ Moment to column flange & M_u = 0.92 & [kip-ft] \\ \end{tabular}$

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

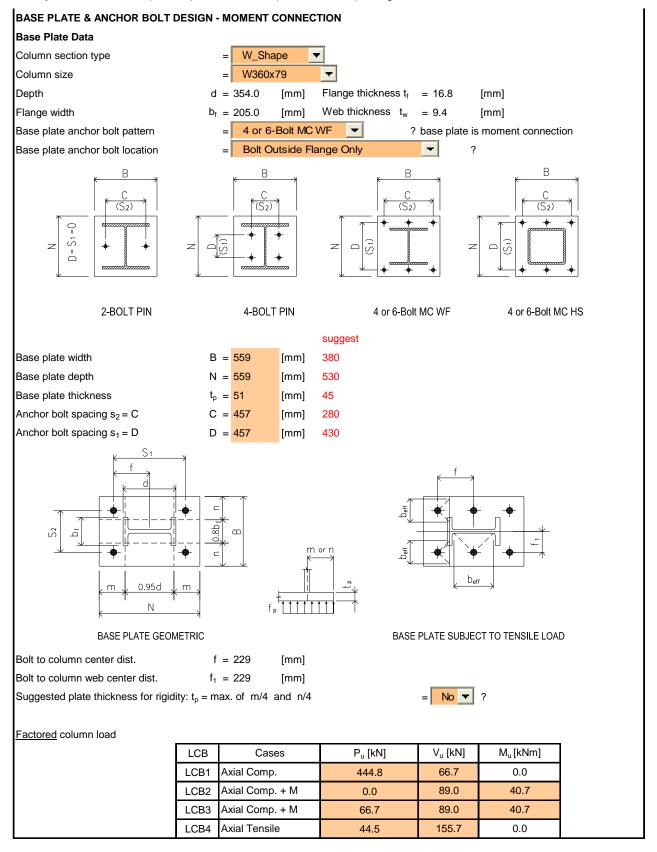
Base plate required thickness $t_{p2} = \sqrt{\frac{4 M_u}{b_{eff} \phi_b F_y}}$ = 0.00 [in]

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

Anchor Bolt Tensile OK

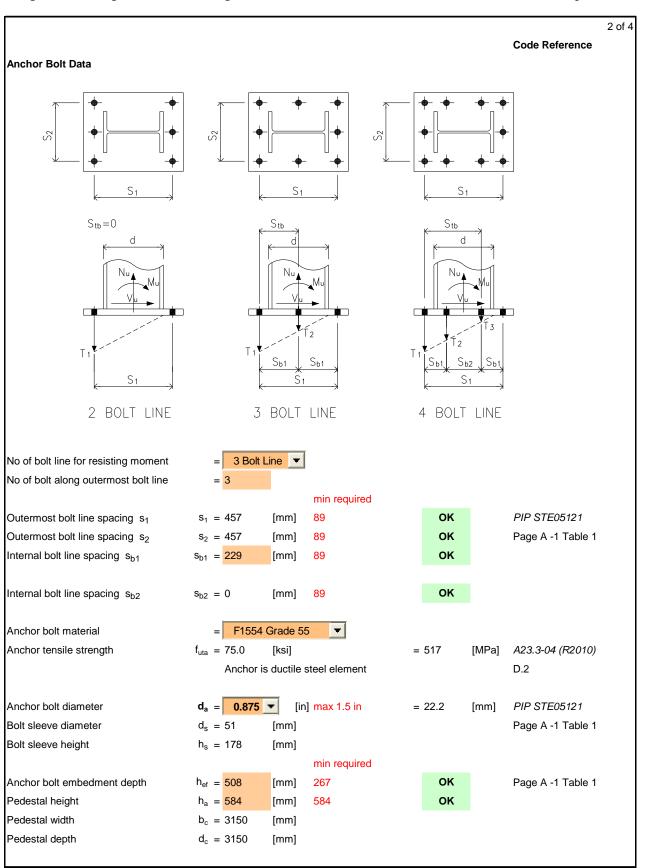
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Example 52: Base Plate (S16-09) & Anchor Bolt (CSA A23.3-04) Design With Anchor Reinforcement



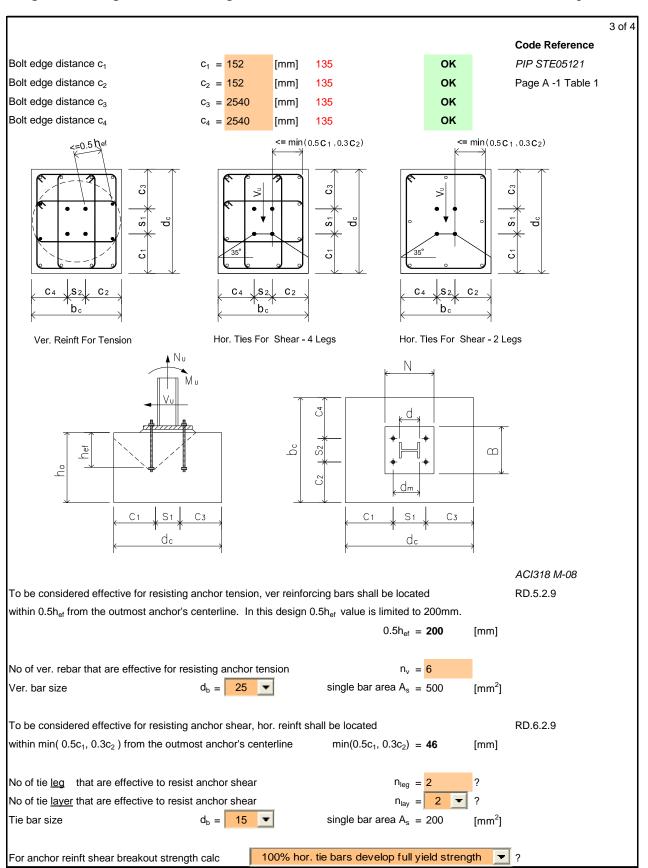
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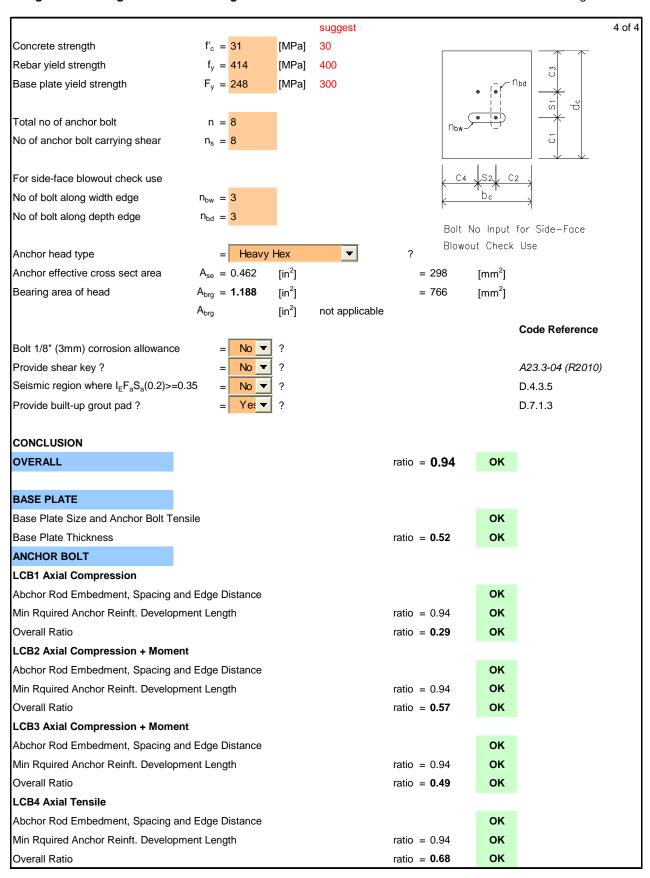
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BASE PLATE DESIGN

Base plate design based on

AISC Design Guide 1: Base Plate and Anchor Rod Design 2nd Edition

CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D

ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary

Code Abbreviation

AISC Design Guide 1

A23.3-04 (R2010)

ACI318 M-08

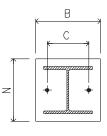
DESIGN DATA

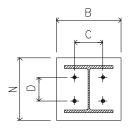
Column section type W_Shape
Column size W360x79

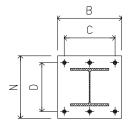
Depth d = 354.0 [mm] Flange thickness $t_f = 16.8$ [mm] Flange width $b_f = 205.0$ [mm] Web thickness $t_w = 9.4$ [mm]

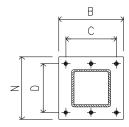
Base plate anchor bolt pattern 4 or 6-Bolt MC WF

base plate is moment connection









2-BOLT PIN

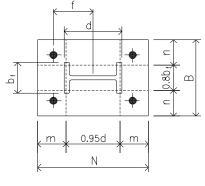
4-BOLT PIN

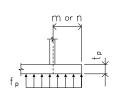
4 or 6-Bolt MC WF

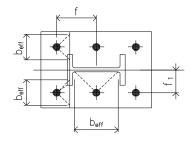
4 or 6-Bolt MC HS

suggest

Base plate width B = 559380 [mm] Base plate depth N = 559[mm] 530 Base plate thickness $t_{p} = 51$ [mm] 45 Anchor bolt spacing C = 457280 [mm] 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

Pedestal width $b_c = 3150$ [mm] >= 724 mm Pedestal depth $d_c = 3150$ [mm] >= 724 mm

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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					

suggest max plate thickness 45 mm

Suggested plate thickness for rigidity: $t_p = \text{max. of } m/4 \text{ and } n/4 = \text{No}$

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 $A_{se} = 0.462$ [in²] = 298 [mm²]

Concrete strength $f_c = 31$ [MPa] Base plate yield strength $F_y = 248$ [MPa]

Code Reference

Bearing on concrete $\phi_c = 0.65$ 8.4.2 Steel anchor resistance factor $\phi_s = 0.85$ 8.4.3 (a)

Base plate bending $\phi_b = 0.90$

CONCLUSION

Strength reduction factor

[Base Plate Size and Anchor Bolt Tensile Is Adequate]

OK

A23.3-04 (R2010)

[The Base Plate Thickness Is Adequate] ratio= 0.52

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DESIGN CHECK				3 of 6
For base plate subject to large mom	nent			Code Reference A23.3-04 (R2010)
Anchor rod tensile resistance	$T_r = n_t A_{se} \phi_s f_{uta} R_{t,s}$ $R_{t,s} = 0.80$ for ductile steel in terms	= 524.0 nsion	[kN]	D.6.1.2 (D-3) D.5.4(a)
W Shapes	m = (N - 0.95d) / 2 $n = (B - 0.8b_f) / 2$	= 111.3 = 197.4	[mm]	AISC Design Guide 1 3.1.2 on Page 15
HSS Rectangle Shapes	m = (N - 0.95d) / 2 $n = (B - 0.95b_f) / 2$	= 111.3 = 182.0	[mm]	3.1.3 on Page 16
HSS Round Shapes	m = (N - 0.8d) / 2 $n = (B - 0.8b_f) / 2$	= 137.8 = 137.8	[mm]	3.1.3 on Page 16
m value used for design	m =	= 111.3	[mm]	
n value used for design	n =	= 197.4	[mm]	
Suggested plate thickness for rigidit	y: $t_p = max$. of m/4 and n/4	= 49.4	[mm]	
Base plate area	$A_1 = B \times N$	= 3.1E+05	[mm ²]	
Pedestal area	$A_2 = b_c x d_c$	= 9.9E+06	[mm ²]	
	$k = min [sqrt(A_2/A_1), 2]$	= 2.00		A23.3-04 (R2010) 10.8.1
	$\phi_c P_n = \phi_c \ 0.85 \ f_c' \ A_1 \ k$	= 10696.4	[kN]	10.0.1
		> P _u	ОК	
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
	$\lambda = \min(\frac{2\sqrt{X}}{1 + \sqrt{1 - X}}, 1)$	= 0.2		
	$\lambda n' = \lambda \ \text{sqrt(} \ \text{dxb}_{\text{f}} \text{)} / 4$	= 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]	
	$t_p = L \sqrt{\frac{2 P_u}{\phi_b F_y B N}}$	= 22.3	[mm]	
Raco Diato P v N OK				
Base Plate B x N OK				

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LCB2: Axial Compression + Mon	nent				Code Reference
	$P_u = 0.1$	l [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]	
	$q_{max} = f_{p(n)}$	_{nax)} x B	= 19142	[N/mm]	
		2 - P _u / (2q _{max})	= 279.4	[mm]	
	e > e _{cr}		ent case applied		
Small moment case	This case doe	es not apply			AISC Design Guide 1
Bearing length	Y = N -		= 0.0	[mm]	-
Verify linear bearing pressure	$q = P_u$	/ Y	= 0	[N/mm]	
31	1 4		< q _{max}	ОК	
	$f_n = P_n$	/BY	= 0.0	[MPa]	
TURRE	m = ma	ax(m , n)	= 197.4	[mm]	ADIE
If Y>=m	$t_{reg1} = 1.4$	19m sart(f _a / F _a)	= 0.0	[mm]	Eq. 3.3.14a-1
	Ted!				
If Y <m< td=""><td>$t_{req2} = 2.1$</td><td>/ BY $49m$</td><td>= 0.0</td><td>[mm]</td><td>Eq. 3.3.15a-1</td></m<>	$t_{req2} = 2.1$	/ BY $49m $	= 0.0	[mm]	Eq. 3.3.15a-1
	t _{min} = ma	ax (t _{req1} , t _{req2})	= 0.0	[mm]	
Large moment case	This case app	olies			
Check if real solution of Y exist	var ₁ = (f +		= 258064	[mm ²]	
	•	' _u (e+f) / q _{max}	= 4255	[mm ²]	
	var ₁ > va		.200	OK	
	1911 1911				
Bearing length	Y = (f	$+\frac{N}{2}$ $\pm \sqrt{\left(f+\frac{N}{2}\right)^2}$ $-$	$\frac{2P_{u}(e+f)}{q_{max}} = 4.2$	[mm]	Eq. 3.4.3
Anchor rod tension force	$T_u = q_m$	_{ax} Y - P _u	= 80.4	[kN]	Eq. 3.4.2
	ratio = 0.1		< T _r	ОК	l [']
			•		
At anchor rod tension interface					
	x = f -	d/2 + t _f / 2	= 60.0	[mm]	Eq. 3.4.6
					•
	$t_{req-t} = 2.7$	$11\sqrt{\frac{T_u \times BF_y}{BF_y}}$	= 12.4	[mm]	Eq. 3.4.7a
At conc. bearing interface					
3	m = ma	ax(m . n)	= 197.4	[mm]	
If Y>=m		19m sqrt($f_{p(max)} / F_y$)	= 0.0	[mm]	Eq. 3.3.14a-2
	•		_ 0.0	[]	₁ . 0.0 <u>L</u>
If Y <m< td=""><td>$t_{req-b} = 2.1$</td><td>$1\sqrt{\frac{f_{p(max)} Y\left(m-\frac{Y}{2}\right)}{F_{y}}}$</td><td>= 22.5</td><td>[mm]</td><td>Eq. 3.3.15a-2</td></m<>	$t_{req-b} = 2.1$	$1\sqrt{\frac{f_{p(max)} Y\left(m-\frac{Y}{2}\right)}{F_{y}}}$	= 22.5	[mm]	Eq. 3.3.15a-2
	t _{min} = ma	ax (t _{req-t} , t _{req-b})	= 22.5	[mm]	
		/			

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LCB3: Axial Compression + Mon	nent				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]				
	$q_{max} = f_{p(max)} x B$	1	= 19142	[N/mm]				
	$e_{crit} = N/2 - P_u$	/ (2q _{max})	= 277.7	[mm]				
	e > e _{crit}	Large moment case a	pplied					
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}	ОК				
	$f_p = P_u / BY$		= 0.0	[MPa]				
TUICCE	m = max(m, m)	n)	= 197.4	[mm]	ADIE			
If Y>=m	$t_{req1} = 1.49 \text{m so}$	qrt(f _p / F _y)	= 0.0	[mm]	Eq. 3.3.14a-1			
	fnY	$\left(m-\frac{Y}{2}\right)$						
If Y <m< td=""><td>$f_{p} = P_{u}/BY$ $m = max(m, t_{req1} = 1.49m sc$ $t_{req2} = 2.11\sqrt{\frac{f_{p}Y}{m}}$</td><td><u>(2)</u> F_y</td><td>= 0.00</td><td>[mm]</td><td>Eq. 3.3.15a-1</td></m<>	$f_{p} = P_{u}/BY$ $m = max(m, t_{req1} = 1.49m sc$ $t_{req2} = 2.11\sqrt{\frac{f_{p}Y}{m}}$	<u>(2)</u> F _y	= 0.00	[mm]	Eq. 3.3.15a-1			
	$t_{min} = max (t_{rec})$	₁₁ , t _{req2})	= 0.0	[mm]				
Large moment case	This case applies							
Check if real solution of Y exist	$var_1 = (f + N/2)^{r/2}$	^2	= 258064	$[mm^2]$				
	$var_2 = 2P_u (e+f)$	/ q _{max}	= 5846	$[mm^2]$				
	$var_1 > var_2$			ок				
Bearing length	$Y = \left(f + \frac{N}{2}\right) \pm \frac{1}{2}$	$\pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_u(e + f)}{q_{max}}}$	= 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	ок				
At anchor rod tension interface								
	x = f - d/2 + 1	t _f / 2	= 60.0	[mm]	Eq. 3.4.6			
	$t_{\text{req-t}} = 2.11 \sqrt{\frac{T_u}{BF}}$	x y	= 9.2	[mm]	Eq. 3.4.7a			
At conc. bearing interface								
	m = max(m,	n)	= 197.4	[mm]				
If Y>=m	$t_{req-b} = 1.49 \text{m so}$		= 0.00	[mm]	Eq. 3.3.14a-2			
If Y <m< td=""><td>$t_{\text{req-b}} = 2.11 \sqrt{\frac{f_{\text{p(m)}}}{m}}$</td><td>$\frac{1}{F_y} \frac{Y\left(m - \frac{Y}{2}\right)}{F_y}$</td><td>= 26.3</td><td>[mm]</td><td>Eq. 3.3.15a-2</td></m<>	$t_{\text{req-b}} = 2.11 \sqrt{\frac{f_{\text{p(m)}}}{m}}$	$\frac{1}{F_y} \frac{Y\left(m - \frac{Y}{2}\right)}{F_y}$	= 26.3	[mm]	Eq. 3.3.15a-2			
	$t_{min} = max (t_{rec})$	_{q-t} , t _{req-b})	= 26.3	[mm]				
Base Plate B x N OK								

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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)

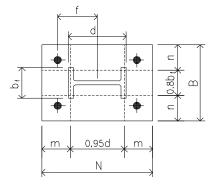
ratio = 0.05 > P_u **OK**

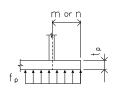
Bolt pattern Bolt Outside Flange Only

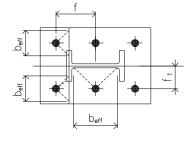
Total No of anchor bolt n = 8

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 $b_{eff} = 2 x a$ = 120 [mm]

Base plate required thickness $t_{p1} = \sqrt{\frac{4 \text{ M}_u}{b_{\text{eff}} \phi_b \text{ F}_v}} = 7.1 \text{ [mm]}$

Bending to Column Web

Moment lever arm a = 224 [mm] Moment to column flange $M_u = 1.2$ [kNm]

Effective plate width $b_{eff} = 2 x a$ = 448 [mm]

Base plate required thickness $t_{p2} = \sqrt{\frac{4 \text{ M}_u}{b_{\text{eff}} \phi_b \text{ F}_y}}$ = 0.0 [mm]

 $t_{min} = max(t_{p1}, t_{p2}) = 7.1$ [mm]

Anchor Bolt Tensile OK



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3.0 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

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