ACI 349.2R-97 (Reapproved 2002)

Embedment Design Examples

Reported by ACI Committee 349

Charles A. Zalesiak Chairman

Gunnar A. Harstead

Christopher Heinz

Charles J. Hookham

Richard E. Klingner

Timothy J. Lynch

Dragos A. Nuta

Hans G. Ashar Ranjit Bandyopadhyay * Ronald A. Cook* Jack M. Daly Arobindo Dutt Branko Galunic Dwaine A. Godfrey Herman L. Graves III

* Major contributor to the report † Deceased

Appendix B of ACI 349 was developed to better define the design requirements for steel embedmnts revisions are periodically made to the code as a result of on-going research and testing. As with other concretebuilding codes, the design of embedments attempts to assure a ductile failure mode so that the reinforcement yields before the concrete fails. In embedments designed for direct loading, the concrete pullout strength must be greater than the tensile strength of the steel. This report presents a series of design examples of ductile steel embedments. These examples have been updated to include the revision incorparated in Appendix B of ACI 349-97.

Keywords: Anchorage (structural); anchor bolts; anchors (fasteners); embedment; inserts; loads (forces); load transfer; moments; reinforced concrete; reinforcing steels; shear strength; structural design; studs; tension.

CONTENTS

Introduction	• • • • • • •	 p. 349.2R-2
Notation		 . p. 349.2R-2

ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in planning, designing, executing, and inspecting construction. This document is intended for the use of individuals who are competent to evaluate the significance and limitations of its content and recommendations and who will accept responsibility for the application of the material it contains. The American Concrete Institute disclaims any and all responsibility for the stated principles. The Institute shall not be liable for any loss or damage arising therefrom.

Reference to this document shall not be made in contract documents. If items found in this document are desired by the Architect/Engineer to be a part of the contract documents, they shall be restated in mandatory language for incorporation by the Architect/ Engineer.

Richard S. Orr* Robert B. Pan Julius V. Rotz[†] Robert W. Talmadge Chen P. Tan Frederick L. Moreadith Richard E. Toland Donald T. Ward Albert Y. C. Wong

> PART A—Examples: Ductile single embedded element in semi-in nite concrete. . . . p. 349.2R-3

Example A1	Single stud, tension only
Example A2	Single stud, shear only
Example A3	Single stud, combined tension and shear
Example A4	Anchor bolt, combined tension and shear
Example A5	Single rebar, combined tension and shear

PART B-Examples: Ductile multiple embedded elements in semi-in nite concrete. .p. 349.2R-10

Example B1	Four-stud rigid embedded plate, tension only
Example B2(a)	Four-stud rigid embedded plate,
	combined shear and uniaxial moment
Example B2(b)	Four-stud flexible embedded plate,
	combined shear and uniaxial moment
Example B2(c)	Four-bolt rigid surface-mounted plate,
	combined shear and uniaxial moment
Example B3(a)	Four-stud rigid embedded plate,
	combined tension, shear, and
	uniaxial moment
Example B3(b)	Four-stud flexible embedded plate,
	combined tension, shear, and
	uniaxial moment
Example B4	Four-stud rigid embedded plate in thin slab,
	tension only
APPENDIX A	—Projected area (A _{cp})

ACI 349.2R-97 became effective October 16, 1997. Copyright © 2002, American Concrete Institute.

All rights reserved including rights of reproduction and use in any form or by any means, including the making of copies by any photo process, or by electronic or mechanical device, printed, written, or oral, or recording for sound or visual reproduc-

tion or for use in any knowledge or retrieval system or device, unless permission in writing is obtained from the copyright proprietors.

INTRODUCTION

This report has been prepared by members of the ACI 349 Sub-Committee on Steel Embedments to provide examples of the application of the ACI 349 Code to the design of steel embedments. The ACI 349 Committee was charged in 1973 with preparation of the code covering concrete structures in nuclear power plants. At that time, it was recognized that design requirements for steel embedments were not well defined and a special working group was established to develop code requirements. After much discussion and many drafts, Appendix B was approved and issued in the 1978 Supplement of ACI 349 covering the design of steel embedments. Subsequently, the Sub-Committee has continued to monitor on-going research and testing and to incorporate experience of applying the Code. Periodic revisions have been made to the Code and Appendix B.

The underlying philosophy in the design of embedments is to attempt to assure a ductile failure mode. This is similar to the philosophy of the rest of the concrete building codes wherein, for example, flexural steel for a beam is limited to assure that the reinforcement steel yields before the concrete crushes. In the design of an embedment for direct loading, the philosophy leads to the requirement that the concrete pull-out strength must be greater than the tensile strength of the steel.

This report includes a series of design examples starting with simple cases and extending to more complex cases for ductile embedments. The format for each example follows the format of the Strength Design Handbook, SP-17, and provides a reference back to the code paragraph for each calculation procedure.

NOTATION

- а depth of equivalent stress block, in.
- effective stress area defined by the projected area $A_{CD} =$ of the 45 degree stress cone radiating towards the attachment from the bearing edge of the anchor, sq. in.
- $A_c =$ effective stress area of anchor, sq. in.
- $A_h =$ area of anchor head, sq. in.
- $A_s =$ area of steel, sq. in.
- $A_{st} =$ area of steel required to resist tension, sq. in.
- $A_{sv} =$ area of steel required to resist shear, sq. in.

- reduction in effective stress area to account for $A_r =$ limited depth of concrete beyond the bearing surface of the embedment, sq. in.
- $A_{vf} =$ area of shear friction reinforcement, sq. in.
- b = width of embedded or surface mounted plate, or width of an anchor group, measured out to out of bearing edges of the outermost anchor heads, in.
- В = overlapping stress cone factor (see Appendix A)
- С = spacing or cover dimension, in.
- С = compressive reaction
- $d_b =$ nominal diameter of reinforcing bar, in.
- $d_h =$ diameter of anchor head or reinforcing bar, in.
- ds = diameter of tensile stress component, in.
- F_v = specified yield strength of steel plate, psi
- f'_c = specified compressive strength of concrete, psi
- specified tensile strength of steel, psi
- specified yield strength of steel, psi
- $f_{ut} = f_y = h =$ overall thickness of concrete member, in.
- $k_{tr} =$ transverse reinforcement index
- /_d = development length, in.
- $L_d =$ embedment depth of anchor head measured from attachment of anchor head to tensile stress component, to the concrete surface, in.
- $M_n =$ nominal moment strength
- $M_u =$ factored moment load on embedment
- $M_V =$ elastic moment capacity of steel plate
- n´ = number of threads per inch
- $P_d =$ design pullout strength of concrete in tension
- $P_n =$ nominal axial strength
- $P_u =$ factored external axial load on the anchorage
- R = radius of 45 degree stress cone, in. (see A_{CD})
- S = spacing between anchors, in.
- t = thickness of plate, in.
- Т = tension force
- T_h = thickness of anchor head, in.
- V_n = nominal shear strength
- Vu = factored shear load on embedments
- α = reinforcement location factor
- β coating factor =
- reinforcement size factor = γ
- = μ coefficient of friction
- φ = strength reduction factor

PART A

EXAMPLES: Ductile single embedded element in semi-infinite concrete

Example A1	Single stud, tension only
Example A2	Single stud, shear only
Example A3	Single stud, combined tension and shear
Example A4	Anchor bolt, combined tension and shear
Example A5	Single rebar, combined tension and shear

Example A1—Single stud, tension only

Design an embedment using a stud welded to an embedded plate.

Given:

 $f'_c = 4000 \text{ psi}$ $f_y = 50,000 \text{ psi}$ $f_{ut} = 60,000 \text{ psi}$ $P_u = 8 \text{ kips}$ where P_u is the required factored external load as defined in Section 9.2 of the Code.



CODE SECTION	DESIGN PROCEDURE	CALCULATION			
	STEP 1: Determine required steel area of the stud				
	Assume that the load is applied directly over the stud and that a plate size of 3 in. \times 3 in. \times 3/ ₈ in. has been established by requirements of the attachment.				
B.6.5.1	Equate the external (required strength) and internal (design strength) forces and solve for the required steel area for the stud.	$P_u = \phi P_n = \phi A_s f_y$ $A_s = 8/[(0.9)(50)] = 0.18 \text{ in.}^2$ Use one $\frac{1}{2}$ in. diameter stud, $A_s = 0.196 \text{ in.}^2 > 0.18 \text{ in.}^2$ OK			
	STEP 2: Check anchor head bearing				
B.5.1.1(a) B.4.5.2	 a) Area of the anchor head (A_h) (including the area of the tensile stress component) is at least 2.5 times the area of the tensile stress component. b) Thickness of the anchor heat (T_h) is at least 1.0 times the greatest dimension from the outer most bearing edge of the anchor head to the face of the tensile stress component. c) Bearing area of head is approximately evenly distributed around the perimeter of the tensile stress component. 	$A_{h} = \pi (d_{h}/2)^{2} = 0.79 \text{ in.}^{2}$ (per manufacturer's data, $d_{h} = 1 \text{ in.}$) $A_{h}/A_{s} = 0.79/0.196 = 4 > 2.5 \text{OK}$ $T_{h} = 0.312 \text{ in.} \text{ (per manufacturer's data)}$ $(d_{h} - d_{s})/2 = 0.25 \text{ in.}$ $T_{h} = 0.312 > 0.25 \text{OK}$ Head and tensile stress component are concentric. OK			
STEP 3: Determine required embedment length for the stud to prevent concrete cone failure					
B.5.1.1 B.4.2	The design pullout strength of the concrete, P_d , must exceed the minimum specified tensile strength $(A_s f_{ut})$ of the tensile stress component. $P_d > A_s f_{ut}$ $P_d = \phi 4 \sqrt{f'_c} A_{cp}$ $A_{cp} = \pi [(L_d + d_h/2)^2 - (d_h/2)^2]$ Compute L_d from the equation: $\pi [L_d + d_h/2)^2 - (d_h/2)^2] \phi 4 \sqrt{f'_c} \ge A_s f_{ut}$	$\begin{array}{l} A_s f_{ut} = 0.196 \times 60 = 11.8 \ \text{kips} \\ \phi 4 \ \sqrt{f_c'} = 0.65 \times 4 \times \ \sqrt{(4000)} \\ = 165 \ \text{psi} \ (\text{see Note 2}) \\ \pi [(L_d + 0.5)^2 - 0.5^2] 0.165 \ge 11.8 \\ L_d (L_d + 1.0) \ge 22.8 \\ L_d^2 + L_d - 22.8 \ge 0 \\ L_d \ge 4.30 \ \text{in.} \\ \text{Use } \frac{1}{2} \ \text{in.} \ \text{diameter stud } 1^{-5}_{16} \ \text{in.} \ \text{long,} \\ \text{which has an effective length of } 4.87 \ \text{in.} \\ \text{giving } L_d = 4.87 + 0.38 = 5.25 \ \text{in.} \end{array}$			

Example A1, continued

CODE SECTION	DESIGN PROCEDURE	CALCULATION
	STEP 4: Check plate thickness	
	Since the load is applied directly over the stud, the only requirement on plate thickness is that it satisfy the minimum thickness required for stud welding.	Stud welding of $1/2$ in. diameter studs is acceptable on $3/8$ in. thick plate per manufacturer. OK

- NOTE: 1) In the above example, the embedment length L_d is taken to the face of the concrete. If the plate were larger than the stress cone, then the embedment length would exclude the thickness of the embedded plate.
 - 2) In all design examples, the strength reduction factor ϕ for concrete pullout is taken as 0.65 per Category (d) of Section B.4.2.

Example A2—Single stud, shear only

Design an embedment using a stud welded to an embedded plate.

Given:

 $f'_{c} = 4000 \text{ psi}$ $f_{y} = 50,000 \text{ psi}$ $f_{ut} = 60,000 \text{ psi}$ $V_{u} = 6 \text{ kips}$

where V_u is the required factored external load as defined in Section 9.2 of the Code.



CODE SECTION	DESIGN PROCEDURE	CALCULATION	
	STEP 1: Determine required steel area of the s	tud	
B.6.5.2.2	Use the shear friction provision of Section 11.7 with $\phi = 0.85$, $\mu = 0.9$. Equate the external (required strength) and internal (design strength) forces and solve for the required steel area for the stud.	$V_{u} = \phi V_{n} = \phi \mu A_{vf} f_{y}$ $A_{vf} = V_{u} / (\phi \mu f_{y})$ $A_{vf} = 6 / (0.85 \times 0.9 \times 50) = 0.16 \text{ in.}^{2}$ Use one ¹ / ₂ in. diameter stud, $A_{s} = 0.196 \text{ in.}^{2} > 0.16 \text{ in.}^{2}$	
	STEP 2: Check anchor head bearing		
B.4.5.2	a) Procedure is identical to that in Example A1	$A_h/A_s = 0.79/0.196 = 4 > 2.5$ OK	
	b) Procedure is identical to that in Example A1	$T_h = 0.312$ in. (per manufacturer's data) $(d_h - d_s)/2 = 0.25$ in. $T_h = 0.312 > 0.25$ OK	
	c) Procedure is identical to that in Example A1	Head and tensile stress component are concentric.	
STEP 3: Determine required embedment length for the stud to prevent concrete cone failure			
B.4.5.2 B.5.1.1	Procedure is identical to that in Example A1 since tensile capacity of the stud must be developed.	Use $1/_2$ in. diameter stud $5-3/_{16}$ in. long (see calculation in Example A1)	
	STEP 4: Check plate thickness		
	Select plate thickness such that $d_s/t < 2.7^*$	t > 0.5/2.7 = 0.185 ³ /8 in. thick plate is OK.	

- NOTE: The provisions of Section 11.7.5 on shear strength are not applicable at the surface between the steel plate and the concrete. Shear loads at this interface are carried by local bearing and wedge action as described in commentary Section B.4.3.
- * Ref.: "Shear Strength of Thin Flange Composite Sections," G. G. Goble, *AISC Engineering Journal*, April, 1968.

Example A3—Single stud, combined tension and shear

Design an embedment using a stud welded to an embedded plate.

Given:

 $f'_{c} = 4000 \text{ psi}$ $f_{y} = 50,000 \text{ psi}$ $f_{ut} = 60,000 \text{ psi}$ $P_{u} = 4 \text{ kips}$ $V_{u} = 2 \text{ kips}$

where P_u and V_u are the required factored external loads as defined in Section 9.2 of the Code.



CODE SECTION	DESIGN PROCEDURE	CALCULATION		
	STEP 1: Determine required steel area of the st	tud		
B.6.5.1	Equate the external (required strength) and internal (design strength) tension forces and solve for the required steel area for tension.	$P_u = \phi P_n = \phi A_{st} f_y$ $A_{st} = 4/(0.9 \times 50) = 0.09 \text{ in.}^2$		
B.6.5.2.2	Use the shear friction provision of Section 11.7 with φ = 0.85, μ = 0.9.	$V_{u} = \phi V_{n} = \phi \mu A_{sv} f_{y}$ $A_{sv} = V_{u} / (\phi \mu f_{y})$		
11.7 Eq. (11-26)	Equate the external (required strength) and internal (design strength) forces and solve for the required steel area for shear.	$A_{sv} = 2/(0.85 \times 0.9 \times 50) = 0.05 \text{ in.}^2$		
B.6.5.3.2	Sum the area of steel required for tension with the area of steel required for shear. Total Area $A_s = A_{st} + A_{sv}$	$A_s = 0.09 + 0.05 = 0.14$ in. ² Use one ¹ / ₂ in. diameter stud, $A_s = 0.196$ in. ² > 0.14 in. ² OK		
STEP 2: Check anchor head bearing				
B.4.5.2	a) Procedure is identical to that in Example A1	A _h /A _s = 0.79/0.196 = 4 > 2.5 OK		
	b) Procedure is identical to that in Example A1	$T_h = 0.312$ in. (per manufacturer's data) $(d_h - d_s)/2 = 0.25$ in. $T_h = 0.312 > 0.25$ OK		
	c) Procedure is identical to that in Example A1	Head and tensile stress component are concentric. OK		
STEP 3: Determine required embedment length for the stud to prevent concrete cone failure				
B.4.2 B.5.1.1	Procedure is identical to that in Example A1	Use $\frac{1}{2}$ in. diameter stud $\frac{5-3}{16}$ in. long (see calculation in Example A1)		
	STEP 4: Calculate minimum plate thickness			
	Select plate thickness such that $d_s/t < 2.7^*$	t > 0.5/2.7 = 0.185 3_{8} in. thick plate is OK		

NOTE: The provisions of Section 11.7.5 on shear strength are not applicable at the surface between the steel plate and the concrete. Shear loads at this interface are carried by local bearing and wedge action as described in commentary Section B.4.3.

* Ref.: "Shear Strength of Thin Flange Composite Sections," G. G. Goble, AISC Engineering Journal, April, 1968.

Example A4—Single bolt, combined tension and shear

Design an embedment using a high strength bolt (A 325).

Given:

 $f'_{c} = 4000 \text{ psi}$ $f_{y} = 81,000 \text{ psi}$ $f_{ut} = 105,000 \text{ psi}$ $P_{u} = 40 \text{ kips}$ $V_{u} = 20 \text{ kips}$

where P_u and V_u are the required factored external loads as defined in Section 9.2 of the Code.



CODE SECTION	DESIGN PROCEDURE	CALCULATION		
STEP 1: Determine required steel area of the stud				
B.6.5.1	Equate the external (required strength) and internal (design strength) tension forces and solve for the required steel area for tension.	$P_u = \phi P_n = \phi A_{st} f_y$ $A_{st} = 40/(0.9 \times 81) = 0.55 \text{ in.}^2$		
B.6.5.2.1	Use provision for contact surface of the base plate flush with the surface of the concrete, $\phi = 0.85$.	$V_u = \phi V_n = \phi (0.7 f_y A_{sv})$ $A_{sv} = V_u / [(0.7)(\phi f_y)]$		
	Equate the external (required strength) and internal (design strength) forces and solve for the required steel area for shear.	$A_{sv} = 20/(0.7 \times 0.85 \times 81) = 0.41 \text{ in.}^2$		
B.6.5.3.2	Sum the area of steel required for tension with the area of steel required for shear. Total Area $A_s = A_{st} + A_{sv}$	$A_s = 0.55 + 0.41 = 0.96 \text{ in.}^2$ Use one 1- ¹ / ₄ in. diameter bolt, 7 threads per inch. Tensile stress area = 0.97 in. ² $A_s = 0.97 \text{ in.}^2 > 0.96 \text{ in.}^2$ OK		
STEP 2: Check anchor head bearing				
B.4.5.2	a) Procedure is identical to that in Example A1	A 325 Heavy Hex Head for $1-1/4$ in. diameter bolt width across flats = 2.0 in., thickness = 0.78 in.		
		$A_h = (1.0)^2 \times 2 \times \sqrt{3} = 3.46 \text{ in.}^2$ $A_h/A_s = 3.46/0.97 = 3.57$ > 2.5 OK		
	b) Procedure is identical to that in Example A1	$(d_h - d_s)/2 = (2 \times 2\sqrt{3} - 1.25)/2$ = 0.53 in. $T_h = 0.78 > 0.53$ OK		
	c) Procedure is identical to that in Example A1	Head and tensile stress component are concentric.		
STEP 3: Determine required embedment length for the bolt to prevent concrete cone failure				
B.4.2	Procedure is identical to that in Example A1	$A_s f_{ut} = 0.97 \times 105 = 102$ kips		
B.5.1.1		$\phi 4 \sqrt{f'_c} = 0.65 \times 4 \times \sqrt{(4000)} = 165 \text{ psi}$ [$(L_d + 1.125)^2 - 1.125^2$]0.165 ≥ 102 $L_d (L_d + 2.25) \ge 196.8$		
		$L_d \ge -1.125 + \sqrt{(1.125^2 + 196.8)} = 12.95$ in.		

 $P_u = 15$ kips

Example A5—Single rebar, combined tension and shear

Design an embedment using a straight reinforcing bar welded to an embedment plate.

Given:

 $f'_c = 4000 \text{ psi}$ $f_v = 60,000 \text{ psi}$ $(\leq 60,000 \text{ OK per Code Section } 3.5.3.3)$ $f_{ut} = 90,000$ psi (based on typical test results) $\tilde{P}_u = 15$ kips $V_{ii} = 5$ kips

where P_{ii} and V_{ii} are the required factored external loads as defined in Section 9.2 of the Code.



and plate per AWS D1.4

PART B

EXAMPLES: Ductile multiple embedded element in semi-infinite concrete

Example B1	Four-stud rigid embedded plate, tension only
Example B2(a)	Four-stud rigid embedded plate, combined shear and uniaxial moment
Example B2(b)	Four-stud flexible embedded plate, combined shear and uniaxial moment
Example B2(c)	Four-bolt rigid surface-mounted plate, combined shear and uniaxial moment
Example B3(a)	Four-stud rigid embedded plate, combined tension, shear, and uniaxial moment
Example B3(b)	Four-stud flexible embedded plate, combined tension, shear, and uniaxial moment
Example B4	Four-stud rigid embedded plate in thin slab, tension only

Example B1—Four-stud rigid embedded plate, tension only

Design an embedment with four welded studs and a rigid embedded plate for a $3 \times 3 \times 3/_{16}$ in. A 501 structural tube attachment.

Given:

$f'_{c} = 4000$ $f_{y} = 50,00$ $f_{ut} = 60,00$ $F_{y} = 36,00$ $P_{u} = 18 \text{ kip}$ where P_{u} is the re-	psi 0 psi (studs) 0 psi 0 psi (plate) os equired factored external load ion 9.2 of the Code.	+ +	+	$P_{u} = $	18 kips •
CODE SECTION	DESIGN PROCEDURE		(
STEP 1: Determine required steel area of the stud					
B.6.5.1	Equate the external (required strength) and internal (design strength) forces and solve for the required steel area for the stud.		$P_u = \phi P_n = \phi \mu$ $A_s = 18/45 = 0$ Use four ³ / ₈ in	. <i>A_sf_y</i> 0.40 in. ² . diameter studs,	

B.6.5.1	Equate the external (required strength) and internal (design strength) forces and solve for the required steel area for the stud.	$P_u = \phi P_n = \phi \mu A_s f_y$ $A_s = 18/45 = 0.40 \text{ in.}^2$ Use four ³ / ₈ in. diameter studs, $A_s = 0.442 \text{ in.}^2 > 0.40 \text{ in.}^2$ OK
	STEP 2: Check anchor head bearing	
B.4.5.2	a) Procedure is identical to that in Example A1	$A_h/A_s = 0.79/0.196 = 4 > 2.5$ OK
	b) Procedure is identical to that in Example A1	$T_h = 0.312$ in. (per manufacturer's data) $(d_h - d_s)/2 = 0.25$ in.
		$T_h = 0.312 > 0.25$ OK
	c) Procedure is identical to that in Example A1	Head and tensile stress component are concentric. OK
	STEP 3: Determine required stud spacing and to prevent concrete pullout	embedment length
B.5.1.1	The design pullout strength of the concrete, P_d , must exceed the minimum specified tensile strength of the tensile stress components. $P_d \ge A_s f_{ut}$ $P_d = \phi 4 \sqrt{f_c} A_{cp}$ Where A_{cp} = the projected area of the 45 deg stress cones radiating toward the attachment from the bearing edge of the anchors. This area must be limited by overlapping stress cones and by the bearing area of the anchor heads.	$A_{s}f_{ut} = 4 \times 0.110 \times 60 = 26.4 \text{ kips}$ $\phi 4 \sqrt{f_{c}'} = 0.65 \times 4 \times \sqrt{(4000)} = 165 \text{ psi}$ $A_{cp \ min} = A_{s}f_{ut}/(\phi 4 \sqrt{f_{c}'})$ $= 26,400/165 = 160.6 \text{ in.}^{2}$

Example B1, continued

CODE SECTION	DESIGN PROCEDURE	CALCULATION
	STEP 3: (continued)	
	For a four-stud plate with studs at spacing <i>S</i> and radius <i>R</i> of the projected stress cone, the projected area is (see Appendix A) $A_{cp} = (4\pi - 2B)R^2 - 4A_h \qquad (2R > S > \sqrt{2}R)$ $A_{cp} = (3\pi - B)R^2 + S^2 - 4A_h \qquad (\sqrt{2}R > S)$ The radius of the projected stress cone is $L_d + d_h/2$ at the underside of the embedded plate, and $L_d + d_h/2 + t$ at the outer surface of the concrete and plate. Conservatively neglect the thickness of the plate <i>t</i> .	The projected area of the stress cones may be calculated for each standard stud length (L_d) and a range of stud spacings, <i>S</i> (see Table B1-1). All values greater than: $A_{cp\ min} = A_s f_{ut} / (\phi 4 \sqrt{f_c'})$ are then satisfactory. Select $3/_8$ in. $\times 4^{-1}/_8$ in. stud with effective length of 3.71 in. at 6 in. spacing.
	NOTE: The above calculation utilizes an exact calculation of the projected area. In many cases, an approximate calculation is sufficient. Such a method is used in Example B2. For the stud configuration selected above ($R = 4.09$ in., S = 6 in.), the approximate method would give: $A_{cp} = 6 \times 6 + 4 \times 6 \times 4.09 + \pi \times 4.09^2$ $= 4 \times \pi \times 0.375^2 = 184.9$ in. ² This compares with the exact value of 175.4 in. ² calculated in Table B1-1	

Table B1-1—Projected areas (A_{cp}) for varying L_d and S

Development		Spacing S			
inches	Radius <i>R,</i> inches	4 in.	5 in.	6 in.	7 in.
3.71	4.09 in. ²	129.5 in. ²	152.4 in. ²	175.4 in. ²	195.2 in. ²
5.71	6.09 in. ²	226.0 in. ²	258.0 in. ²	290.8 in. ²	324.4 in. ²

Example B1, continued

CODE SECTION	DESIGN PROCEDURE	CALCULATION
	STEP 4: Calculate required plate thickness	
B.3.1 B.6.2	Try an 8 in. \times 8 in. plate The plate must transmit to the studs all loads used in the design of the attachment. The design strength for embedments shall be based on a maximum steel stress of ϕf_y .	$ \begin{array}{c} b\\ +\\ +\\ +\\ +\\ +\\ +\\ +\\ b\\ +$
	Calculate the bending strength of the plate based on the yield moment capacity using yield stress. Evaluate plate sections to determine minimum load capacity.	At face of tube (a-a): $M = 9 \times 1.5 = 13.5 \text{ inkips}$ $0.9 \times M_y = \frac{1}{6} \times 8 \times t^2 \times 0.9 \times 36$ $= 43.2 t^2$ $t_{min} = \sqrt{(13.5/43.2)} = 0.56 \text{ in.}$ On diagonal (b-b): $M = 4.5 \times 1.5 \sqrt{2} = 9.5 \text{ inkips}$ $M_y = \frac{1}{6} \times 5 \sqrt{2} \times t^2 \times 0.9 \times 36$ $= 38.2 t^2$ $t_{min} = \sqrt{(9.5/38.2)} = 0.50 \text{ in.}$ Use 8 in. $\times 8 \text{ in.} \times \frac{5}{8}$ in. embedded plate

Example B2(a)—Four-stud rigid embedded plate, combined shear and uniaxial moment

Design an embedment using welded studs and a rigid embedded plate for a $3 \times 3 \times 1/_4$ in. A 501 structural tube attachment.

Given:

Given:
$$f'_c = 4000 \text{ psi}$$
 $f_y = 50,000 \text{ psi}$ (studs) $f_{ut} = 60,000 \text{ psi}$ $F_y = 36,000 \text{ psi}$ (plate) $M_u = 70 \text{ in.-kips}$ $V_u = 12.4 \text{ kips}$ $W_u = 12.4 \text{ kips}$ $W_u = 70 \text{ in.-kips}$ $V_u = 12.4 \text{ kips}$ $W_u = 12.4 \text{ kips}$ $V_u = 12.4 \text{ kips}$ $M_u = 70 \text{ in.-kips}$ $V_u = 12.4 \text{ kips}$ $M_u = 70 \text{ in.-kips}$ $V_u = 12.4 \text{ kips}$ $M_u = 70 \text{ in.-kips}$ $V_u = 12.4 \text{ kips}$ $M_u = 70 \text{ in.-kips}$ $V_u = 12.4 \text{ kips}$ $M_u = 70 \text{ in.-kips}$ $V_u = 12.4 \text{ kips}$ $M_u = 70 \text{ in.-kips}$ $V_u = 12.4 \text{ kips}$ $M_u = 70 \text{ in.-kips}$ $V_u = 12.4 \text{ kips}$ $M_u = 70 \text{ in.-kips}$ $V_u = 12.4 \text{ kips}$ $M_u = 70 \text{ in.-kips}$ $3 \times 3 \times 1/4^m$ TubeSECTIONDESIGN PROCEDURECALCULATIONSTEP 1: Design for moment

	STEP 1: Design for moment	
	Try a 7 in. \times 7 in. plate with 5 in. \times 5 in. stud spacing	$T = A_e f_y$
10.2.7	Assume a uniform stress block for concrete compression zone and the two top studs as the tension components.	$ \begin{array}{c} 6'' \\ \hline \\ C \\ 0.85 f'_{c} \end{array} M_{u} = 70 $
9.2	Equate the internal forces and solve for <i>a</i> :	$0.85f_c'ab = A_e f_y$
9.3.1		$a = A_e f_y / (0.85 f_c' b)$ $a = A_e 50 / [0.85 (4)(7)]$ $a = 2.10 A_e$
	Equate the external (required strength) and internal moment (design strength) and solve the resulting quadratic equation for A_e :	$M_u = \phi M_n$ 70.0 = 0.9 A _e (50)(6 - 2.10 A _e /2) A _e = 0.275 in. ²
	For 2 studs in tension, A_e per stud is:	A_e /stud = 0.138 in. ²
	Try ¹ / ₂ in. diameter studs	A_e /stud = 0.196 in. ²
	STEP 2: Design for shear	
B.6.5.2	Since this is an embedded base plate, Section B.6.5.2.2 is applicable.	
	The stud area not used for moment is available for shear transfer by shear friction.	$A_{sv} = 2(0.196 - 0.138) + 2(0.196)$ $A_{sv} = 0.508 \text{ in.}^2$
B.6.5.2.2	Shear-friction coefficient	0.90
	Nominal shear strength	$V_n = 0.90A_{Vf}f_y$ $V_n = 0.90 (0.508)(50)$ $V_n = 22.9$ kips
B.6.2.2	Capacity reduction factor for shear	φ = 0.85
9.2 9.3	Design shear strength must be greater than the required strength	$\phi V_n = 0.85 (22.9 \text{ kips})$ $\phi V_n = 19.5 > 12.4 \text{ kips}$ OK

Example B2(a), continued

CODE SECTION	DESIGN PROCEDURE	CALCULATION	
STEP 3: Design for rigid base plate			
	In order to ensure rigid base plate behavior, it is essential that the base plate not yield on either the compression or tension side of the connection.	$T = A_e f_y$ $f = A_e f_y$ $f = \int dt$ No Yield $f = \int dc$ $f = \frac{1}{c} \frac{a/2}{t}$	
	Determine minimum base plate thickness to prevent base plate yielding. The moment in the plate at the edge of the attached member is used for sizing the base plate thickness. The larger of the moment on the tension side or the compression side will control the design of the base plate.	$M = Td_t$ or $M = Cd_c$ whichever is greater	
	Moment on tension side	$M = Td_t$ $M = A_e f_y d_t$ M = (0.275)(50)(1.0) M = 13.75 inkips	
	Moment on compression side Note that $C = T$. NOTE: For this example, it is only necessary to show that d_c is greater than d_t , and then calculate the moment as $M = (d_c)(C)$. With multiple rows of anchors in tension (e.g., a middle row of anchors), both the moment on the tension side and the moment on the compression side need to be determined. This general procedure is shown in this example.	$M = Cd_c$ $M = A_e f_y d_c$ M = (0.275)(50)(6 - 4 - a/2) M = 0.275 (50)[2 - 2.1(0.275)/2] $M = 23.5 \text{ inkips} \leftarrow \text{ controls}$	
	Nominal moment capacity of base plate	$M_n = F_y S$ $M_n = F_y (bt^{2/6}) = (36)(7)t^{2/6}$ $M_n = 42t^2$	
	Determine minimum base plate thickness to prevent yielding of plate. Note that the plate thickness is calculated using the nominal strength of the anchors. The ϕ factor is not included since the calculation of plate thickness should be based on the maximum nominal tensile force in the anchor rather than the design force. A ϕ factor of 0.9 is used in calculating the required area of the anchor in Step 1	$42t^2 = 23.5$ t = 0.75 in. use $3/4$ in. plate	

Example B2(a), continued

CODE SECTION	DESIGN PROCEDURE	CALCULATION
	STEP 4: Embedment length	
B.5.1.1	Calculate design load assuming all studs may resist concurrent tensile loads. This assumption of all four studs in tension assures ductility even in the event of a pure tension load. If the designer can assure that such a tensile condition cannot occur, it is sufficient only to develop two of the studs at a time since the other two studs are in compression.	$P_{ut} = A_s f_{ut}$ = 4(0.196)(60) = 47.0 kips
	Check bearing requirements of stud head	See Example A1
B.4.2	Calculate capacity of concrete	$P_{d} = \phi 4 \sqrt{(f_{c}')} A_{cp}$ where: $\phi = 0.65$ $A_{cp} = \text{projected area of concrete}$ R = radius of projected cones Try 1/ ₂ in. diameter stud 6-1/ ₈ in. long having an effective length, $L_{d} = 5.69$ in. $R = L_{d}$ stud + plate thickness + stud head radius = 5.69 + 0.75 + 0.5 = 6.94 in.
	Assume the outer boundaries of the stress cones are connected by tangents. NOTE: Although slightly unconservative, the assumption above is reasonable for embedments where the embedment radius (<i>R</i>) exceeds the spacing between individual anchors. For most embedments, particularly those in the "rigid" plate category, the embedment radius will usually exceed the anchor spacing.	$A_{cp} = 5(5) + 4(5)(6.94) + (6.94)^2\pi - 4(0.79) = 312.0 \text{ in.}^2$ Therefore, $P_d = 4(0.65)^* \sqrt{(4000)} (312.0) = 51,300 \text{ lb}$ $P_d = 51.3 \text{ kips} > 47.0 \text{ kips}$

7″

0

.

5″

Example B2(b)—Four-stud flexible embedded plate, combined shear and uniaxial moment

Design an embedment using welded studs and a flexible embedded plate for a $3 \times 3 \times \frac{1}{4}$ in. A 501 structural tube attachment.

Given:



CODE SECTION	DESIGN PROCEDURE	CALCULATION
	STEP 1: Design for moment	
	Try a 7 in. \times 7 in. plate with 5 in. \times 5 in. stud spacing	T - A f
10.2.7	Determine the amount of tensile steel required for the applied moment.	
	If the base plate is not stiff enough to obtain rigid base plate behavior, a hinge will form on the com- pression side of the base plate at the edge of the attached member. This will cause the compres- sive resultant to move inward toward the attached member.	$\begin{array}{c} \mathbf{C} \\ \mathbf{C} \\ \mathbf{C} \\ \mathbf{C} \\ \mathbf{M}_{u} = 70 \end{array}$
	Physically, the nearest the compressive reaction can be to the edge of the attached member is a distance " <i>c</i> " equal to the yield moment of the plate divided by the compressive reaction.	
	<i>Reference</i> : Cook, R. A., and Klingner, R. E., "Ductile Multiple-Anchor Steel-to-Concrete Connections," <i>Journal of Structural Engineering</i> , American Society of Civil Engineers, V. 118, No. 6, June, 1992, pp. 1645-1665.	
	NOTE: For simplicity, it may also be assumed that the compressive reaction is located at the edge of the attached member.	
	Assume a $\frac{5}{8}$ in. thick base plate and determine yield capacity M_n of plate.	$M_n = F_y S$ $M_n = (36)(7)(0.625)^2/6$ $M_n = 16.4$ inkips
	Determine <i>c</i> . NOTE: From summation of forces $T = C = A_e f_y$	$c = M_n/C$ $c = F_y S/A_e f_y$ $c = 16.4/A_e (50)$ $c = 0.328/A_e$

Example B2(b), continued

CODE SECTION	DESIGN PROCEDURE	CALCULATION	
STEP 1: continued			
9.2 9.3.1	Equate the external (required strength) and internal (design strength) moment and solve the resulting linear equation for A_e :	$M_{u} = \phi M_{n}$ $70.0 = 0.9 A_{e}(50)(4 + c)$ $70.0 = 0.9 A_{e}(50)(4 + 0.328/A_{e})$ $70.0 = 45 (4A_{e} + 0.328)$ $A_{e} = 0.307 \text{ in.}^{2}$	
	For 2 studs in tension, A_e per stud is:	A_e /stud = 0.153 in. ²	
	Try $1/_2$ in. diameter studs	A _e /stud - 0.196 in. ²	
	STEP 2: Design for shear		
B.6.5.2	Since this is an embedded base plate, Section B.6.5.2.2 is applicable.		
B.6.5.3	The stud area not used for moment is available for shear transfer by shear friction.	$A_{sv} = 2(0.196 - 0.153) + 2(0.196)$ $A_{sv} = 0.478 \text{ in.}^2$	
B.6.5.2.2	Shear-friction coefficient	0.90	
	Nominal shear strength	$V_n = 0.90 A_{sv} f_y$ $V_n = 0.90 (0.478) (50)$ $V_n = 21.5$ kips	
B.6.2.2	Capacity reduction factor for shear	φ = 0.85	
9.2 9.3	Design shear strength must be greater than the required strength	$\phi V_n = 0.85(21.5 \text{ kips})$ $\phi V_n = 18.3 \text{ kips} > 12.4 \text{ kips}$ OK	
	STEP 3: Design for flexible base plate		
	In order to ensure that prying action does not occur on the tension side of the base plate, it is essential that the base plate not yield on the tension side of the connection.	$T = A_e f_y$ dt No Yield Yield OK C t	
	Determine minimum base plate thickness to prevent base plate yielding and possible prying action on tension side.	$M = Td_t$ $M = A_e f_y d_t$ M = (0.307)(50)(1.0) M = 15.35 inkips	
	Nominal moment capacity of base plate. Note that a ϕ = 0.90 is already included.	$ \begin{split} M_n &= F_y S \\ M_n &= F_y (bt^{2}/6) \\ M_n &= (36)(7)(0.625)^{2}/6 \\ M_n &= 16.4 \text{ inkips} > 15.35 \text{ inkips} \\ & \frac{5}{8} \text{ in. plate is OK} \end{split} $	

Example B2(b), continued

CODE SECTION	DESIGN PROCEDURE	CALCULATION
	STEP 4: Embedment length	
	See Example B2(a)	

NOTE:

As can be seen from this flexible base plate example ($\frac{5}{8}$ in. base plate) and Example B2(a) with a rigid base plate ($\frac{3}{4}$ in. base plate), there is very little difference between the two analyses for a typical two row connection. The real advantages of flexible base plate analysis become apparent with multiple rows of anchors.

In the case of multiple rows of anchors, the compressive reaction becomes so large that the assumption of rigid base plate behavior results in excessively thick plates. For multiple row connections, the flexible base plate procedure will result in more reasonable base plate thicknesses.

Example B2(c)—Four-bolt rigid surface-mounted plate, combined shear and uniaxial moment

Design an embedment using cast-in-place bolts and a rigid surface-mounted plate for a $3 \times 3 \times 1/4$ in. A 501 structural tube attachment.

Given:

- f'c f_y f_{ut} F_y M_u = 4000 psi =
- 105,000 psi (bolts) 125,000 psi (bolts) =
- 36,000 psi (plate) =
- = 70 in.-kips

$$V_u = 12.4$$
 kips

where M_u and V_u are the required factored external loads as defined in Section 9.2 of the Code.



CODE SECTION	DESIGN PROCEDURE	CALCULATION
	STEP 1: Design for moment	
10.2.7	Try a 7 in. \times 7 in. plate with 5 in. \times 5 in. bolt spacing	$T = A_e f_y$
	Determine the amount of tensile steel required for the applied moment. Assume standard concrete beam compression block.	$ \begin{array}{c} $
	Equate the internal forces and solve for <i>a</i> :	$0.85 f_c' ab = A_e f_y$ $a = A_e f_y / (0.85 f_c' b)$ $a = A_e 105 / [0.85 (4)(7)]$ $a = 4.41 A_e$
9.2 9.3.1	Equate the external (required strength) and internal (design strength) moments and solve the resulting quadratic equation for A_e :	$M_u = \phi M_n$ 70.0 = 0.9 A _e (105)(6 - 4.41A _e /2) A _e = 0.130 in. ²
	For 2 bolts in tension, A_e per bolt is:	A_e / bolt = 0.065 in. ²
	Try $\frac{3}{8}$ in. diameter bolts (for $\frac{3}{8}$ in. threaded bolts $A_e = 0.078$ in. ²)	A_e / bolt = 0.078 in. ²
	STEP 2: Design for shear	
B.6.5.2	Since this is a surface mounted plate, Section B.6.5.2.1 is applicable.	
B.6.5.2.1	The nominal shear strength is the sum of the shear strength provided by the anchors and the friction force between the base plate and concrete due to the compressive reaction, taken as 0.4 in this example.	$V_n = 0.70 A_{vs} f_y + 0.40 C$

Example B2(c), continued

CODE SECTION	DESIGN PROCEDURE	CALCULATION		
STEP 2: continued				
B.6.5.3	The anchor area not used for moment is available for shear transfer. Assume threads in shear plane.	$A_{vs} = 2(0.078 - 0.065) + 2(0.078)$ $A_{vs} = 0.182 \text{ in.}^2$		
	Shear contribution from friction between the base plate and concrete due to the compressive reaction.	0.40 C = 0.40 T = 0.40($A_e f_y$) = 0.40(0.130)(105) 0.40 C = 5.46 kips		
	Nominal shear strength from anchors and friction between the base plate and concrete	$V_n = 0.70 A_{vs} f_y + 0.40 C$ $V_n = 0.70(0.182)(105) + (5.46)$ $V_n = 18.8$ kips		
B.6.2.2	Capacity reduction factor for shear	φ = 0.85		
9.2 9.3	Design shear strength must be greater than the required strength.	$\phi V_n = 0.85(18.8 \text{ kips})$ $\phi V_n = 16.0 \text{ kips} > 12.4 \text{ kips}$ OK		
	STEP 3: Design for rigid base plate			
	In order to ensure rigid base plate behavior, it is essential that the base plate not yield on either the compression or tension side of the connection. NOTE: This step in the example is for information only. Actual design of the base plate is not covered by ACI 349 Appendix B. Although the design procedure shown is appropriate for base plate design, the actual design values used should be based on the appropriate structural steel code. Determine minimum base plate thickness to prevent base plate yielding. The moment in the plate at the edge of the attached member is used for sizing the base plate thickness. The larger of the moment on the tension side or the compression side will control the design of the base plate. Moment on tension side	$T = A_e f_y$ No Yield dc dc dc dc dc dc dc dc		
	Moment on compression side: Note that $C = T$. NOTE: For this example, it is only necessary to show that d_c is greater than d_t , and then calculate the moment as $M = (d_c)(C)$. With multiple rows of anchors in tension (e.g., a middle row of anchors), both the moment on the tension side and the moment on the compression side need to be determined. This general procedure is shown in this example.	$M = Cd_c$ $M = A_e f_y d_c$ M = (0.130)(105)(6 - 4 - a/2) M = 0.130(105)[2 - 4.41(0.130)/2] $M = 23.4 \text{ inkips} \leftarrow \text{controls}$		

Example B2(c), continued

CODE SECTION	DESIGN PROCEDURE	CALCULATION
	STEP 3: continued	
	Nominal moment capacity of base plate	$M_n = F_y S$ $M_n = F_y (bt^{2}/6) = (36)(7)t^{2}/6$ $M_n = 42t^2$
	Determine minimum base plate thickness to prevent yielding of plate. Note that the plate thickness is calculated using the nominal strength of the anchors. It is not necessary to include a ϕ factor in this calculation of plate thickness. A ϕ factor of 0.9 is used in calculating the required area of the anchor in Step 1.	$42t^2 = 23.4$ t = 0.74 in. use $3/4$ in. plate
	STEP 4: Embedment length	
	The calculation of the required embedment length is similar to that in Example B2(a).	

7″

0

Ο

5″

0

Example B3(a)—Four-stud rigid embedded plate, combined tension, shear, and uniaxial moment

Design an embedment using welded studs and a rigid embedded plate for a $3 \times 3 \times 1/4$ in. A 501 structural tube attachment.

Given:

 $f'_{c} = 4000 \text{ psi}$ $f_{y} = 50,000 \text{ psi} (\text{studs})$ $f_{ut} = 60,000 \text{ psi}$ $F_{y} = 36,000 \text{ psi} (\text{plate})$ $M_{u} = 70 \text{ in.-kips}$ $V_{u} = 12.4 \text{ kips}$ $P_{u} = 11.1 \text{ kips}$ where M_{u} , P_{u} , and V_{u} are the required factored external loads as defined in Section 9.2 of the Code. $3 \times 3 \times \frac{1}{4}$ Tube

CODE SECTION	DESIGN PROCEDURE	CALCULATION		
STEP 1: Design for moment and tension				
	Try a 7 in. \times 7 in. plate with 5 in. \times 5 in. stud spacing	$T = A_e f_y$		
10.2.7	Assume a uniform stress block for concrete compression zone and the top two studs as the tension components.	$ \begin{array}{c} & & \\ & & $		
	See Example B2(a), the attachment plate is assumed rigid.	$0.85f_c' \longrightarrow 0$		
9.2 9.3.1	Sum external (required strength) and internal (design strength) forces.	$50A_e - 0.85(4)(a)(7) = 11.1$ $a = 2.1A_e - 0.466$		
	Sum moments about the center line of base plate (line of axial load). $\phi = 0.9$ for flexure	$\begin{aligned} 70.0 &= 0.9[50A_e(2.5) \\ &+ 0.85f_c' ab(d-2.5-a/2)] \\ 77.78 &= 50A_e(2.5) \\ &+ 0.85(4)(2.1A_e-0.466) \\ &\times (7)[6-2.5-0.5(2.1A_e-0.466)] \end{aligned}$		
	Solving the quadratic equation:	$A_e = 0.46 \text{ in.}^2$		
	Steel / Stud	$0.46/2 = 0.23 \text{ in.}^2$		
	Provide ⁵ / ₈ in. diameter studs	A _e = 0.30 in. ²		
STEP 2: Design for shear				
	Assume the total stud area not used for moment and tension is available for shear transfer by shear friction.	$A_{sv} = 2(0.30 - 0.23) + 2(0.30)$ = 0.74 in. ²		
	Nominal shear capacity $\mu = 0.9$	$V_n = 50(0.74)(0.9)$ = 33.3 kips		
9.2 9.3	Shear capacity $\phi = 0.85$	φ <i>V_n</i> = 0.85(33.3) = 28.3 kips > 12.4 kips, OK		
STEP 3 and STEP 4				
	See Example B2(a)			

Example B3(b)—Four-stud flexible embedded plate, combined tension, shear, and uniaxial moment

Design an embedment using welded studs and a flexible embedded plate for a $3 \times 3 \times 1/4$ in. A 501 structural tube attachment.

Given:

 $f'_{c} = 4000 \text{ psi}$ $f_{y} = 50,000 \text{ psi} (\text{studs})$ $f_{ut} = 60,000 \text{ psi}$ $F_{y} = 36,000 \text{ psi} (\text{plate})$ $M_{u} = 70 \text{ in.-kips}$ $V_{u} = 12.4 \text{ kips}$ $P_{u} = 11.1 \text{ kips}$ where M_{u}, P_{u} , and V_{u} are the required factored external loads as defined in Section 9.2 of the Code.



CODE SECTION	DESIGN PROCEDURE	CALCULATION	
STEP 1: Tension in top studs			
	Try a 7 in. \times 7 in. plate with 5 in. \times 5 in. stud spacing	$ \begin{array}{c} T = A_{\theta}f_{y} \\ \hline P_{u} = 11.1 \\ \hline P_{u} = 70 \\ \hline M_{u} = 70 \end{array} $	
	For simplicity, it is assumed that the compressive reaction is located at the edge of the attached member. Lever arm for moment = 4 in.; ϕ = 0.9	$\phi T = 70/4 + 11.1(1.5)/4$ $\phi T = 21.7$ kips T = 24.0 kips	
	Tension in each stud	24.0/2 = 12.0 kips	
	A_e , area for each stud	A _e = 12.0/50 = 0.24 in. ²	
	Try ⁵ / ₈ in. diameter stud	A_e provided = 0.30 in. ²	
STEP 2: Design for shear			
B.6.5.3	The stud area not used for moment and tension is available for shear transfer by shear friction.	$A_{vf} = 2(0.30 - 0.24) + 2(0.30)$ = 0.72 in. ²	
B.6.5.2.2	Nominal shear strength $\mu = 0.9$	$V_n = 0.9(0.72)(50)$ = 32.40 kips	
B.6.2.2	Capacity reduction factor for shear	φ = 0.85	
9.2 9.3	Design shear strength must be equal to or greater than the required strength.	$\phi V_n = 0.85(32.4)$ = 27.5 kips > 12.4 kips, OK	
STEP 3 and STEP 4			
	See Example B2(b)		

Example B4—Four-stud rigid embedded plate in thin slab, tension only

Determine the reduction of projected stress area due to limited concrete thickness for the embedment of Example B1.

Given:

$$f'_c = 4000 \text{ psi}$$

 $f_y = 50,000 \text{ psi} (\text{studs})$
 $f_{ut} = 60,000 \text{ psi}$

Thickness of concrete slab = 6 in.



CODE SECTION	DESIGN PROCEDURE	CALCULATION	
STEP 1: Determine area available for stress reduction			
B.4.2	Total stress reduction area = $(b + 2L_d + 2t - 2h)^2$	 = [6.75 + 2(3.71) + 2(0.625) - 2(6.5)]² = 5.86 in.² < 14.8 in.² area still available for stress reduction, OK Ductility requirements met. 	

COMMENT:

Example B1 describes an embedment assembly with a projected area of 175.4 sq. in. when there was a minimum required area of 160.6 in.². this leaves approximately 14.8 sq. in. available for reduction in projected area before the minimum requirements for concrete strength are no longer met.

Due to biaxial symmetry of the assembly in Example B1, all sides of the rectangular stress reduction area are equal and, therefore, only one side needs to be found.

Note that the projected area calculated in Example B1, conservatively neglects the thickness of the plate and uses $L_d = 3.71$ in. rather than the length of 4.33 in. to the face of concrete.

APPENDIX A—Projected area (A_{cp}) for four studs

This appendix develops the projected area of four stress cones at the surface of the concrete. The studs are located at the corners of a rectangle with spacing S_x and S_y in each direction. The radius of the projected stress cones (45 deg cone angle) is $R = L_d + d_h/2$.

For two overlapping stress cones of radius R and spacing S, the angle α of the common segment is given by:

 $\cos(1/2\alpha) = S/(2R)$ $\alpha = 2\cos^{-1}[S/(2R)]$

The area of the common segment A_{seg} equals the area of the two sectors minus the area of the triangles:

$$A_{seg} = 2 \left[\alpha R^2 / 2 - R^2 \sin(\alpha/2) \cos(\alpha/2) \right]$$
$$= (\alpha - \sin \alpha) R^2$$

CASE I

The projected area is equal to the area of four full cones minus the area of the four overlapping portions minus the area of the four heads.

$$A_{cp1} = 4\pi R^2 - 2(\alpha_x - \sin\alpha_x)R^2 - 2(\alpha_y - \sin\alpha_y)R^2 - 4A_h$$

CASE II

minus the area of the four heads.

The projected area is equal to the area of the central rectangle plus the area of the four three quarter cones minus the area of the four overlapping portions outside the rectangle There are two cases of overlapping stress cones. In Case I, there is no overlap at the center of the rectangle since $R < 1/2 (S_x^2 + S_y^2)^{1/2}$. In Case II, all four stress cones overlap. The projected area for the two cases is formulated below.





CASEI



SUMMARY

$$A_{cp1} = (4\pi - 2B)R^2 - 4A_h$$
$$A_{cp2} = (3\pi - B)R^2 + S_x S_y - 4A_h$$

where

$$B = (\alpha_x - \sin \alpha_x) + (\alpha_y - \sin \alpha_y)$$
$$\alpha_x = 2\cos^{-1}(S_x/2R)$$
$$\alpha_y = 2\cos^{-1}(S_y/2R)$$



