

Guide to the Concrete Capacity Design (CCD) Method— Embedment Design Examples

Reported by ACI Committee 349



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American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
U.S.A.

Phone: 248-848-3700
Fax: 248-848-3701

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Guide to the Concrete Capacity Design (CCD) Method—Embedment Design Examples

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Ronald J. Janowiak
Chair

Omesh B. Abhat	Werner Fuchs	Christopher Heinz*	Richard S. Orr
Adeola K. Adediran*	Branko Galunic*	Charles J. Hookham	Bozidar Stojadinovic
Hansraj G. Ashar	Partha S. Ghosal	Jagadish R. Joshi	Barendra K. Talukdar
Ranjit L. Bandyopadhyay*	Herman L. Graves III*	Richard E. Klingner	Donald T. Ward
Peter J. Carrato	Orhan Gurbuz	Nam-Ho Lee	Andrew S. Whittaker
Ronald A. Cook	James A. Hammell*	Dan J. Naus	Albert Y. C. Wong
Rolf Eligehausen	Gunnar A. Harstead	Dragos A. Nuta	Charles A. Zalesiak

*Committee 349 members who were major contributors to the development of this report.

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INTRODUCTION

This report was prepared by the members of the ACI 349 Subcommittee on Steel Embedments to provide examples of the application of ACI 349 to the design of steel embedments. The first edition of this report, published in 1997, was based on ACI 349-97 that used the 45-degree cone breakout model for determining the concrete breakout strength. The 2001 edition of the Code* marked a major departure from the previous editions with the adoption of the concrete capacity design (CCD) method. The model for the concrete breakout strength used in the CCD method is a breakout prism having an angle of approximately 35 degrees. In addition, the concrete breakout strength for a single anchor away from the edge is proportional to the embedment depth raised to the power of 1.5 and not embedment depth squared, as used in the previous versions of the Code. These and other changes in the Code result in designs that are somewhat different than those obtained using previous editions of the Code. The examples used in this report are based on the ACI 349-06, Appendix D, and illustrate how the CCD method is applied. In previous editions of ACI 349, the anchorage design was given in Appendix B. Because ACI 349 is a dependent code, the chapters and Appendixes in ACI 349 are updated to be consistent with ACI 318.

As in previous Codes, 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 breakout, concrete pullout, side-face blowout, and pryout strength should be greater than the tensile or shear strength of the steel portion of the embedment.

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

NOTATION

A_{brg}	=	bearing area of the head of stud or anchor bolt, in. ²
$A_{brg,pl}$	=	the effective bearing area of a steel base plate, in. ²
A_D	=	gross cross-sectional area of anchor, in. ²
A_H	=	gross cross-sectional area of anchor head, in. ²
A_{Nc}	=	projected concrete failure area of a single anchor or group of anchors, for calculation of strength in tension (A_{Nc} shall not be taken greater than nA_{Nco}), in. ² , see D.5.2.1
A_{Nco}	=	projected concrete failure area of a single anchor, for calculation of strength in tension if not limited by edge distance or spacing, in. ² , see D.5.2.1

A_{se}	=	effective cross-sectional area of anchor, in. ²
$A_{se,t}$	=	effective cross-sectional area of anchor (required to resist tension loads), in. ²
$A_{se,v}$	=	effective cross-sectional area of anchor (required to resist shear loads), in. ²
A_{Vc}	=	projected concrete failure area of a single anchor or group of anchors, for calculation of strength in shear (A_{Vc} shall not be taken greater than nA_{Vco}), in. ² , see D.6.2.1
A_{Vco}	=	projected concrete failure area of a single anchor, for calculation of strength in shear, if not limited by corner influences, spacing, or member thickness, in. ² , see D.6.2.1
a	=	moment arm from row of anchors to mid-thickness of adjacent steel tube wall, in.
b	=	width of steel base plate, in.
b_{eff}	=	effective width of steel base plate, in.
b_f	=	flange width of supported steel member, in.
C	=	anchor head dimension, see figure in Tables 4(a) through (c) , in.
C_F	=	the compressive resultant force between the embedment and the concrete resulting from factored moment and factored axial load applied to the embedment, lb
C_m	=	the resultant compressive force in concrete due to factored moment acting on a steel base plate, kips
c_{a1}	=	distance from the center of an anchor shaft to the edge of concrete in one direction. If shear is applied to anchor, c_{a1} is taken in the direction of the applied shear. If the tension is applied to the anchor, c_{a1} is the minimum edge distance, in.
c_{a2}	=	distance from center of an anchor shaft to the edge of concrete in the direction orthogonal to c_{a1} , in.
c_{ac}	=	critical edge distance required to develop the basic concrete breakout strength of a post-installed anchor in uncracked concrete without supplementary reinforcement to control splitting, in., see D.8.6
$c_{a,max}$	=	maximum distance from center of an anchor shaft to the edge of concrete, in.
$c_{a,min}$	=	minimum distance from center of an anchor shaft to the edge of concrete, in.
d	=	moment arm from resultant compression force on base plate to center of tension force in anchors, in.
d_c	=	distance from resultant compression force to adjacent edge of supported steel member, in.
d_e	=	distance from edge of steel base plate to the resultant compression force, in.
d_h	=	nominal diameter of anchor head, in.
d_o	=	outside diameter of anchor or shaft diameter of headed stud or headed bolt, in.
d_s	=	depth of supported steel member, in.

*Note: Wherever the term "Code" is used, it signifies ACI 349.

d_t	= distance from center of tension force in anchors and adjacent edge of supported steel member, in.	n	= number of anchors in a group
e'_N	= distance between resultant tension load on a group of anchors loaded in tension and the centroid of the group of anchors loaded in tension (e'_N is always positive), in.	OD	= outside diameter of steel washer, in.
e'_V	= distance between resultant shear load on a group of anchors loaded in shear in the same direction and the centroid of the group of anchors loaded in shear in the same direction (e'_V is always positive), in.	P_u	= factored axial force; to be taken as positive for compression and negative for tension, lb
F	= anchor head dimension, see figure in Tables 4(a) through (c), in.	s	= center-to-center spacing of items, such as anchors, in.
F_d	= ductility factor, 0.85, per D.3.6.1	T	= factored tensile force in a single anchor or a row of anchors, kips
F_y	= specified minimum yield strength of steel plate, ksi	t	= thickness of washer or plate, in.
f'_c	= specified compressive strength of concrete, psi	t_f	= flange thickness of supported steel member, in.
f_{uta}	= specified tensile strength of anchor steel, psi	t_w	= web thickness of supported steel member, in.
f_{ya}	= specified yield strength of anchor steel, psi	V_b	= basic concrete breakout strength in shear of a single anchor in cracked concrete, lb, see D.6.2.2 or D.6.2.3
H	= anchor head thickness, see figure in Tables 4(a) through (c), in.	V_{cb}	= nominal concrete breakout strength in shear of a single anchor, lb, see D.6.2.1
h_a	= thickness of member in which an anchor is located, measured parallel to anchor axis, in.	V_{cbg}	= nominal concrete breakout strength in shear of a group of anchors, lb, see D.6.2.1
h_{ef}	= effective embedment depth of anchor, in., see D.8.5	V_{cp}	= nominal concrete pryout strength of a single anchor, lb, see D.6.3
ID	= inside diameter of steel washer, in.	V_{cpg}	= nominal concrete pryout strength of a group of anchors, lb, see D.6.3
k_c	= coefficient for basic concrete breakout strength in tension	V_f	= shear resisting force provided by friction resulting from compressive forces on steel base plate, kips
k_{cp}	= coefficient for pryout strength	V_n	= nominal shear strength, lb
L	= overall length of anchor, in.	V_{sa}	= nominal strength in shear of a single anchor or group of anchors as governed by the steel strength, lb, see D.6.1.1 or D.6.1.2
ℓ_e	= load bearing length of anchor for shear, not to exceed $8d_o$, in., see D.6.2.2	V_u	= factored shear force at section, kips
M_n	= nominal flexural strength at section, kip-in.	V_{ua}	= factored shear force applied to a single anchor or group of anchors, lb
M_p	= plastic moment of steel plate, kip-in.	w	= moment arm from corner anchor to midthickness of adjacent steel tube wall, in.
M_u	= factored moment at section, kip-in.	Z	= plastic section modulus of steel base plate, in. ³
M_y	= moment corresponding to onset of yielding at extreme fiber of steel plate, kip-in.	ϕ	= strength reduction factor, see D.4.4 and D.4.5
N_b	= basic concrete breakout strength in tension of a single anchor in cracked concrete, lb, see D.5.2.2	$\Psi_{c,N}$	= factor used to modify tensile strength of anchors based on presence or absence of cracks in concrete, see D.5.2.6
N_{cb}	= nominal concrete breakout strength in tension of a single anchor, lb, see D.5.2.1	$\Psi_{c,P}$	= factor used to modify pullout strength of anchors based on presence or absence of cracks in concrete, see D.5.3.5
N_{cbg}	= nominal concrete breakout strength in tension of a group of anchors, lb, see D.5.2.1	$\Psi_{c,V}$	= factor used to modify shear strength of anchors based on presence or absence of cracks in concrete and presence or absence of supplementary reinforcement, see D.6.2.7
N_n	= nominal strength in tension, lb	$\Psi_{cp,N}$	= factor used to modify tensile strength of post-installed anchors intended for use in uncracked concrete without supplementary reinforcement, see D.5.2.7
N_p	= pullout strength in tension of a single anchor in cracked concrete, lb, see D.5.3.4	$\Psi_{ec,N}$	= factor used to modify tensile strength of anchors based on eccentricity of applied loads, see D.5.2.4
N_{pn}	= nominal pullout strength in tension of a single anchor, lb, see D.5.3.1	$\Psi_{ec,V}$	= factor used to modify shear strength of anchors based on eccentricity of applied loads, see D.6.2.5
N_{sa}	= nominal strength of a single anchor or group of anchors in tension as governed by the steel strength, lb, see D.5.1.1 or D.5.1.2		
N_{sb}	= side-face blowout strength of a single anchor, lb		
N_{sbg}	= side-face blowout strength of a group of anchors, lb		
N_{ua}	= factored tensile force applied to anchor or group of anchors, lb		

$\Psi_{ed,N}$ = factor used to modify tensile strength of anchors based on proximity to edges of concrete member, see [D.5.2.5](#)

$\Psi_{ed,V}$ = factor used to modify shear strength of anchors based on proximity to edges of concrete member, see [D.6.2.6](#)

Note: When used in the design examples that follow, kips is used instead of lb (1 kip = 1000 lb). Also, kip-inch or kip-in. is used interchangeably with in.-kip.

COMMENTARY

ACI 349-06 specifies acceptance criteria for tension and shear loads on individual anchors and on groups of anchors. It specifies that the loads be determined by elastic analysis. Plastic analysis is permitted provided that deformational compatibility is taken into account, equilibrium is satisfied on the deformed geometry (taking into account the change in stiffness due to yielding), deformation does not lead to structural instability, and the nominal strength of the anchor is controlled by ductile steel elements. This document does not provide detailed methods of analyses as to how to calculate the loads on anchors, but does specify design rules when the internal tension or shear loads are eccentric.

The evaluation of loads in each anchor and the effects on the group strength is well defined in the design examples for single anchors ([Examples A1 to A4](#)) and four anchors under tension ([Examples B1 and B4](#)).

[Examples B2 and B3](#) have four anchors under applied shears and moments. The embedment depth is selected such that the anchor strength under tension loads is controlled by ductile yielding of the steel.

When designing the base plates in each problem, no distinction between the AISC load factors (and ϕ -factors) and the ACI load factors (and ϕ -factors) is made. The Engineer should reconcile the differences between these two codes when designing the base plate.

When the Engineer is faced with base plate and anchorage configuration differing from those used in these design examples, the Engineer must apply the Code requirements and use rational assumptions appropriate for these other design configurations.

Strength reduction factor ϕ for frictional resistance is not explicitly defined in the Code. As frictional resistance is not related to a steel mode of failure, the examples have used the ϕ -factor from [D.4.4c](#) or [D.4.5c](#) (depending on whether 9.2 or C.2 of the Code is used, respectively).

PART A—Examples: Ductile single embedded element in semi-infinite concrete

Example A1—Single stud, tension only, no edge effects

Design an embedment using a stud welded to an embedded plate. The stud is located sufficiently away from the edges on the concrete so that there are no edge effects.

Given:

Concrete edges

$$c_{a1} = c_{a2} = 12 \text{ in.}$$

$$h_a = 18 \text{ in.}$$

Concrete

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

Stud material (A29/A108)*

$$f_{ya} = 51 \text{ ksi}$$

$$f_{uta} = 65 \text{ ksi}$$

Plate

3 x 3 x 3/8 in. thick

$$F_y = 36 \text{ ksi}$$

Loads

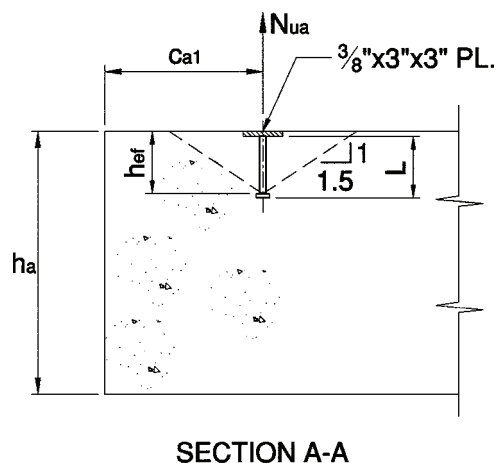
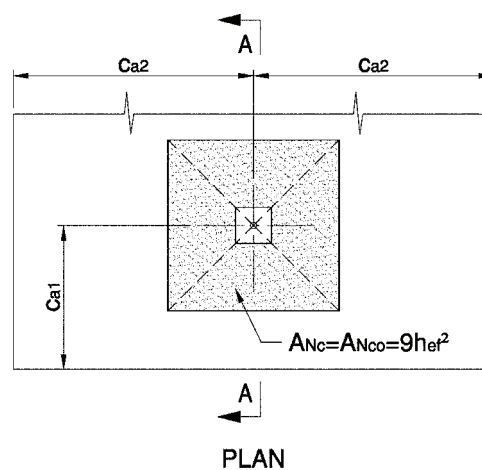
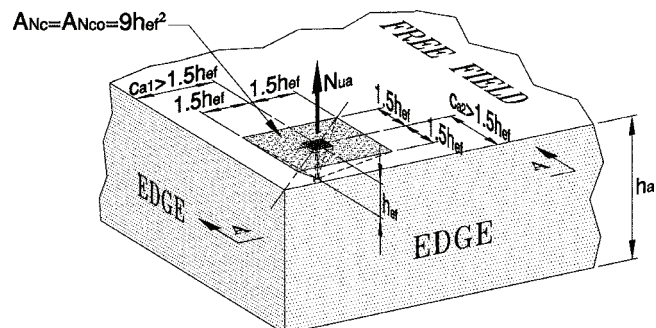
$$N_{ua} = 8 \text{ kips}$$

Where N_{ua} is the applied factored external load using load factors from Appendix C of the Code.

Assumptions:

- Concrete is cracked.
- ϕ -factors are based on Condition B in D.4.5 of the Code (no supplementary reinforcement).

*Stud material is A29/A108, material properties per AWS D1.1, 2006, Table 7.1, Type B stud. Yield strength = 51 ksi; tensile strength = 65 ksi. It has elongation of 20% and reduction in area of 50%; meets the definition of a ductile steel element given in D.1, and meets the tensile strength requirements of D.5.1.2 and D.6.1.2: $f_{uta} \leq 1.9f_{ya}$ ($65 \leq 1.9 \times 51 = 96.9 < 125 \text{ ksi}$).



CODE SECTION	DESIGN PROCEDURE	CALCULATION						
STEP 1: Determine required steel area and diameter of the stud.								
D.4.1.1 D.5.1.2	Equate the external factored load to the internal design strength and solve for the required steel area of the stud.	$\phi N_n \geq N_{ua}$ $N_n = N_{sa} = n A_{se} f_{uta}$ Equation no. (D-1) (D-3)						
D.3.6.1 D.4.5 D.5.1	Assume embedment will be designed as ductile in accordance with D.3.6.1 (in Step 2). Therefore, $\phi = 0.80$ for tension. Calculate nominal steel strength of selected stud. (Appendix A, Table 6) <table><tr><td>Anchor diameter, in.</td><td>Effective area A_{se}, in.²</td></tr><tr><td>3/8</td><td>0.110</td></tr><tr><td>1/2</td><td>0.196</td></tr></table> controls	Anchor diameter, in.	Effective area A_{se} , in. ²	3/8	0.110	1/2	0.196	$N_{ua} = 8 = \phi n A_{se} f_{uta}$ $= 0.80 \times 1.0 \times A_{se} \times 65 \text{ kips}$ $A_{se, req} = 0.154 \text{ in.}^2 \text{ required}$ Use one 1/2 in. diameter stud $A_{se} = 0.196 \text{ in.}^2 > 0.154 \text{ in.}^2$ (D-3)
Anchor diameter, in.	Effective area A_{se} , in. ²							
3/8	0.110							
1/2	0.196							
D.5.1.2	Calculate the nominal steel strength N_{sa} .	$N_{sa} = n A_{se} f_{uta}$ $= 1.0 \times 0.196 \times 65$ $= 12.74 \text{ kips}$ (D-3)						
D.5.1.2	Material properties are given. See footnote on previous page. Check that D.5.1.2 is met. (See also Table 6, Appendix A for additional stud properties.)	$f_{uta} = 65 \leq 1.9 f_{ya} = 1.9 \times 51 = 96.9 \text{ ksi}$ $\leq 125,000 \text{ psi}$ OK						
STEP 2: Determine required embedment length for the stud to prevent concrete breakout failure in tension.								
D.5.2 D.3.6.1	Calculate the required embedment depth for the stud to prevent concrete breakout failure. The depth will be selected so that the stud will be governed by the strength of the ductile steel element. This will produce a ductile embedment and justify the use of the ϕ -factor for steel used previously. In accordance with D.3.6.1, the design of the embedment will be controlled by the strength of the embedment steel. Following D.3.6.1, this goal is met when the nominal steel strength of the anchor, N_{sa} , is set equal to 0.85 times the nominal strength of the concrete-controlled strengths (N_{cbg} , N_p , etc.). Note that 0.85 is not a ϕ -factor.	From Step 1: $N_{sa} = 12.74 \text{ kips}$ $0.85 N_{cb, req} = N_{sa}$ $N_{cb, req} = N_{sa} / 0.85$ $= 12.74 / 0.85$ $= 14.99 \text{ kips}$						
D.5.2.1	Concrete breakout strength for a single stud: For a single stud away from edge:	$N_{cb} = (A_{Nc} / A_{Nco}) \psi_{ed, N} \psi_{c, N} \psi_{cp, N} N_b \quad \text{lb}$ (D-4) $A_{Nc} = A_{Nco} = 9 h_{ef}^2$ (D-6) $A_{Nc} / A_{Nco} = 1.0$						
D.5.2.5 D.5.2.6 D.5.2.7	Modification factors for: Edge effects $\psi_{ed, N}$ Concrete cracking $\psi_{c, N}$ Splitting control factor $\psi_{cp, N}$ applies to post-installed anchors only	$\psi_{ed, N} = 1.0$ (D-10) $\psi_{c, N} = 1.0$ $\psi_{cp, N} = \text{N/A for studs}$						
D.5.2.2	$k_c = 24$ for cast-in headed stud Assume $h_{ef} < 11 \text{ in.}$ Determine required embedment length $h_{ef, req}$.	$N_b = k_c \sqrt{f'_c} h_{ef}^{1.5} \text{ lb}$ (D-7) $= 24 \sqrt{4000} h_{ef}^{1.5}$ $= 1518 h_{ef}^{1.5} \text{ lb}$ $= 1.52 h_{ef}^{1.5} \text{ kips}$ $N_{cb, req} = 14.99 \text{ kips}$ $= 1.0 \times 1.0 \times 1.0 \times 1.52 h_{ef}^{1.5}$ $h_{ef, req} = 4.60 \text{ in.}$ Use 1/2 x 4-3/4 in. long stud.*						
	The embedment length is calculated as the total stud length, minus head thickness, plus plate thickness, minus burnoff. Standard length and head dimensions are given by the manufacturer. Typical values are given in Table 6, Appendix A.	$h_{ef, provided} = 4.75 - 0.312 + 0.375 - \text{burnoff (0.125 in.)}$ $= 4.69 \text{ in.} > 4.60 \text{ in.}$ OK						
	Calculate N_{cb} using $h_{ef, provided}$.	$N_{cb} = 1.52 \times 4.69^{1.5}$ $= 15.44 \text{ kips} \geq N_{cb, req} = 14.99 \text{ kips}$ OK						

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 3: Check pullout strength of stud.		
D.5.3 D.3.6.1	Calculate the pullout strength of the stud in tension in accordance with D.5.3. Design embedment as ductile in accordance with D.3.6.1.	
D.5.3.1	Concrete is cracked per problem statement.	$N_{pn} = \psi_{c,p} N_p$ (D-14)
D.5.3.4	Calculate pullout strength of anchor.	$N_p = 8A_{brg} f'_c$ $= 8A_{brg} \times 4$ $= 32A_{brg}$ kips (D-15)
D.5.3.5	$\psi_{c,p} = 1.0$ for cracked concrete.	$\psi_{c,p} = 1.0$
	Calculate the bearing area. From manufacturer data, stud head diameter is 1.0 in. for a 1/2 in. diameter stud (see also Table 6 in Appendix A). The number of studs is denoted as n .	$A_{brg} = n \times (1.0^2 - 0.5^2)/4$ $= 0.59 \text{ in.}^2$
		$N_{pn} = 1.0 \times 32 \times 0.59$ $= 18.88 \text{ kips}$
D.3.6.1	Design embedment as ductile, in accordance with D.3.6.1: $0.85N_{pn} \geq N_{sa}$	$0.85N_{pn} = 0.85 \times 18.88$ $= 16.05 \text{ kips} > N_{sa} = 12.74 \text{ kips}$
		Therefore ductile OK
		Use 1/2 in. diameter x 4-3/4 in. long stud.
STEP 4: Check concrete side-face blowout.		
D.5.4	Because this stud is far away from the an edge, side-face blowout N_{sb} will not be a factor, and will not be checked in this example.	N/A
STEP 5: Summary.		
Given	Applied load	$N_{ua} = 8 \text{ kips}$
Step 1 D.4.5.a	Design steel tensile strength	$\phi N_{sa} = 0.8 \times 12.74 = 10.19 \text{ kips}$
Step 2 D.4.5.c	Design concrete breakout strength	$\phi N_{cb} = 0.75 \times 15.44 = 11.58 \text{ kips}$
Step 3 D.4.5.c	Design concrete pullout strength	$\phi N_{pn} = 0.75 \times 18.88 = 14.16 \text{ kips}$
Step 4 D.4.5.c	Design concrete side-face blowout strength	$\phi N_{sb} = \text{N/A}$
D.4.1.2	Design strength of stud in tension	$\phi N_n = \min(\phi N_{sa}, \phi N_{cb}, \phi N_{pn})$ $= \min(10.19, 11.58, 14.16)$ $= 10.19 \text{ kips} > N_{ua} = 8 \text{ kips}$ OK
D.3.6.1	Ductility	$\min(0.85N_{cb}, 0.85N_{pn}) > N_{sa}$ $\min(0.85 \times 15.44, 0.85 \times 18.88)$ $= 13.12 > 12.74 \text{ kips}$ OK
STEP 6: Check plate thickness.		
AISC	Because the load is applied directly over the stud, the only requirement on plate thickness is that it satisfies the minimum thickness required for stud welding.	Stud welding of 1/2 in. diameter studs is acceptable on 3/8 in. thick plate per D.6.2.3. OK
*The length of stud selected is not a standard length. User should consult with manufacturer(s) for readily available lengths. In the above example, the effective embedment length h_{ef} is taken to the face of the concrete. If the plate was larger than the projected surface area, then the embedment length would exclude the thickness of the embedded plate.		

Example A2—Single stud, shear only

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

Given:

Edges

$$c_{a1} = 10 \text{ in.}$$

$$c_{a2} = 18 \text{ in.}$$

$$h_a = 18 \text{ in.}$$

Concrete

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

Stud material (A29/A108)*

$$f_{ya} = 51 \text{ ksi}$$

$$f_{uta} = 65 \text{ ksi}$$

Plate

Assume 3 x 3 x 3/8 in. thick

$$F_y = 36 \text{ ksi}$$

Loads

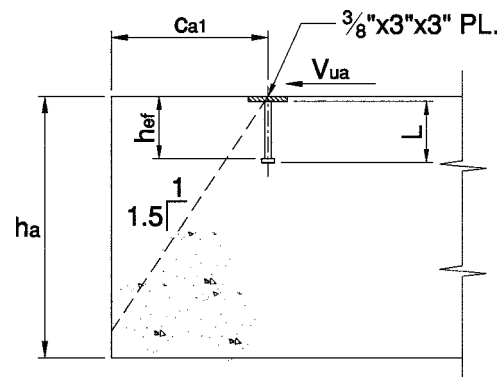
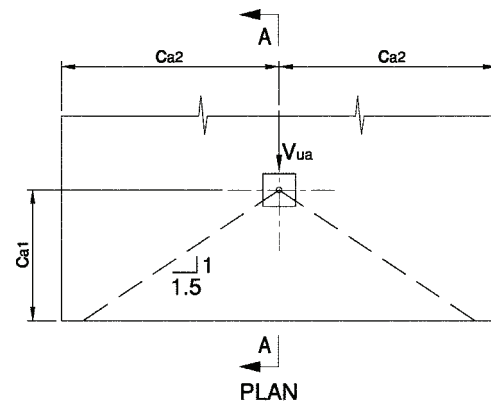
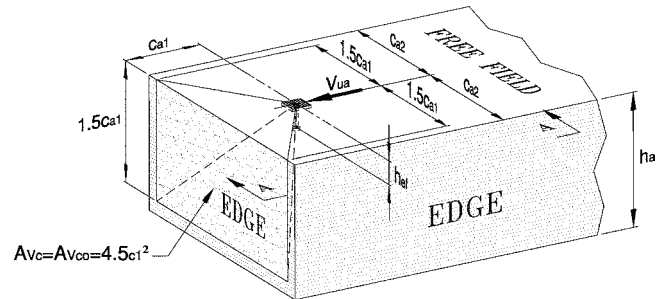
$$V_{ua} = 6 \text{ kips}$$

Where V_{ua} is the applied factored external load using load factors from Appendix C of the Code.

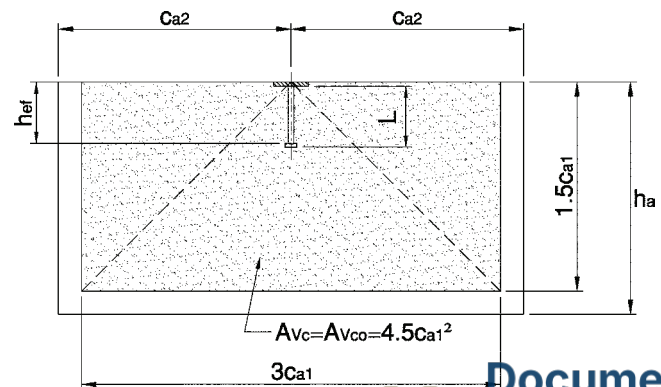
Assumptions:

- Concrete is cracked.
- ϕ -factors are based on Condition B in D.4.5 of the Code (no supplementary reinforcement).

*Stud material is A29/A108, material properties per AWS D1.1, 2006, Table 7.1, Type B stud. Yield strength = 51 ksi; tensile strength = 65 ksi. It has elongation of 20% and reduction in area of 50%; meets the definition of a ductile steel element given in D.1, and meets the tensile strength requirements of D.5.1.2 and D.6.1.2: $f_{uta} \leq 1.9f_{ya}$ ($65 \leq 1.9 \times 51 = 96.9 < 125 \text{ ksi}$).



SECTION A-A



FRONT ELEVATION

CODE SECTION	DESIGN PROCEDURE	CALCULATION									
STEP 1: Determine required steel area of the stud.											
D.4.1 D.6.1.2	Equate the external factored load to the internal design strength and solve for the required steel area of the stud.	Equation no. $\phi V_n \geq V_{ua}$ $V_n = V_{sa} = n A_{se} f_{uta}$ (D-2) (D-18)									
D.3.6.1 D.4.5	Assume embedment will be designed as ductile in accordance with D.3.6.1 (in Step 2). Therefore, $\phi = 0.75$ for shear loads.	$A_{se, req} = V_{ua} / (\phi n f_{uta})$ $\phi = 0.75$									
D.6.1	Design required area of stud using the steel strength provision (Appendix A, Table 6)	$A_{se, req} = 6 / (0.75 \times 1 \times 65)$ $= 0.123 \text{ in.}^2$ required									
	<table border="0"> <tr> <td><u>Anchor diameter, in.</u></td><td><u>Effective area A_{se}, in.²</u></td><td></td></tr> <tr> <td>3/8</td><td>0.110</td><td></td></tr> <tr> <td>1/2</td><td>0.196</td><td>controls</td></tr> </table>	<u>Anchor diameter, in.</u>	<u>Effective area A_{se}, in.²</u>		3/8	0.110		1/2	0.196	controls	Use one 1/2 in. diameter stud $A_{se} = 0.196 \text{ in.}^2 > 0.123 \text{ in.}^2$ $V_{sa} = n A_{se} f_{uta}$ $= 1.0 \times 0.196 \times 65$ $= 12.74 \text{ kips}$ OK (D-18)
<u>Anchor diameter, in.</u>	<u>Effective area A_{se}, in.²</u>										
3/8	0.110										
1/2	0.196	controls									
D.6.1.2	Calculate the nominal shear capacity of the selected stud.	$f_{uta} = 65 \leq 1.9 f_{ya} = 1.9 \times 51 = 96.9 \text{ ksi}$ $\leq 125 \text{ ksi}$ OK									
	Material properties are given. See footnote on previous page. Check that D.6.1.2 is met. (See also Table 6, Appendix A for additional stud properties.)										
STEP 2: Determine required edge distance to prevent concrete breakout failure in shear.											
D.3.6.1	Ensure that the embedment design is controlled by the strength of the embedment steel. The requirement for ductile design is given in D.3.6.1. For shear load, this requires that: $0.85 V_{cb} \geq A_{se} f_{uta}$	$0.85 V_{cb} \geq A_{se} f_{uta}$ $\min V_{cb} = A_{se} f_{uta} / 0.85$ $= 0.196 \times 65 / 0.85$ $= 14.99 \text{ kips}$									
D.6.2.1	Calculate concrete breakout strength V_{cb} in shear for a single stud.	$V_{cb} = (A_{Vc} / A_{Vco}) \psi_{ed, V} \psi_{c, V} V_b$ (D-20)									
	Calculate projected area for a single stud. See figures above for illustration of A_{Vco} . Because edges are far enough away, A_{Vc} and A_{Vco} are equal.	$A_{Vco} = 4.5 c_{a1}^2$ $A_{Vc} = A_{Vco}$ $A_{Vc} / A_{Vco} = 1.0$ (D-22)									
D.6.2.3	For cast-in headed studs, or headed bolts, that are welded to steel attachments having a minimum thickness equal to the greater of 3/8 in. or half the anchor diameter, the basic concrete breakout strength V_b is determined using D.6.2.3.	$V_b = 8 (\ell_e / d_o)^{0.2} \sqrt{d_o} \sqrt{f'_c} c_{a1}^{1.5}$ (D-24)									
	See definition in D.6.2.2 for limits on ℓ_e .	$\ell_e \leq 8 d_o$ Assume $\ell_e = 2.5 \text{ in.}$ $(\ell_e / d_o) = 2.5 / 0.5 = 5.0$									
		$V_b = 8 \times 5^{0.2} \times 0.5^{0.5} \times 4000^{0.5} \times c_{a1}^{1.5}$ $= 494 c_{a1}^{1.5} \text{ lb}$ $= 0.494 c_{a1}^{1.5} \text{ kips}$									
D.6.2.6 D.6.2.7	Modification factors for shear for: Edge effects $\psi_{ed, V}$ Concrete cracking $\psi_{c, V}$	$\psi_{ed, V} = 1.0$ $\psi_{c, V} = 1.0$									
	Concrete is cracked per problem statement. No additional supplementary steel is provided.	$V_{cb, req} = 14.99 \text{ kips}$ $= 1.0 \times 1.0 \times 1.0 \times 0.494 c_{a1}^{1.5}$ $= 0.494 \times c_{a1}^{1.5} \text{ kips}$									
		$c_{a1, req} = 9.73 \text{ in. (required)} < 10 \text{ in. (provided)}$ OK									
		Strength controlled by steel									
	Calculate V_{cb} using $c_{a1} = 10 \text{ in. provided.}$	$V_{cb} = 0.494 \times 10^{1.5} = 15.62$ $> 12.74 \text{ kips}$									

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 3: Check the required embedment length for the stud to prevent concrete pryout failure.		
D.6.3	Determine the required effective embedment length to prevent pryout.	$V_{sa} = 12.74$ kips See Step 1 (Note: same value as for tension N_{sa}) (D-18)
D.3.6.1	Ductility requirements of D.3.6.1 shall be satisfied: $0.85V_{cp} \geq V_{sa}$	$0.85V_{cp} \geq V_{sa} = 12.74$ $V_{cp,req} = 12.74/0.85$ $= 14.99$ kips
D.6.3	Design required embedment depth, from the concrete pryout strength requirement. Assume $h_{ef} > 2.5$ in. Therefore, $k_{cp} = 2.0$. N_{cb} is the required concrete breakout strength in tension. Calculate the required embedment depth of the anchor to prevent breakout. The approach is identical to that for tension used in Example A1 .	$V_{cp} = k_{cp}N_{cb}$ $k_{cp} = 2.0$ (D-28) $N_{cb,req} = 14.99/2.0$ $= 7.50$ kips (required)
D.5.2	Because this is a single stud away from edges, modification factors are all 1.	$N_{cb} = (A_{Nc}/A_{Nco})\psi_{ed,N}\psi_{c,N}N_b$ (D-4) $A_{Nc}/A_{Nco} = 1.0$ $\psi_{ed,N} = 1.0$ $\psi_{c,N} = 1.0$
D.5.2.2	Basic concrete breakout strength for a single anchor in tension: $k_c = 24$ for cast-in headed studs. Assume $h_{ef} < 11$ in.	$N_b = k_c \sqrt{f'_c} h_{ef}^{1.5}$ (D-7) $= 2400 \sqrt{4000} h_{ef}^{1.5}$ $= 1.52 h_{ef}^{1.5}$ kips $7.50 = 1.0 \times 1.0 \times 1.0 \times 1.52 h_{ef}^{1.5}$ $h_{ef,req} = 2.90$ in. (required) Use 1/2 x 3-1/2 in. long stud.
	See Appendix A, Table 6 , for stud head dimensions. Note that 0.312 in. is head thickness and 0.125 in. is burnoff.	h_{ef} provided: $h_{ef} = 3.5 - 0.312 - 0.125$ $= 3.06$ in. > 2.90 OK
	Calculate V_{cp} using $h_{ef} = 3.06$ in.	$V_{cp} = k_{cp}N_{cb}$ $= 2.0 \times 1.52 \times 3.06^{1.5}$ $= 16.27 > 12.74$ kips OK
STEP 4: Check pullout strength of stud to check head of the stud.		
	Checking of stud head is required to develop the concrete breakout strength N_{cb} used to check concrete pryout.	
D.5.3	Procedure is the same as that used in Example A1 . Calculate the nominal pullout strength N_{pn} of the anchor in tension in accordance with D.5.3.	$N_{pn} = \psi_{c,p}N_p$ (D-14)
D.5.3.4		$N_p = 8A_{brg}f'_c$ (D-15) $= 8 \times 4 \times A_{brg}$ $= 32A_{brg}$ kips
D.5.3.5	Concrete is cracked per problem statement. Therefore, $\psi_{c,p} = 1.0$. Bearing area is based on manufacturer data. (See Table 6 in Appendix A .)	$\psi_{c,p} = 1.0$ $A_{brg} = 0.589$ in. ² $N_{pn} = 1 \times 32 \times 0.589$ $= 18.8$ kips
D.3.6.1	Design embedment as ductile, in accordance with D.3.6.1: $0.85N_{pn} > N_{sa}$ N_{sa} for this problem is calculated in the pryout section shown in Step 3.	$0.85N_{pn} = 0.85 \times 18.8$ $= 16$ kips > $N_{sa} = 12.74$ kips Therefore ductile OK Use 1/2 in. diameter x 3-1/2 in. long stud Specified 1/2 in. diameter anchor head dimension OK

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 5: Summary of design strength.		
Given	Applied load	$V_{ua} = 6$ kips
Step 1 D.4.5.a	Design steel shear strength	$\phi V_{sa} = 0.75 \times 12.74 = 9.56$ kips OK
Step 2 D.4.5.c	Design concrete breakout strength	$\phi V_{cb} = 0.75 \times 15.62 = 11.72$ kips OK
Step 3 D.4.5.c	Design concrete pryout strength	$\phi V_{cp} = 0.75 \times 16.27 = 12.20$ kips
D.4.1.2	Design strength of stud in shear	$\phi V_n = \min(\phi V_{sa}, \phi V_{cb}, \phi V_{cp})$ $= \min(9.56, 11.72, 12.20)$ $= 9.56 > V_{ua} = 6$ kips OK
D.3.6.1	Ductility	$\min(0.85V_{cb}, 0.85V_{cp}) \geq V_{sa}$ $\min(0.85 \times 15.62, 0.85 \times 16.27)$ $= 13.28 > 12.74$ kips OK
STEP 6: Check plate thickness.		
AISC	Select plate thickness equal to or greater than 3/8 in. or half the anchor diameter. Tests* have also shown that the plate rupture is prevented when $d_o/t < 2.7$.	$t = \max(3/8 \text{ in.}, 0.5/2 = 0.25 \text{ in.})$ $= 3/8 \text{ in.}$ $\min t > 0.5/2.7 = 0.19 \text{ in.} < 3/8 \text{ in.}$ 3/8 in. thick plate is OK
*Goble, G. G., 1968, "Shear Strength of Thin Flange Composite Sections," <i>AISC Engineering Journal</i> , Apr.		

Example A3—Single stud, combined tension and shear

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

Given:

Edges

$$c_{a1} = 12 \text{ in.}$$

$$c_{a2} = 20 \text{ in.}$$

$$h_a = 18 \text{ in.}$$

Concrete

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

Stud material (A29/A108)*

$$f_{ya} = 51 \text{ ksi}$$

$$f_{uta} = 65 \text{ ksi}$$

Plate

3 x 3 x 3/8 in. thick

$$F_y = 36 \text{ ksi}$$

Loads

$$N_{ua} = 8 \text{ kips}$$

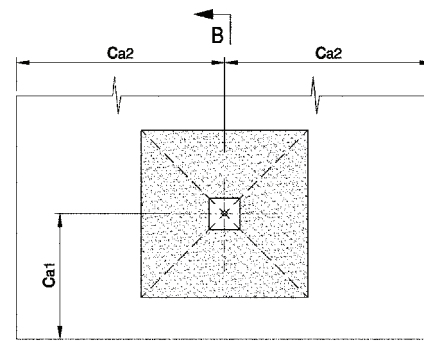
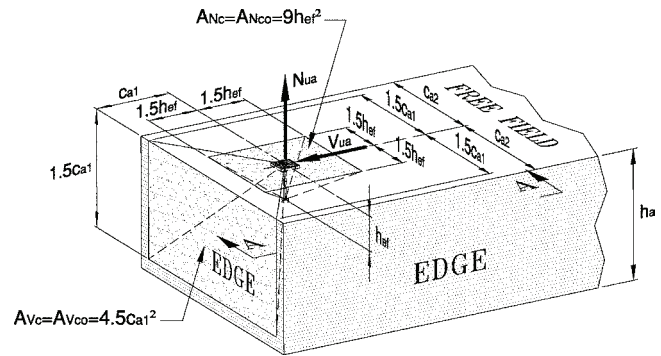
$$V_{ua} = 6 \text{ kips}$$

Where N_{ua} and V_{ua} are the applied factored external loads using load factors from Appendix C of the Code.

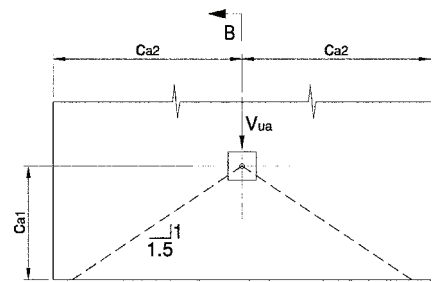
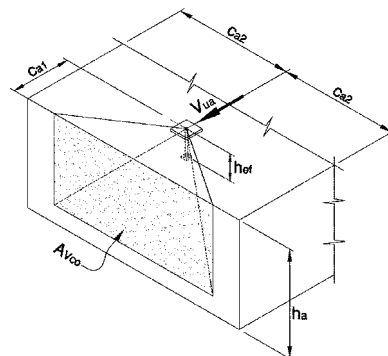
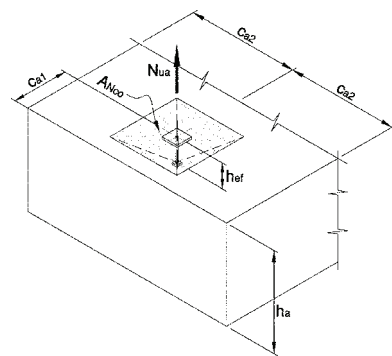
Assumptions:

- Concrete is cracked.
- ϕ -factors are based on Condition B in D.4.5 of the Code (no supplementary reinforcement).

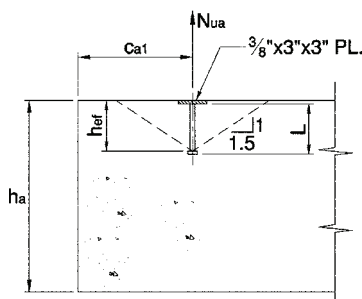
*Stud material is A29/A108, material properties per AWS D1.1, 2006, Table 7.1, Type B stud. Yield strength = 51 ksi; tensile strength = 65 ksi. It has elongation of 20% and reduction in area of 50%; meets the definition of a ductile steel element given in D.1, and meets the tensile strength requirements of D.5.1.2 and D.6.1.2: $f_{uta} \leq 1.9f_{ya}$ ($65 \leq 1.9 \times 51 = 96.9$ ksi).



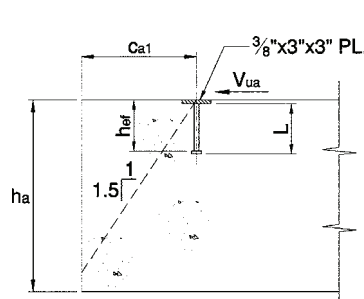
PLAN (TENSION ONLY)



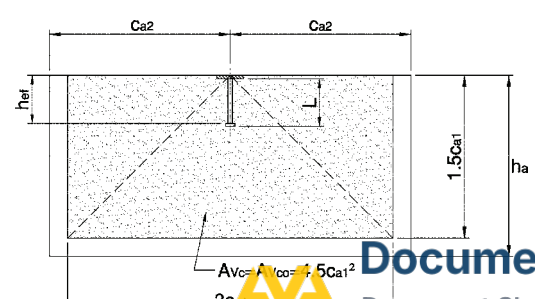
PLAN (SHEAR ONLY)



SECTION B-B (TENSION ONLY)



SECTION B-B (SHEAR ONLY)



FRONT ELEVATION

CODE SECTION	DESIGN PROCEDURE	CALCULATION						
STEP 1: Determine required steel area of the stud.								
D.4.1.1	Equate the external factored load to the internal design strength and solve for the required steel area of the stud.	$\phi N_n \geq N_{ua}$ $N_n = N_{sa} = n A_{se,t} f_{uta}$ Equation no. (D-1) (D-3)						
D.5.1 D.5.1.2	Use the tension provisions of D.5.1 to determine the required steel area for tension load. $A_{se,t}$ = required steel area for tension load.	$A_{se,t} = N_{ua}/(\phi n f_{uta})$ $= (8/0.80 \times 1.0 \times 65)$ $A_{se,t} = 0.154 \text{ in.}^2$						
D.3.6.1 D.4.5 D.5.1.2/ D.6.1.2	Assume embedment will be designed as ductile in accordance with D.3.6.1 (in Step 2). Therefore, $\phi = 0.80$ for tension and 0.75 for shear. Steel material is ductile. (See asterisked note on previous page.)	$A_{se,t} = 0.154 \text{ in.}^2$						
D.6.1	Use the shear provisions of D.6.1 to determine the required steel area for shear. $A_{se,v}$ = required steel area for shear. Add the area of steel required for tension to the area of steel required for shear.	$\phi V_n \geq V_{ua}$ $V_n = V_{sa} = n A_{se,v} f_{uta}$ (D-2) (D-17) $A_{se,v} = V_{ua}/(\phi n f_{uta})$ $= 6/(0.75 \times 1.0 \times 65)$ $= 0.123 \text{ in.}^2$						
D.7.3	Total required area $A_{se,req} = (A_{se,t} + A_{se,v})/1.2$ This assumes interaction between tension and shear, which will be checked in Step 8 . <table><tr><td><u>Anchor diameter, in.</u></td><td><u>Effective area A_{se}, in.²</u></td></tr><tr><td>1/2</td><td>0.196</td></tr><tr><td>5/8</td><td>0.307</td></tr></table> controls (Refer to Appendix A, Table 6) Use one 5/8 in. diameter stud. $A_{se} = 0.307 \text{ in.}^2 > 0.231 \text{ in.}^2$ Calculate the nominal steel strength N_{sa} . Calculate the nominal steel strength V_{sa} .	<u>Anchor diameter, in.</u>	<u>Effective area A_{se}, in.²</u>	1/2	0.196	5/8	0.307	$A_{se,req} = (0.154 + 0.123)/1.2 = 0.231 \text{ in.}^2$ $N_{sa} = n A_{se} f_{uta}$ $= 1.0 \times 0.307 \times 65$ $= 19.96 \text{ kips}$ $V_{sa} = n A_{se} f_{uta}$ $= 1.0 \times 0.307 \times 65$ $= 19.96 \text{ kips}$ OK
<u>Anchor diameter, in.</u>	<u>Effective area A_{se}, in.²</u>							
1/2	0.196							
5/8	0.307							
STEP 2: Determine required embedment length for the stud to prevent concrete breakout failure in tension.								
D.5.2 D.5.2.1	Calculate the required embedment depth for the stud to prevent concrete breakout failure. The depth will be selected so that the stud will be governed by the strength of the ductile steel element. This will produce a ductile embedment and justify the use of the ϕ -factor used previously. The steel capacity is based on the selected stud diameter.	From Step 1: $N_{sa} = 19.96 \text{ kips}$ (D-3)						
D.3.6.1	The requirements for ductile design are given in D.3.6.1. For tension load, this requires that $0.85 N_{cb} \geq N_{sa}$	$0.85 N_{cb,req} \geq N_{sa}$ $N_{cb,req} = N_{sa}/0.85$ $= 19.96/0.85$ $= 23.48 \text{ kips}$						
D.5.2.1	Calculate concrete breakout strength for a single anchor. For a single stud away from edge:	$N_{cb} = (A_{Nc}/A_{Nco}) \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad \text{lb}$ (D-4) $A_{Nc}/A_{Nco} = 1.0$						
D.5.2.5 D.5.2.6 D.5.2.7	Modification factors for: Edge effects $\psi_{ed,N}$ Concrete cracking $\psi_{c,N}$ $\psi_{cp,N}$ applies to post-installed anchors only	$\psi_{ed,N} = 1.0$ $\psi_{c,N} = 1.0$ $\psi_{cp,N} = \text{N/A for studs}$						



CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 2 (cont.)		
D.5.2.2	$k_c = 24$ for cast-in-place stud Assume $h_{ef} < 11$ in. Determine required embedment length $h_{ef,req}$. The embedment length is calculated as the total stud length, minus head thickness, plus plate thickness, minus burnoff. Head dimensions are given by the manufacturer. Typical values are given in Table 6, Appendix A. Calculate N_{cb} using $h_{ef,provided}$.	$N_b = k_c \sqrt{f'_c} h_{ef}^{1.5} \text{ lb} \quad (\text{D-7})$ $= 24 \sqrt{(4000)} h_{ef}^{1.5}$ $= 1.52 h_{ef}^{1.5} \text{ kips}$ $23.48 = 1.0 \times 1.0 \times 1.0 \times 1.52 h_{ef}^{1.5}$ $h_{ef,req} = 6.20 \text{ in. (required)}$ Use 5/8 x 6-3/4 in. long stud $h_{ef,provided} = 6.75 - 0.312 - 0.187 + 0.375 \text{ in.}$ $= 6.63 \text{ in.} > 6.20 \text{ in.} \quad \text{OK}$ $c_{a1} = 12 \text{ in.} > 1.5 h_{ef} = 1.5 \times 6.63 = 9.94 \text{ in.}$ Edge distance has no effect. $N_{cb} = 1.52 \times 6.63^{1.5} = 25.95 \text{ kips}$
STEP 3: Check pullout strength of stud.		
D.5.3	Stud head is required to develop the concrete breakout strength N_{cb}. Procedure is similar to that used in Example A1. Calculate the nominal pullout strength N_{pn} of the stud in tension in accordance with D.5.3.	$N_{pn} = \psi_{c,p} N_p \quad (\text{D-14})$ $N_p = 8 A_{brg} f'_c \quad (\text{D-15})$ $= 8 \times A_{brg} \times 4$ $= 32 A_{brg} \text{ kips}$
D.5.3.4 D.5.3.5	Concrete is cracked per problem statement. Therefore, $\psi_{c,p} = 1.0$. Bearing area is based on manufacturer data. (Appendix A, Table 6).	$\psi_{c,p} = 1.0$ $A_{brg} = 0.92 \text{ in.}^2$ $N_{pn} = 1.0 \times 32 \times 0.92$ $= 29.44 \text{ kips}$ Table 6
D.3.6.1	Design embedment as ductile, in accordance with D.3.6.1: $0.85 N_{pn} \geq N_{sa}$ N_{sa} for this problem is calculated in Step 1.	$0.85 N_{pn} = 0.85 \times 29.44$ $= 25.02 \text{ kips} > N_{sa} = 19.96 \text{ kips}$ Therefore ductile 5/8 in diameter x 6-3/4 in. long stud. OK
STEP 4: Check concrete side-face blowout.		
D.5.4 RD.5.4	Because this stud is far away from an edge, side-face blowout N_{sb} will not be a factor. According to the commentary, side-face blowout is not a concern if $c_{a,min} > 0.4 h_{ef}$. In this example: $h_{ef} = 6.63 \text{ in.}$ $0.4 h_{ef} = 0.4 \times 6.63 = 2.6 \text{ in.}$ $c_{a,min} = 12 \text{ in.} > 2.6 \text{ in.}$ Because $c_{a,min} > 0.4 h_{ef}$, side-face blowout calculation is not required. The calculation will be done to illustrate the method.	$N_{sb} = 160 c \sqrt{A_{brg}} \sqrt{f'_c} \quad (\text{D-16})$ $c_{a,min} = 12 \text{ in.}$ $A_{brg} = 0.92 \text{ in.}^2$ $f'_c = 4000 \text{ psi}$ $N_{sb} = 160 \times 12 \times 0.92^{0.5} \times 4000^{0.5} = 116.5 \text{ kips}$ $0.85 N_{sb} = 99.0 \text{ kips} > N_{sa} = 19.96 \text{ kips} \quad \text{OK}$ 5/8 in. diameter x 6-3/4 in. long stud. OK

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 5: Determine required edge distance to prevent concrete breakout failure in shear.		
D.3.6.1	Ensure that the embedment design is controlled by the strength of the embedment steel. The requirement for ductile design is given in D.3.6.1. For shear load, this requires that: $0.85V_{cb} > A_{se}f_{uta}$	$0.85V_{cb} > A_{se}f_{uta}$ $V_{cb,req} = A_{se}f_{uta}/0.85$ $= 0.307 \times 65/0.85$ $= 23.48 \text{ kips}$
D.6.2.1	Calculate concrete breakout strength V_{cb} in shear for a single stud.	$V_{cb} = (A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}V_b$ (D-20)
	Calculate projected area for a single stud. See figures above for illustration of A_{Vco} . Because edges are far enough away, A_{Vc} and A_{Vco} are equal.	$A_{Vco} = 4.5c_{a1}^2$ (D-22) $A_{Vc} = A_{Vco}$ $A_{Vc}/A_{Vco} = 1.0$
D.6.2.3	For cast-in headed studs, or headed bolts, that are welded to steel attachments having a minimum thickness equal to the greater of 3/8 in. or half the anchor diameter, the basic concrete breakout strength V_b is determined using D.6.2.3.	$V_b = 8(\ell_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f'_c} c_{a1}^{1.5}$ (D-24)
	See definition in D.6.2.2 for limits on ℓ_e .	$\ell_e/d_o = 6.63/0.625 = 10.61 > 8.0$ Use $\ell_e/d_o = 8.0$
D.6.2.6 D.6.2.7	Modification factors for shear for: Edge effects $\Psi_{ed,V}$ Concrete cracking $\Psi_{c,V}$	$\Psi_{ed,V} = 1.0$ $\Psi_{c,V} = 1.0$
	Concrete is cracked per problem statement. No additional supplementary steel is provided.	$23.48 = 1.0 \times 1.0 \times 1.0 \times 0.606c_{a1,req}^{1.5}$ $c_{a1,req} = 11.45 \text{ in.} < 12.0 \text{ in.}$ OK
Step 1	Steel strength in shear V_{sa} . Calculate V_{cb} using $c_{a1} = 12 \text{ in.}$ provided.	Strength controlled by steel $V_{sa} = 19.96 \text{ kips}$ $V_{cb} = 0.606c_{a1}^{1.5}$ $= 0.606(12)^{1.5}$ $= 25.19 \text{ kips}$
STEP 6: Check concrete pryout failure.		
D.6.3	Pryout strength of the stud is checked using D.6.3. For an anchor or stud designed for tension, this will not govern. The calculations are for illustration of this method.	
D.3.6.1	Ductility requires of D.3.6.1 shall be satisfied: $0.85V_{cp} > V_{sa}$	$0.85V_{cb} > V_{sa}$ $V_{cb,req} = V_{sa}/0.85$ $= 19.96/0.85$ $= 23.5 \text{ kips}$
D.6.3	N_{cb} is calculated in Step 2	$V_{cp} = k_{cp}N_{cb}$ (D-28) $k_{cp} = 2.0$ $N_{cb} = 25.95 \text{ kips}$ $V_{cp} = 2 \times 25.95$ $= 51.9 \text{ kips} >> 23.5 \text{ kips}$ 5/8 in. diameter x 6-3/4 in. long stud OK

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 7: Summary		
D.4.1.2	TENSION	
	Applied load	$N_{ua} = 8$ kips
Step 1 D.4.5.a	Steel tensile strength	$\phi N_{sa} = 0.8 \times 19.96 = 15.97$ kips
Step 2 D.4.5.c	Concrete breakout strength	$\phi N_{cb} = 0.75 \times 25.95 = 19.46$ kips
Step 3 D.4.5.c	Concrete pullout strength	$\phi N_{pn} = 0.75 \times 29.44 = 22.08$ kips
Step 4 D.4.5.c	Concrete side-face blowout strength	$\phi N_{sb} = 0.75 \times 116.5 = 87.40$ kips
D.4.1.2	Design strength of stud in tension	$\phi N_n = \min(\phi N_{sa}, \phi N_{cb}, \phi N_{pn})$ $= \min(15.97, 19.46, 22.08, 87.40)$ $= 15.97$ kips $> N_{ua} = 8$ kips
	SHEAR	
	Applied load	$V_{ua} = 6$ kips
Step 1 D.4.5.a	Steel strength	$\phi V_{sa} = 0.75 \times 19.96 = 14.97$ kips
Step 2 D.4.5.c	Concrete breakout strength	$\phi V_{cb} = 0.75 \times 25.19 = 18.89$ kips
Step 3 D.4.5.c	Concrete pryout strength	$\phi V_{cp} = 0.75 \times 51.9 = 38.9$ kips
D.4.1.2	Design strength of stud in shear	$\phi V_n = \min(\phi V_{sa}, \phi V_{cb}, \phi V_{cp})$ $= \min(14.97, 18.89, 38.9)$ $= 14.97$ kips $> V_{ua} = 6$ kips
D.3.6.1	Ductility tension:	$\min(0.85N_{cb}, 0.85N_{pn}, 0.85N_{sb}) \geq N_{sa}$ $\min(0.85 \times 25.95, 0.85 \times 29.44, 0.85 \times 116.5)$ $\min(22.06, 25.02, 99.03)$ $= 22.06 > N_{sa} = 19.96$ kips
	Ductility shear:	$\min(0.85V_{cb}, 0.85V_{cp}) > V_{sa}$ $\min(0.85 \times 25.19, 0.85 \times 51.9)$ $\min(21.41, 44.12)$ $= 21.41 > V_{sa} = 19.96$ kips
STEP 8: Check interaction of tension and shear forces		
D.7 D.7.1	$V_{ua}/\phi V_n > 0.2$ Full strength in tension shall not be permitted.	$V_{ua}/\phi V_n = 6.0/14.97 = 0.40 > 0.2$
D.7.2	$N_{ua}/\phi N_n > 0.2$ Full strength in shear shall not be permitted.	$N_{ua}/\phi N_n = 8.0/15.97 = 0.50 > 0.2$
D.7.3	$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2$	$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} = 0.50 + 0.40 = 0.90$ $0.90 < 1.2$
STEP 9: Calculate minimum plate thickness		
D.6.2.3	Select plate thickness equal to or greater than 3/8 in. or half the anchor diameter. Tests* have also shown that plate rupture is prevented when $d_o/t < 2.7$.	$t = \max(3/8 \text{ in.}, 0.5/2 = 0.25 \text{ in.})$ $= 3/8 \text{ in.}$ $t_{req} > 0.5/2.7 = 0.19 \text{ in.} < 3/8 \text{ in.}$ 3/8 in. thick plate
*Goble, G. G., 1968, "Shear Strength of Thin Flange Composite Sections," <i>AISC Engineering Journal</i> , Apr.		



Example A4—Single bolt, combined tension and shear

Design an embedment using a high-strength bolt, F 1554 Gr. 105 (AB105).

Given:

Edges

$$c_{a1} = 24 \text{ in.}$$

$$c_{a2} = 24 \text{ in.}$$

$$h_a = 36 \text{ in.}$$

Concrete

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

Bolt material (AB105)*

$$f_{ya} = 105 \text{ ksi}$$

$$f_{uta} = 125 \text{ ksi}$$

Plate (face mounted)

3/8 in. thick

$$F_y = 36 \text{ ksi}$$

Loads

$$N_{ua} = 40 \text{ kips}$$

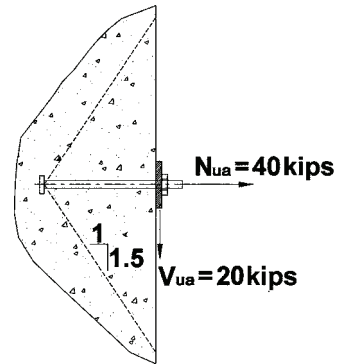
$$V_{ua} = 20 \text{ kips}$$

Where N_{ua} and V_{ua} are the applied factored external loads using load factors from Appendix C of the Code.

Assumptions:

- Concrete is cracked.
- ϕ -factors are based on Condition B in **D.4.5** of the Code (no supplementary reinforcement).

*ASTM F 1554-00 specification, Grade 105, Class 1A, bolt material will be used. Bolt identification is AB105 with a tensile strength in the range of 125 to 150 ksi, and minimum yield strength of 105 ksi for 1/4 and 3/8 in. diameters. Reductions in area requirements may vary. For anchor diameters < 2 in., elongation in 2 in. is 15% and reduction in area is 45% and meets the definition of a ductile steel element given in **D.1**. Also, $\max f_{uta} = 1.4f_{ya}$. According to **D.6.1.2**, f_{uta} shall be $\leq 1.9f_{ya}$ or 125,000 psi. See also **Table 1 in Appendix A** for other materials.



*Note: the breakout failure planes are similar to the failure planes shown in the figures in **Example A3**.*

CODE SECTION	DESIGN PROCEDURE	CALCULATION						
STEP 1: Determine required steel area of the bolt.								
D.4.1.1	Equate the external factored load to the internal design strength and solve for the required steel area of the bolt.	$\phi N_n \geq N_{ua}$ $N_n = N_{sa} = n A_{se,t} f_{uta}$ Equation no. (D-1) (D-3)						
D.5.1 D.5.1.2	Use the tension provisions of D.5.1 to determine the required steel area for tension load. $A_{se,t}$ = required steel area for tension load.	$A_{se,t} = N_{ua}/(\phi n f_{uta})$ $= (40/0.80 \times 1.0 \times 125) = 0.40 \text{ in.}^2$						
D.3.6.1 D.4.5a	Assume embedment will be designed as ductile in accordance with D.3.6.1 (in Step 2). Therefore, $\phi = 0.80$ for tension and 0.75 for shear. Assume bolt is far away from edge.							
D.6.1 D.4.5.a D.6.1.2	Use the shear provisions of D.6.1 to determine the required steel area for shear load. $A_{se,v}$ = required steel area for shear.	$\phi V_n \geq V_{ua}$ $V_n = V_{sa} = n 0.6 A_{se,v} f_{uta}$ $A_{se,v} = V_{ua}/(n 0.6 \phi f_{uta})$ $= 20/(1.0 \times 0.6 \times 0.75 \times 125)$ $= 0.36 \text{ in.}^2$ (D-2) (D-18)						
D.7.3	Add the area of steel required for tension to the area of steel required for shear. Total required area $A_{se,req} = (A_{se,t} + A_{se,v})/1.2$ This assumes interaction between tension and shear, which will be checked in Step 8. <table><tr><td>Anchor diameter, in.</td><td>Effective area A_{se}, in.²</td></tr><tr><td>1.0</td><td>0.606</td></tr><tr><td>1.125</td><td>0.763</td></tr></table> controls	Anchor diameter, in.	Effective area A_{se} , in. ²	1.0	0.606	1.125	0.763	$A_{se,req} = (0.40 + 0.36)/1.2 = 0.63 \text{ in.}^2$ Use one 1-1/8 in. diameter headed bolt. $A_{se} = 0.76 \text{ in.}^2 > 0.64 \text{ in.}^2$ OK
Anchor diameter, in.	Effective area A_{se} , in. ²							
1.0	0.606							
1.125	0.763							
	Calculate the nominal steel strength N_{sa} in tension.	$N_{sa} = n A_{se} f_{uta}$ $= 1.0 \times 0.76 \times 125$ $= 95.0 \text{ kips}$						
	Calculate the nominal steel strength V_{sa} in shear.	$V_{sa} = n 0.6 A_{se} f_{uta}$ $= 1.0 \times 0.6 \times 0.76 \times 125$ $= 57.0 \text{ kips}$						
STEP 2: Determine required embedment length for the bolt to prevent concrete breakout failure in tension.								
D.5.2	Calculate the required embedment depth for the bolt to prevent concrete breakout failure. The depth will be selected so that the anchor will be governed by the strength of the ductile steel element. This will produce a ductile embedment and justify the use of the ϕ -factor used previously. The steel capacity is based on the selected anchor diameter. Use N_{sa} to determine required h_{ef} .	From Step 1: $N_{sa} = 95.0 \text{ kips}$						
D.3.6.1	The requirements for ductile design are given in D.3.6.1. For tension load, this requires that $0.85 N_{cb} \geq N_{sa}$	$0.85 N_{cb,req} \geq N_{sa}$ $N_{cb,req} = N_{sa}/0.85$ $= 95/0.85$ $= 111.8 \text{ kips}$						
D.5.2.1	Calculate concrete breakout strength for a single bolt. For a single bolt away from edge:	$N_{cb} = (A_{Nc}/A_{Nco}) \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$ (D-4) $A_{Nc}/A_{Nco} = 1.0$						

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 2 (cont.)		
D.5.2.5 D.5.2.6 D.5.2.7	Modification factors for: Edge effects $\psi_{ed,N}$ Concrete cracking $\psi_{c,N}$ $\psi_{cp,N}$ applies to post-installed anchors only Basic concrete breakout strength: $k_c = 24$ for cast-in-place anchors. Determine the required effective embedment depth $h_{ef,req}$. Determine bolt length L . Calculate N_{cb} using $h_{ef,provided}$. Note: for $11 \text{ in.} \leq h_{ef} \leq 25 \text{ in.}$, the basic concrete breakout strength N_b can alternatively be calculated using Eq. (D-8) that yields larger loads. This was neglected in this example.	$\psi_{ed,N} = 1.0$ $\psi_{c,N} = 1.0$ $\psi_{cp,N} = N/A$ for bolts $N_b = k_c \sqrt{f'_c} h_{ef}^{1.5}$ $= 24 \sqrt{(4000)} h_{ef}^{1.5}$ $= 1.52 h_{ef}^{1.5}$ kips (D-7) $N_{cb} = (A_{Nc}/A_{Nco}) \psi_{ed,N} \psi_{c,N} N_b$ (D-4) $111.8 = 1.0 \times 1.0 \times 1.0 \times 1.52 h_{ef}^{1.5}$ $h_{ef,req} = 17.57 \text{ in.}$ $L = 17.57 + 0.75$ (thickness of head) $= 18.32 \text{ in.}$ Use 18.5 in. Use 1-1/8 in. diameter, F 1554 Gr. 105 bolt embedded 18.5 in. into the concrete. $h_{ef,provided} = 18.5 - 0.75$ $= 17.75 \text{ in.} > 17.57 \text{ in.}$ OK $N_{cb} = 1.52 \times 17.75^{1.5} = 113.7 \text{ kips}$
STEP 3: Check pullout strength of bolt.		
D.5.3	Procedure is the same as that used in Example A1. Calculate the pullout strength of the bolt in tension in accordance with D.5.3. Calculate pullout strength.	$N_{pn} = \psi_{c,p} N_p$ (D-14) $N_p = 8 A_{brg} f'_c$ $= 8 \times A_{brg} \times 4$ $= 32 A_{brg}$ (D-15)
D.5.3.5	Concrete is cracked per problem statement. $\psi_{c,p} = 1.0$ for cracked concrete. ASTM F 1554 recommends a heavy hex head and washer. Appendix A, Table 4(c), $A_{brg} = 1.85 \text{ in.}^2$	$\psi_{c,p} = 1.0$
D.3.6.1	For this combination of steel strength and concrete strength, the anchor head alone is not sufficient to develop the required bearing strength. Find required net bearing area. Design embedment as ductile, in accordance with D.3.6.1: $0.85 N_{pn} \geq N_{sa}$ See Appendix A, Table 4, for anchor head areas. Design a washer to meet the required bearing area. From Table 5, one can see that the SAE washers will not work, and U.S. standard washers seem too thin. Try a square plate.	$N_{pn} = 1.0 \times 32 \times 1.85 = 59.2 \text{ kips}$ $< N_{sa} = 95 \text{ kips}$ (No good) $0.85 N_{pn} = N_{sa} = 95 \text{ kips}$ (N_{sa} from Step 1) $N_{pn,req} = 95 / 0.85 = 111.8 \text{ kips}$ $111.8 = 32 A_{brg}$ $A_{brg,req} = 111.8 / 32 = 3.49 \text{ in.}^2$ $A_D = 0.99 \text{ in.}^2$ anchor area $A_{total,req} = 3.49 + 0.99 = 4.48 \text{ in.}^2$ For 1-1/8 in. bolt, hex head area is: $A_H = 2.85 \text{ in.}^2$ $a_{req} = 4.48 - 2.85 = 1.63 \text{ in.}^2$ Use a square plate 2-1/4 in. each side $A_{washer} = 2.25^2$ $= 5.06 \text{ in.}^2 > 4.48 \text{ in.}^2$

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 3 (cont.)		
D.3.6.1	Calculate N_{pn} using $A_{brg,provided}$. Check ductility: $0.85N_{pn} > N_{sa}$. According to D.5.2.8, when adding a washer at the head of anchor, it is permitted to calculate the projected area of the failure surface by projecting the failure surface outward $1.5h_{ef}$ from the effective perimeter of the washer. Therefore, the concrete breakout strength can be revised.	$A_{brg,provided} = 5.06 - 0.99$ $= 4.07 \text{ in.}^2 > 3.49 \text{ in.}^2$ OK $N_{pn} = 1.0 \times 32 \times 4.07 = 130.24 \text{ kips}$ $0.85N_{pn} = 0.85 \times 130.24 = 110.7 \text{ kips} > N_{sa} = 95 \text{ kips}$ Use 1-1/8 in. diameter AB 105 bolt, embedded 18.5 in., with a square washer 2-1/4 in. sides and 3/8 in. thick.
STEP 4: Check concrete side-face blowout.		
D.5.4	Because this anchor is far away from an edge, side-face blowout will not be a factor, and will not be checked in this example.	
STEP 5: Determine required edge distance to prevent concrete breakout failure in shear.		
D.6.1.2	Compute nominal steel strength in shear.	$V_{sa} = n0.6A_{se}f_{uta}$ $= 1.0 \times 0.6 \times 0.76 \times 215$ $= 57 \text{ kips}$
D.3.6.1	Ensure that the embedment design is controlled by the strength of the embedment steel. The requirement for ductile design is given in D.3.6.1. For shear load, this requires that: $0.85V_{cb} > V_{sa}$	$0.85V_{cb} > V_{sa}$ $V_{cb,req} = V_{sa}/0.85$ $= 57/0.85$ $= 67.06 \text{ kips}$
D.6.2.1	Calculate concrete breakout strength V_{cb} in shear for a single anchor.	$V_{cb} = (A_{Vc}/A_{Vco})\psi_{ed,V}\psi_{c,V}V_b$ (D-20)
	Calculate projected area for a single stud. See figures (for Example A3) for illustration of A_{Vco} . Because edges are far enough away, A_{Vc} and A_{Vco} are equal.	$A_{Vco} = 4.5c_{a1}^2$ (D-22) $A_{Vc} = A_{Vco}$ $A_{Vc}/A_{Vco} = 1.0$
D.6.2.2	The basic concrete breakout strength V_b is determined using D.6.2.2. See definition in D.6.2.2 for limits on ℓ_e.	$V_b = 7(\ell_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f'_c} c_{a1}^{1.5}$ (D-23) $\ell_e/d_o = 17.75/1.125 = 15.78 > 8.0$ Use $\ell_e/d_o = 8.0$ $V_b = 7 \times 8^{0.2} \times 1.125^{0.5} \times 4000^{0.5} \times c_{a1}^{1.5}$ $= 712c_{a1}^{1.5} \text{ lb}$ $= 0.712c_{a1}^{1.5} \text{ kips}$
D.6.2.6 D.6.2.7	Modification factors for: Edge effects $\psi_{ed,V}$ Concrete cracking $\psi_{c,V}$ Concrete is cracked per problem statement. No additional supplementary steel is provided. Calculate V_{cb} using $c_{a1} = 24 \text{ in.}$ provided.	$\psi_{ed,V} = 1.0$ $\psi_{c,V} = 1.0$ $67.06 = 1.0 \times 1.0 \times 1.0 \times 0.712c_{a1,req}^{1.5}$ $c_{a1,req} = 20.72 \text{ in.} < 24.0 \text{ in.}$ OK Strength controlled by steel $V_{cb} = 0.712c_{a1}^{1.5}$ $= 0.712(24)^{1.5}$ $= 83.7 \text{ kips}$
STEP 6: Check concrete pryout failure.		
D.6.3	For a bolt designed for tension, as in Step 2, concrete pryout failure will not govern and, hence, this step is not required. V_{cp} is calculated for illustration. N_{cb} is calculated in Step 2.	$V_{cp} = k_{cp}N_{cb}$ (D-28) $k_{cp} = 2.0$ $N_{cb} = 113.7 \text{ kips}$ $V_{cp} = 2 \times 113.7$ $= 227.4 \text{ kips} >> 95 \text{ kips}$



CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 7: Summary.		
D.4.1.2	TENSION	
	Applied load	$N_{ua} = 40$ kips
Step 1 D.4.5.a	Steel strength	$\phi N_{sa} = 0.8 \times 95.0 = 76.0$ kips
Step 2 D.4.5.c	Concrete breakout strength	$\phi N_{cb} = 0.75 \times 113.7 = 85.3$ kips
Step 3 D.4.5.c	Concrete pullout strength	$\phi N_{pn} = 0.75 \times 130.24 = 97.68$ kips
Step 4 D.4.5.c	Concrete side-face blowout strength	N/A
	Design strength of stud in tension	$\phi N_n = \min(\phi N_{sa}, \phi N_{cb}, \phi N_{pn})$ $= \min(76.0, 85.3, 97.68)$ $= 76.0$ kips $> N_{ua} = 40$ kips
D.4.1.2	SHEAR	
	Applied load	$V_{ua} = 20$ kips
Step 1 D.4.5.a	Steel strength	$\phi V_{sa} = 0.75 \times 57.0 = 42.75$ kips
Step 2 D.4.5.c	Concrete breakout strength	$\phi V_{cb} = 0.75 \times 83.70 = 62.78$ kips
Step 3 D.4.5.c	Concrete pryout strength	$\phi V_{cp} = 0.75 \times 227.4 = 170.55$ kips
D.4.1.2	Design strength of stud in shear	$\phi V_n = \min(\phi V_{sa}, \phi V_{cb}, \phi V_{cp})$ $= \min(42.75, 62.78, 170.55)$ $= 42.75$ kips $> V_{ua} = 20$ kips
D.3.6.1	Ductility in tension:	$\min(0.85N_{cb}, 0.85N_{pn}, 0.85N_{sb}) \geq N_{sa}$ $\min(0.85 \times 113.7, 0.85 \times 130.24, \text{N/A})$ $\min(96.65, 110.70)$ $= 96.65 > N_{sa} = 95.0$ kips
	Ductility in shear:	$\min(0.85V_{cb}, 0.85V_{cp}) > V_{sa}$ $\min(0.85 \times 83.70, 0.85 \times 227.4)$ $\min(71.14, 193.29)$ $= 71.14 > V_{sa} = 57.0$ kips
STEP 8: Check interaction of tension and shear forces.		
D.7	$V_{ua}/\phi V_n > 0.2$	$V_{ua}/\phi V_n = 20/42.75$
D.7.1	Full strength in tension shall not be permitted.	$= 0.47 > 0.2$
D.7.2	$N_{ua}/\phi N_n > 0.2$	$N_{ua}/\phi N_n = 40/76.0$
	Full strength in shear shall not be permitted.	$= 0.53 > 0.2$
		$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2$
		$0.53 + 0.47 = 1.00 < 1.2$
STEP 9: Calculate minimum plate thickness.		
AISC	Select plate thickness using the appropriate steel code. This step is not included in this example.	

PART B—Examples: Ductile multiple embedded elements in semi-infinite concrete

Example B1(a)—Four-stud embedded plate, tension only, wide spacing

Design an embedment with four welded studs and an embedded plate for a 3 x 3 x 3/16 in. A501 structural tube attachment where anchors are spaced at least $3h_{ef}$ apart.

Given:

Concrete edges

$$c_{a1} = c_{a2} > c_{a,min} = 15 \text{ in.}$$

$$h_a = 18 \text{ in.}$$

Concrete

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

Stud material (A29/A108)*

$$f_{ya} = 51 \text{ ksi}$$

$$f_{uta} = 65 \text{ ksi}$$

Plate

$$F_y = 36 \text{ ksi}$$

Load

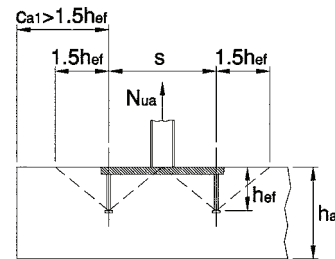
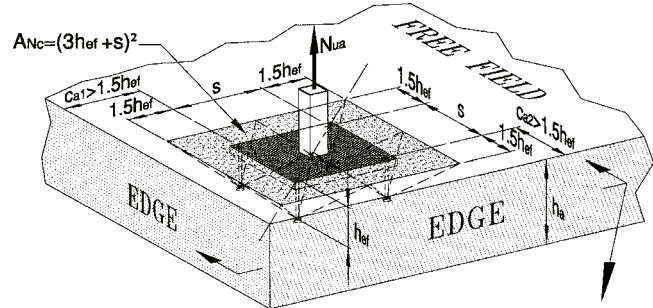
$$N_{ua} = 28 \text{ kips}$$

Where N_{ua} is the applied factored external loads using load factors from Appendix C of the Code. The wide spacing indicates that each of the four anchors develops full tensile capacity.

Assumptions:

- Concrete is cracked.
- ϕ -factors are based on Condition B in D.4.5 of the Code (no supplementary reinforcement).
- Ductile embedment design in accordance with D.3.6.1.

*Stud material is A29/A108, material properties per AWS D1.1, 2006, Table 7.1, Type B stud. Yield strength = 51 ksi; tensile strength = 65 ksi. It has elongation of 20% and reduction in area of 50%; meets the definition of a ductile steel element given in D.1, and meets the tensile strength requirements of D.5.1.2 and D.6.1.2: $f_{uta} \leq 1.9f_{ya}$ ($65 \leq 1.9 \times 51 = 96.9$ ksi).



CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 1: Determine the required stud diameter.		
D.4.1.1	Equate internal design strength ϕN_n to the external factored load N_{ua} .	$\phi N_n > N_{ua}$ Equation no. (D-1)
	In ductile design, the internal strength N_n is controlled by the steel strength of the stud, N_{sa} .	$N_n = N_{sa} = n A_{se} f_{uta}$
D.4.5	The required steel strength of the stud, $N_{sa,req}$, is multiplied by $\phi = 0.80$ for tension because the embedment and the steel stud is ductile and the load factors are based on Appendix C of the Code.	$0.80 N_{sa,req} \geq 28$ $N_{sa,req} = 28/0.8$ $= 35.0$ kips
D.5.1	Solve for the required steel area $A_{se,req}$ for a single stud.	$N_{sa,req} = n A_{se,req} f_{uta}$ $35.0 = 4 \times A_{se,req} \times 65$ $A_{se,req} = 35.0/(4 \times 65)$ $= 0.13 \text{ in.}^2$ (required)
	Anchor diameter, in. Effective area A_{se} , in. ² 3/8 0.110 1/2 0.196 controls (Refer to Appendix A, Table 6)	Use 1/2 in. diameter studs. $A_{se} = 0.196 \text{ in.}^2 > 0.13 \text{ in.}^2$ OK
D.5.1	Determine the nominal tensile strength N_{sa} of four 1/2 in. diameter studs	$N_{sa} = n A_{se} f_{uta}$ $= 4 \times 0.196 \times 65$ $= 51.0$ kips (D-3)
STEP 2: Determine the minimum embedment length and spacing for the studs to prevent concrete breakout failure in tension.		
D.3.6.1	Ensure steel strength controls: To prevent concrete breakout failure in tension, the required design concrete breakout tensile strength has to be greater than the nominal tensile strength of the embedment steel, N_{sa} . The design concrete breakout strength shall be taken as 0.85 times the nominal strength.	From Step 1: $N_{sa} = 51.0$ kips $0.85 N_{cbg,req} > N_{sa}$ $N_{cbg,req} > 51/0.85$ $N_{cbg,req} > 60.0$ kips
D.5.2	N_{cbg} is the nominal concrete breakout strength in tension of a group of anchors.	$N_{cbg} = (A_{Nc}/A_{Nco}) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$ (D-5)
D.5.2.1	A_{Nc} is not calculated because it is assumed that spacing is not limited. Therefore, only the ratio A_{Nc}/A_{Nco} is needed.	$A_{Nc} = 4 \times A_{Nco}$ $A_{Nc}/A_{Nco} = 4$
D.5.2.4	Modification factors are 1.0 for:	
D.5.2.5	Eccentricity effects $\psi_{ec,N}$ Edge effects $\psi_{ed,N}$ $c_{a,min} = 15$ in. per problem statement. Edge effect factor $\psi_{ed,N}$ will be 1.0 as long as $c_{a,min} \geq 1.5 h_{ef}$ or $h_{ef} \leq c_{a,min}/1.5$. Therefore, the embedment h_{ef} needs to be less than $15/1.5 = 10$ in. to ensure no reduction due to edge distance.	$\psi_{ec,N} = 1.0$ $\psi_{ed,N} = 1.0$
D.5.2.6	Concrete cracking $\psi_{c,N}$. Concrete is cracked per problem statement.	$\psi_{c,N} = 1.0$
D.5.2.7	$\psi_{cp,N}$ is not used for cast-in-place anchors.	$\psi_{cp,N} = \text{N/A}$ for studs
D.5.2.2	N_b is the basic concrete breakout strength in tension of single anchor in cracked concrete.	$N_b = k_c \sqrt{f'_c} h_{ef}^{1.5}$ (D-7)
	Assume the embedment will be less than 11 in., and use Eq. (D-7). For cast-in-place anchors, use $k_c = 24$.	$N_b = 24(4000)^{0.5} h_{ef}^{1.5}$ $= 1.52 h_{ef}^{1.5}$ kips
D.5.2	N_{cbg} is the nominal concrete breakout strength in tension of a group of anchors.	$N_{cbg} = (A_{Nc}/A_{Nco}) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} N_b$ $= 4 \times 1.0 \times 1.0 \times 1.0 \times 1.52 \times h_{ef}^{1.5}$ $= 6.08 \times h_{ef}^{1.5}$ (D-5)
	Calculate the minimum required effective embedment depth h_{ef} by setting N_{cbg} equal to $N_{cbg,req}$.	$6.08 \times (h_{ef})^{1.5} = 60.0$ kips $h_{ef,req} = 4.6$ in.

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 2 (cont.)		
D.5.2	Determine the total length L of the stud: The total required length L of the stud is equal to the required effective embedment depth h_{ef} plus the head thickness, plus allowance for burnoff (minus the plate thickness, which is conservatively ignored in this problem*). Typical values for head thickness and burnoff are provided in Table 6 of Appendix A .	$L_{req} = h_{ef,req} + \text{head thickness} + \text{burnoff}$ $= 4.6 + 0.312 + 0.125$ $= 5.04 \text{ in.}$ Use four 1/2 in. diameter x 5-1/4 in. long studs. $h_{ef,provided} = 5.25 - 0.312 - 0.125 \text{ (burnoff)} = 4.81 \text{ in.}$
D.5.2.1	Determine the spacing s: Assume no limits on spacing. Space anchors at three times h_{ef} .	Spacing required between anchors is: $3 \times 4.81 = 14.43 \text{ in.}$
	Determine the actual concrete breakout failure N_{cbg} using the actual embedment and spacing.	Use spacing $s = 15 \text{ in.}$
D.5.2	N_{cbg} is the nominal concrete breakout strength in tension in a group of anchors.	$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}N_b \quad (D-5)$
D.5.2.1	Determine A_{Nc} .	$A_{Nc} = (3h_{ef} + s)^2$ $= (3 \times 4.81 + 15)^2$ $= 866.1 \text{ in.}^2$
D.5.2.1	Determine A_{Nco} .	$A_{Nco} = 9 \times h_{ef}^2$ $= 9 \times (4.81)^2$ $= 208.2 \text{ in.}^2 \quad (D-6)$
D.5.2.1	The ratio A_{Nc}/A_{Nco} of is limited to 4.	$A_{Nc}/A_{Nco} = 866.1/208.2$ $= 4.16 > 4.0$ Use 4.0
	The embedment is less than 11 in. Therefore, Eq. (D-7) is used to calculate the basic concrete breakout strength. Also, because the embedment is less than 10 in., $\psi_{ed,N}$ is 1.0 as assumed previously.	$N_b = 1.52h_{ef}^{1.5}$ $= 1.52 \times (4.81)^{1.5}$ $= 16.03 \text{ kips}$ $N_{cbg} = 4 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 16.03$ $= 64.1 \text{ kips}$
	Check ductility. $0.85N_{cbg} > N_{sa}$	$0.85N_{cbg} = 0.85 \times 64.12$ $= 54.5 \text{ kips} > 51.0 \text{ kips } (N_{sa})$ OK
STEP 3: Check pullout strength of stud.		
D.5.3.1	Determine N_{pn}: N_{pn} is the nominal pullout strength in tension of a single anchor.	$N_{pn} = \psi_{c,p}N_p \quad (D-14)$
D.5.3.1	N_p is the pullout strength in tension of a single anchor in cracked concrete.	$N_p = 8A_{brg}f'_c \quad (D-15)$
D.5.3.5	Concrete is cracked per problem statement.	$\psi_{c,p} = 1.0$
D.5.3.4	Calculate the bearing area. The anchor head diameter is 1.0 in. for a 1/2 in. diameter stud (Table 6, Appendix A).	$A_{brg} = \pi \times (1.00^2 - 0.50^2)/4$ $= 0.59 \text{ in.}^2$ $N_p = 8 \times 0.59 \times 4$ $= 18.9 \text{ kips}$ $N_{pn} = \psi_{c,p}N_p$ $= 1.0 \times 18.9$ $= 18.9 \text{ kips/stud}$ $= 75.6 \text{ kips (4 studs)} \quad (D-14)$
D.3.6.1	Check ductility. $0.85N_{pn} > N_{sa}$	$0.85N_{pn} = 0.85 \times 75.6$ $= 64.3 \text{ kips} > 51.0 \text{ kips } (N_{sa})$

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 4: Check concrete side-face blowout.		
D.5.4.1	Check concrete side-face blowout. If $c_{a1} > 0.4h_{ef}$, side-face blowout will not be a factor. (Same is also true for c_{a2}).	$c_{a1} > 0.4h_{ef}$ $= 0.4 \times 4.81$ $= 1.92 \text{ in.}$ $c_{a1} = 15 \text{ in.} > 1.92 \text{ in.}$ OK
STEP 5: Summary.		
Given	Applied load	$N_{ua} = 28 \text{ kips}$
Step 1 D.4.5.a	Design steel tensile strength	$\phi N_{sa} = 0.8 \times 51.0 = 40.8 \text{ kips}$
Step 2 D.4.5.c	Design concrete breakout strength	$\phi N_{cbg} = 0.75 \times 64.1 = 48.1 \text{ kips}$
Step 3 D.4.5.c	Design concrete pullout strength	$\phi N_{pn} = 0.75 \times 75.6 = 56.7 \text{ kips}$
Step 4 D.4.5.c	Design concrete side-face blowout strength	$\phi N_{sb} = \text{N/A}$
D.4.1.2	Design strength of stud in tension	$\phi N_n = \min(\phi N_{sa}, \phi N_{cbg}, \phi N_{pn})$ $= \min(40.8, 48.1, 56.7)$ $= 40.8 \text{ kips} > N_{ua} = 28 \text{ kips}$ OK
D.3.6.1	Ductility:	$\min(0.85N_{cbg}, 0.85N_{pn}) > N_{sa}$ $\min(0.85 \times 64.1, 0.85 \times 75.6)$ $\min(54.5, 64.3)$ $= 54.5 \text{ kips} > N_{sa} = 51.0 \text{ kips}$ OK
STEP 6: Check plate thickness.		
AISC	Select plate thickness using the appropriate steel code. This step is not included for this example. A sample calculation for a base plate design is provided in Example B1(b) .	

*In the above example, the effective embedment h_{ef} is taken to the face of the concrete. If the plate was larger than the projected surface area, then the embedment length would exclude the thickness of the embedded plate.

Example B1(b)—Four-stud embedded plate, tension only, close spacing

Design an embedment with four welded studs and a rigid embedded plate for a 3 x 3 x 3/16 in. A501 structural tube attachment where anchors are spaced less than $3h_{ef}$ apart.

Given:

Concrete edges

$$c_{a,min} = 15 \text{ in.}$$

$$h_a = 18 \text{ in.}$$

Base plate

$$8 \times 8 \text{ in.}$$

Spacing

$$s = 6 \text{ in.}$$

Concrete

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

Stud material (A29/A108)*

$$f_{ya} = 51 \text{ ksi}$$

$$f_{uta} = 65 \text{ ksi}$$

Plate

$$F_y = 36 \text{ ksi}$$

Load

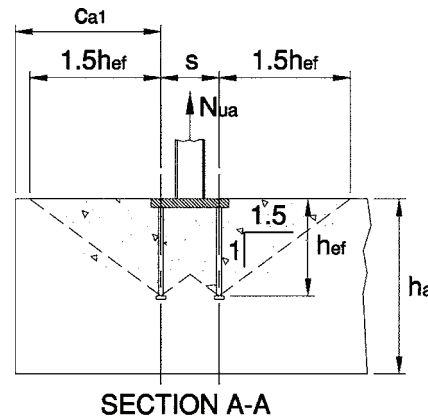
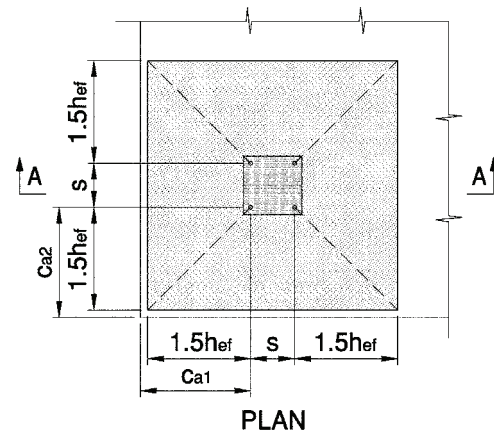
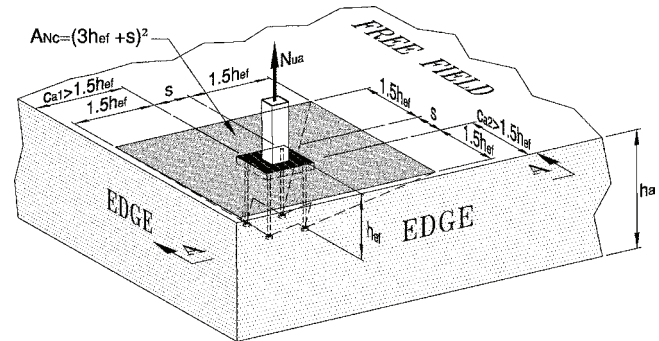
$$N_{ua} = 28 \text{ kips}$$

Where N_{ua} is the applied factored external load using load factors from Appendix C of the Code.

Assumptions:

- Concrete is cracked.
- ϕ -factors are based on Condition B in D.4.5 of the Code (no supplementary reinforcement).
- Ductile embedment design in accordance with D.3.6.1.

*Stud material is A29/A108, material properties per AWS D1.1, 2006, Table 7.1, Type B stud. Yield strength = 51 ksi; tensile strength = 65 ksi. It has elongation of 20% and reduction in area of 50%; meets the definition of a ductile steel element given in D.1, and meets the tensile strength requirements of D.5.1.2 and D.6.1.2: $f_{uta} \leq 1.9f_{ya}$ ($65 \leq 1.9 \times 51 = 96.9$ ksi).



CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 1: Determine the required stud diameter.		
D.4.1.1	Equate internal design strength ϕN_n to the external factored load N_{ua} .	$\phi N_n \geq N_{ua}$ Equation no. (D-1)
	In ductile design, the internal strength N_n is controlled by the steel strength of the stud, N_{sa} .	$N_n = N_{sa} = n A_{se} f_{uta}$
D.4.5	The required steel strength of the stud, $N_{sa,req}$, is multiplied by $\phi = 0.80$ for tension because the embedment and the steel stud is ductile and the load factors are based on Appendix C of the Code.	$0.80 N_{sa,req} \geq 28$ $N_{sa,req} = 28/0.8$ $= 35.0$ kips
D.5.1	Solve for the required steel area $A_{se,req}$ for a single stud.	$N_{sa,req} = n A_{se,req} f_{uta}$ $35.0 = 4 A_{se,req} \times 65$ $A_{se,req} = 35.0/(4 \times 65)$ $= 0.13 \text{ in.}^2$ (required)
	Anchor diameter, in. Effective area A_{se} , in. ² 3/8 0.110 1/2 0.196 controls (Refer to Appendix A, Table 6)	Use 1/2 in. diameter studs. $A_{se} = 0.196 \text{ in.}^2 > 0.13 \text{ in.}^2$ OK
D.5.1	Determine the nominal tensile strength N_{sa} of four 1/2 in. diameter studs.	$N_{sa} = n A_{se} f_{uta}$ $= 4 \times 0.196 \times 65$ $= 51.0$ kips (D-3)
STEP 2: Determine the minimum embedment length and spacing for the studs to prevent concrete breakout failure in tension.		
D.3.6.1	Ensure steel strength controls: To prevent concrete breakout failure in tension, the required design concrete breakout tensile strength has to be greater than the nominal tensile strength of the embedment steel, N_{sa} . The design concrete breakout strength shall be taken as 0.85 times the nominal strength.	From Step 1: $N_{sa} = 51.0$ kips $0.85 N_{cbg,req} > N_{sa}$ $N_{cbg,req} > 51/0.85$ $N_{cbg,req} > 60.0$ kips
D.5.2	N_{cbg} is the nominal concrete breakout strength in tension of a group of anchors.	$N_{cbg} = (A_{Nc}/A_{Nco}) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$ (D-5)
D.5.2.4	Modification factors are 1.0 for:	
D.5.2.5	Eccentricity effects $\psi_{ec,N}$	$\psi_{ec,N} = 1.0$
	Edge effects $\psi_{ed,N}$	$\psi_{ed,N} = 1.0$
	$c_{a,min} = 15$ in. per problem statement. Edge effect factor $\psi_{ed,N}$ will be 1.0 as long as $c_{a,min} \geq 1.5 h_{ef}$ or $h_{ef} \leq c_{a,min}/1.5$. Therefore, the embedment h_{ef} needs to be less than $15/1.5 = 10$ in. to ensure no reduction due to edge distance.	
D.5.2.6	Concrete cracking $\psi_{c,N}$. Concrete is cracked per problem statement.	$\psi_{c,N} = 1.0$
D.5.2.7	$\psi_{cp,N}$ is not used for cast-in-place anchors.	$\psi_{cp,N} = \text{N/A}$ for studs
D.5.2.2	N_b is the basic concrete breakout strength in tension of single anchor in cracked concrete.	$N_b = k_c \sqrt{f'_c} h_{ef}^{1.5}$ (D-7)
	Assume the embedment will be less than 11 in., and use Eq. (D-7). For cast-in-place anchors, use $k_c = 24$.	$N_b = 24(4000)^{0.5} h_{ef}^{1.5}$ $= 1.52 h_{ef}^{1.5}$ kips
D.5.2.1	N_{cbg} is the nominal concrete breakout strength in tension of a group of anchors.	$N_{cbg} = (A_{Nc}/A_{Nco}) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} N_b$ $= (A_{Nc}/A_{Nco}) \times 1.0 \times 1.0 \times 1.0 \times 1.52 \times h_{ef}^{1.5}$ $= (A_{Nc}/A_{Nco}) \times 1.52 \times (h_{ef})^{1.5}$ (D-5)
	Use trial and error; increase h_{ef} until N_{cbg} is equal or greater than the required $N_{cbg,req}$	
	First iteration: Try $h_{ef} = 8$ in.	