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(Reapproved 2014)

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

Reported by ACI Committee 349

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Reported by ACI Committee 349

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

$\psi_{ed,N}$ = 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.

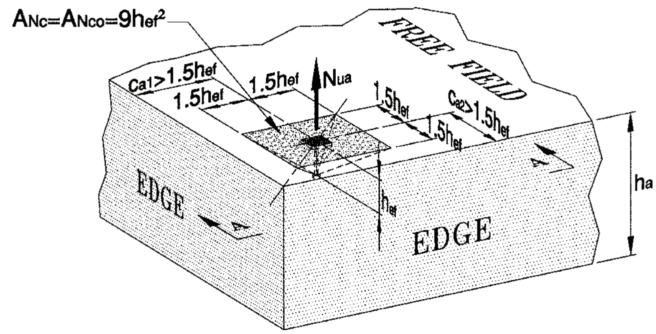
When the Engineer is faced with base plate and anchorage configuration differing from those used in these design examples, the Engineer must apply the Code requirements and use rational assumptions appropriate for these other design configurations.

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

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

$h_a = 18$ in.

Concrete

$f_c' = 4000$ psi

Stud material (A29/A108)*

$f_{ya} = 51$ ksi

$f_{uta} = 65$ ksi

Plate

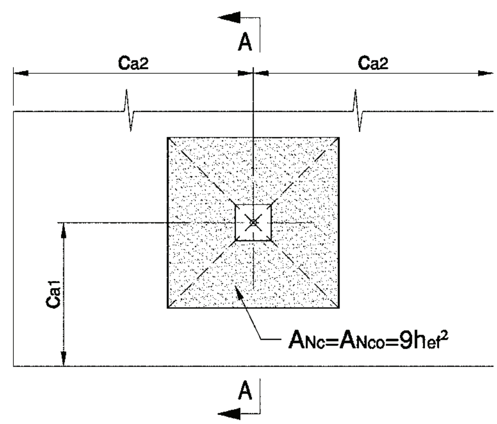
3 x 3 x 3/8 in. thick

$F_y = 36$ ksi

Loads

$N_{ua} = 8$ kips

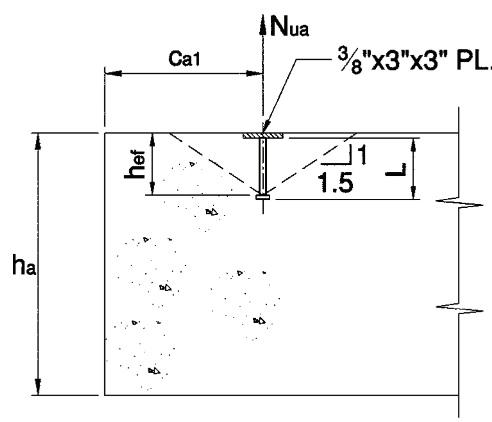
Where N_{ua} is the applied factored external load using load factors from Appendix C of the Code.



PLAN

Assumptions:

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



SECTION A-A

*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: $f_{uta} \leq 1.9 f_{ya}$ ($65 \leq 1.9 \times 51 = 96.9 < 125$ ksi).

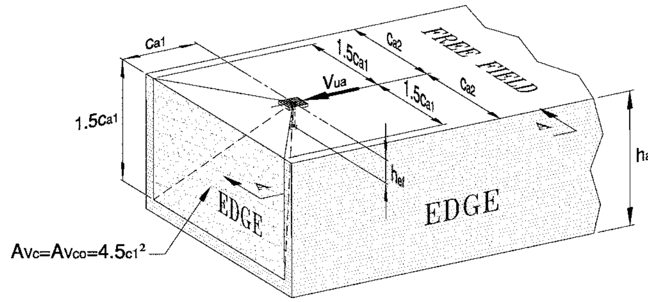
CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 1: Determine required steel area and diameter of the stud.		
		Equation no.
D.4.1.1 D.5.1.2	Equate the external factored load to the internal design strength and solve for the required steel area of the stud.	$\phi N_n \geq N_{ua}$ (D-1) $N_n = N_{sa} = nA_{se} f_{uta}$ (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, ϕ = 0.80 for tension.	$N_{ua} = 8 = \phi nA_{se} f_{uta}$ $= 0.80 \times 1.0 \times A_{se} \times 65$ kips $A_{se,req} = 0.154$ in.^{2} required
	Calculate nominal steel strength of selected stud. (Appendix A, Table 6) Anchor diameter, in. — Effective area A_{se}, in.^{2} 3/8 — 0.110 1/2 — 0.196 **controls**	**Use one 1/2 in. diameter stud** A_{se} = 0.196 in.^{2} > 0.154 in.^{2}
D.5.1.2	Calculate the nominal steel strength N_{sa}.	$N_{sa} = nA_{se} f_{uta}$ (D-3) $= 1.0 \times 0.196 \times 65$ $= 12.74$ kips
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.)	$f_{uta} = 65 \leq 1.9 f_{ya} = 1.9 \times 51 = 96.9$ ksi $\leq 125{,}000$ 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 kips $0.85N_{cb,req} = N_{sa}$ $N_{cb,req} = N_{sa}/0.85$ $= 12.74/0.85$ $= 14.99$ kips
D.5.2.1	Concrete breakout strength for a single stud:	$N_{cb} = (A_{Nc}/A_{Nco})\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$ lb (D-4)
	For a single stud away from edge:	$A_{Nc} = A_{Nco} = 9h_{ef}^2$ (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) $\psi_{c,N} = 1.0$ $\psi_{cp,N}$ = N/A for studs
D.5.2.2	k_c = 24 for cast-in headed stud Assume h_{ef} < 11 in.	$N_b = k_c\sqrt{f'_c}\,h_{ef}^{1.5}$ lb (D-7) $= 24\sqrt{4000}\,h_{ef}^{1.5}$ $= 1518h_{ef}^{1.5}$ lb $= 1.52h_{ef}^{1.5}$ kips
	Determine required embedment length $h_{ef,req}$.	$N_{cb,req} = 14.99$ kips $= 1.0 \times 1.0 \times 1.0 \times 1.52h_{ef}^{1.5}$ $h_{ef,req} = 4.60$ 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 – burnoff (0.125 in.) = 4.69 in. > 4.60 in. OK
	Calculate N_{cb} using $h_{ef,provided}$.	$N_{cb} = 1.52 \times 4.69^{1.5}$ $= 15.44$ kips $\geq N_{cb,req} = 14.99$ 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.	$N_p = 8A_{brg} f_c'$ (D-15) $= 8A_{brg} \times 4$ $= 32A_{brg}$ kips
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*.	$A_{brg} = n \times (1.0^2 - 0.5^2)/4$ $= 0.59$ in.^{2} $N_{pn} = 1.0 \times 32 \times 0.59$ $= 18.88$ kips
D.3.6.1	Design embedment as ductile, in accordance with D.3.6.1: $0.85N_{pn} \geq N_{sa}$	$0.85N_{pn} = 0.85 \times 18.88$ $= 16.05$ kips $> N_{sa} = 12.74$ 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	$N_{ua} = 8$ kips
Step 1 D.4.5.a	Design steel tensile strength	$\phi N_{sa} = 0.8 \times 12.74 = 10.19$ kips
Step 2 D.4.5.c	Design concrete breakout strength	$\phi N_{cb} = 0.75 \times 15.44 = 11.58$ kips
Step 3 D.4.5.c	Design concrete pullout strength	$\phi N_{pn} = 0.75 \times 18.88 = 14.16$ kips
Step 4 D.4.5.c	Design concrete side-face blowout strength	ϕN_{sb} = N/A
D.4.1.2	Design strength of stud in tension	$\phi N_n = \min(\phi N_{sa}, \phi N_{cb}, \phi N_{pn})$ $= \min(10.19, 11.58, 14.16)$ $= 10.19$ kips $> N_{ua} = 8$ kips OK
D.3.6.1	Ductility	$\min(0.85N_{cb}, 0.85N_{pn}) > N_{sa}$ $\min(0.85 \times 15.44, 0.85 \times 18.88)$ $= 13.12 > 12.74$ 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 in.

c_{a2} = 18 in.

h_a = 18 in.

Concrete

f_c' = 4000 psi

Stud material (A29/A108)[*]

f_{ya} = 51 ksi

f_{uta} = 65 ksi

Plate

Assume 3 x 3 x 3/8 in. thick

F_y = 36 ksi

Loads

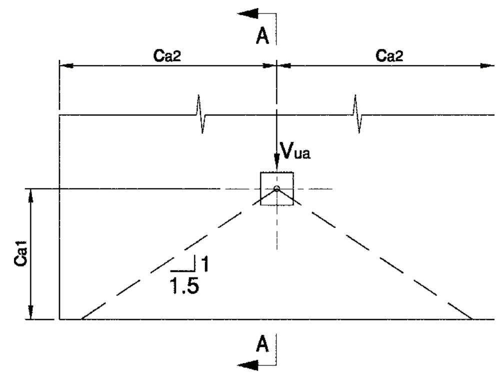
V_{ua} = 6 kips

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

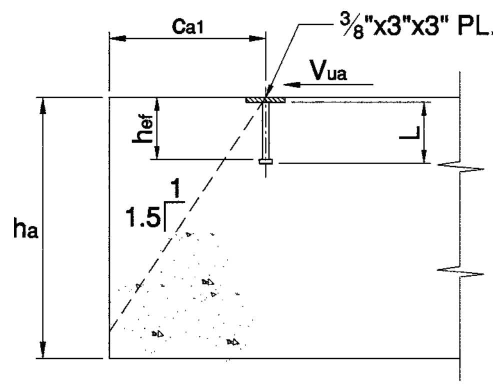
Assumptions:

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

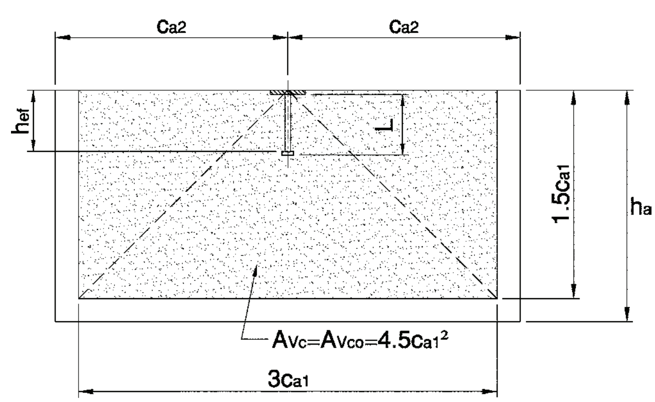
[*]Stud material is A29/A108, material properties per AWS D1.1, 2006, Table 7.1, Type B stud. Yield strength = 51 ksi; tensile strength = 65 ksi. It has elongation of 20% and reduction in area of 50%; meets the definition of a ductile steel element given in D.1, and meets the tensile strength requirements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



PLAN



SECTION A-A



FRONT ELEVATION

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 1: Determine required steel area of the stud.		
		Equation no.
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.	$\phi V_n \geq V_{ua}$ (D-2) $V_n = V_{sa} = nA_{se}f_{uta}$ (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)	$A_{se,req} = 6/(0.75 \times 1 \times 65)$ $= 0.123$ in.^{2} required
	Anchor diameter, in. — Effective area A_{se}, in.^{2} 3/8 — 0.110 1/2 — 0.196 — **controls**	**Use one 1/2 in. diameter stud** $A_{se} = 0.196$ in.$^2 > 0.123$ in.^{2} OK $V_{sa} = nA_{se}f_{uta}$ (D-18) $= 1.0 \times 0.196 \times 65$ $= 12.74$ kips
D.6.1.2	Calculate the nominal shear capacity of the selected stud. 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.)	$f_{uta} = 65 \leq 1.9f_{ya} = 1.9 \times 51 = 96.9$ ksi ≤ 125 ksi OK
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: $0.85V_{cb} \geq A_{se}f_{uta}$	$0.85V_{cb} \geq A_{se}f_{uta}$ min $V_{cb} = A_{se}f_{uta}/0.85$ $= 0.196 \times 65/0.85$ $= 14.99$ 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.	$A_{Vco} = 4.5{c_{a1}}^2$ (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) $\ell_e \leq 8d_o$ Assume $\ell_e = 2.5$ in. $(\ell_e/d_o) = 2.5/0.5 = 5.0$
	See definition in D.6.2.2 for limits on ℓ_e.	$V_b = 8 \times 5^{0.2} \times 0.5^{0.5} \times 4000^{0.5} \times {c_{a1}}^{1.5}$ $= 494{c_{a1}}^{1.5}$ lb $= 0.494{c_{a1}}^{1.5}$ 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.	$V_{cb,req} = 14.99$ kips $= 1.0 \times 1.0 \times 1.0 \times 0.494{c_{a1}}^{1.5}$ $= 0.494 \times {c_{a1}}^{1.5}$ kips $c_{a1,req} = 9.73$ in. (required) < 10 in. (provided) OK Strength controlled by steel
	Calculate V_{cb} using $c_{a1} = 10$ in. provided.	$V_{cb} = 0.494 \times 10^{1.5} = 15.62$ > 12.74 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 (D-18) See Step 1 (Note: same value as for tension N_{sa})
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.	$V_{cp} = k_{cp}N_{cb}$ (D-28) $k_{cp} = 2.0$ $N_{cb,req} = 14.99/2.0$ $= 7.50$ 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. See Appendix A, Table 6, for stud head dimensions. Note that 0.312 in. is head thickness and 0.125 in. is burnoff. Calculate V_{cp} using $h_{ef} = 3.06$ in.	$N_b = k_c\sqrt{f_c'}\ h_{ef}^{1.5}$ (D-7) $= 2400\sqrt{4000}\ h_{ef}^{1.5}$ $= 1.52h_{ef}^{1.5}$ kips $7.50 = 1.0 \times 1.0 \times 1.0 \times 1.52h_{ef}^{1.5}$ $h_{ef,req} = 2.90$ in. (required) **Use 1/2 x 3-1/2 in. long stud.** h_{ef} provided: $h_{ef} = 3.5 - 0.312 - 0.125$ $= 3.06$ in. > 2.90 OK $V_{cp} = k_{cp}N_{cb}$ $= 2.0 \times 1.52 \times 3.06^{1.5}$ $= 16.27 > 12.74$ 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 D.5.3.4	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) $N_p = 8A_{brg}f_c'$ (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	ϕV_{sa} = 0.75 × 12.74 = 9.56 kips	OK
Step 2 D.4.5.c	Design concrete breakout strength	ϕV_{cb} = 0.75 × 15.62 = 11.72 kips	OK
Step 3 D.4.5.c	Design concrete pryout strength	ϕV_{cp} = 0.75 × 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 × 15.62, 0.85 × 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 in., 0.5/2 = 0.25 in.) = 3/8 in. min t > 0.5/2.7 = 0.19 in < 3/8 in. 3/8 in. thick plate is OK	

*Goble, G. G., 1968, “Shear Strength of Thin Flange Composite Sections,” *AISC Engineering Journal*, 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 in.
c_{a2} = 20 in.
h_a = 18 in.

Concrete

f_c' = 4000 psi

Stud material (A29/A108)*

f_{ya} = 51 ksi
f_{uta} = 65 ksi

Plate

3 x 3 x 3/8 in. thick
F_y = 36 ksi

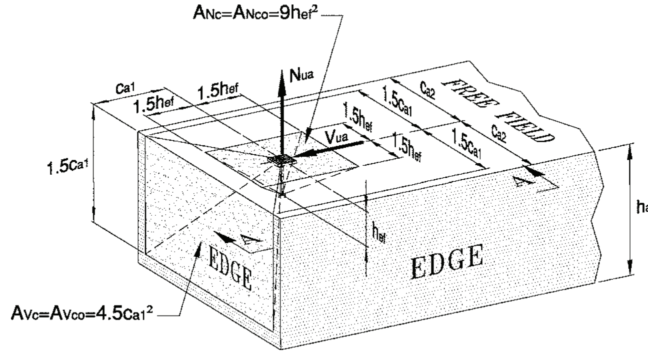
Loads

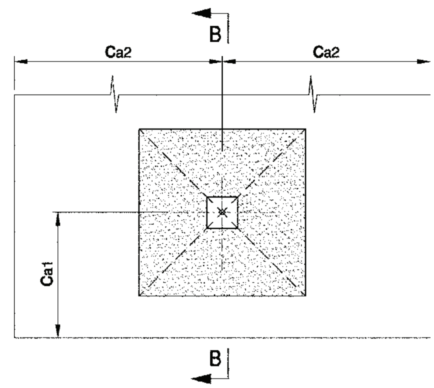
N_{ua} = 8 kips
V_{ua} = 6 *kips*

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

Assumptions:

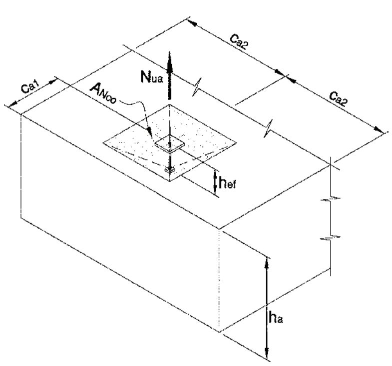
- Concrete is cracked.
- ϕ-factors are based on Condition B in D.4.5 of the Code (no supplementary reinforcement).

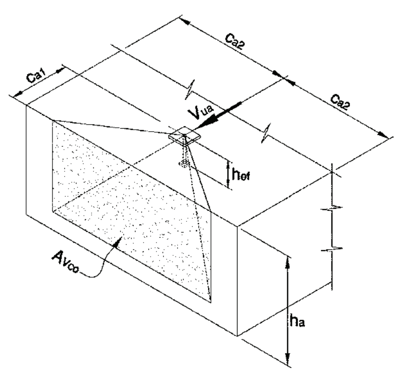


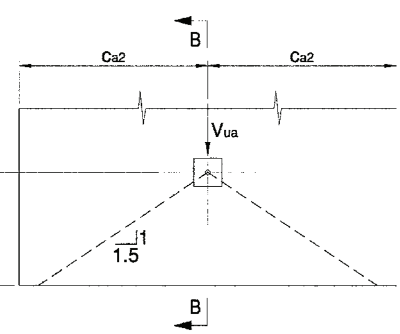


PLAN (TENSION ONLY)

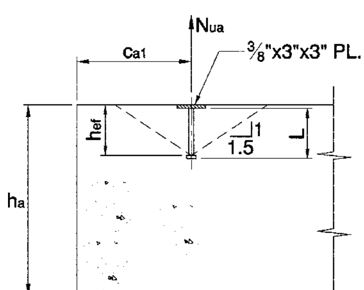
*Stud material is A29/A108, material properties per AWS D1.1, 2006, Table 7.1, Type B stud. Yield strength = 51 ksi; tensile strength = 65 ksi. It has elongation of 20% and reduction in area of 50%; meets the definition of a ductile steel element given in D.1, and meets the tensile strength requirements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



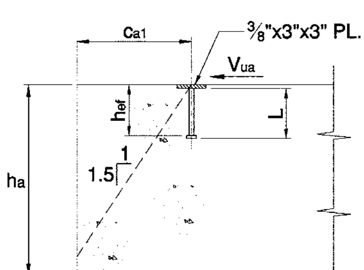




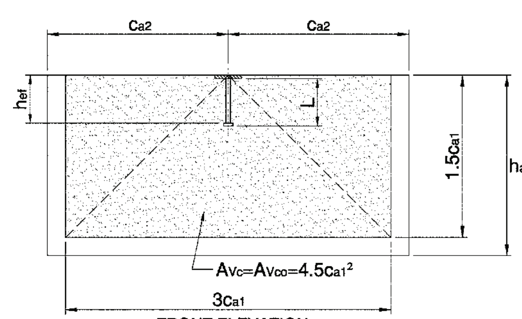
PLAN (SHEAR ONLY)



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.		
		Equation no.
D.4.1.1	Equate the external factored load to the internal design strength and solve for the required steel area of the stud.	$\phi N_n \geq N_{ua}$ (D-1) $N_n = N_{sa} = nA_{se,t}f_{uta}$ (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.	$A_{se,t} = N_{ua}/(\phi n f_{uta})$ $= (8/0.80 \times 1.0 \times 65)$ $A_{se,t} = 0.154$ 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, ϕ = 0.80 for tension and 0.75 for shear. Steel material is ductile. (See asterisked note on previous page.)	
D.6.1	Use the shear provisions of D.6.1 to determine the required steel area for shear load.	$\phi V_n \geq V_{ua}$ (D-2) $V_n = V_{sa} = nA_{se,v}f_{uta}$ (D-17)
	$A_{se,v}$ = required steel area for shear.	$A_{se,v} = V_{ua}/(\phi n f_{uta})$ $= 6/(0.75 \times 1.0 \times 65)$ $= 0.123$ in.^{2}
	Add the area of steel required for tension to the area of steel required for shear.	
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.	$A_{se,req} = (0.154 + 0.123)/1.2 = 0.231$ in.^{2}
	<u>Anchor diameter, in.</u> <u>Effective area A_{se}, in.^{2}</u> 1/2 0.196 5/8 0.307 **controls** (Refer to Appendix A, Table 6)	
		Use one 5/8 in. diameter stud.
		$A_{se} = 0.307$ in.^{2} > 0.231 in.^{2} OK
	Calculate the nominal steel strength N_{sa}.	$N_{sa} = nA_{se}f_{uta}$ $= 1.0 \times 0.307 \times 65$ $= 19.96$ kips
	Calculate the nominal steel strength V_{sa}.	$V_{sa} = nA_{se}f_{uta}$ $= 1.0 \times 0.307 \times 65$ $= 19.96$ kips
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: $N_{sa} = 19.96$ 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 $0.85N_{cb} \geq N_{sa}$	$0.85N_{cb,req} \geq N_{sa}$ $N_{cb,req} = N_{sa}/0.85$ $= 19.96/0.85$ $= 23.48$ kips
D.5.2.1	Calculate concrete breakout strength for a single anchor.	$N_{cb} = (A_{Nc}/A_{Nco})\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$ lb (D-4)
	For a single stud away from edge:	$A_{Nc}/A_{Nco} = 1.0$
	Modification factors for:	
D.5.2.5 D.5.2.6 D.5.2.7	Edge effects $\psi_{ed,N}$ Concrete cracking $\psi_{c,N}$ $\psi_{cp,N}$ applies to post-installed anchors only	$\psi_{ed,N} = 1.0$ $\psi_{c,N} = 1.0$ $\psi_{cp,N}$ = 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.	$N_b = k_c\sqrt{f_c'}\,h_{ef}^{1.5}$ lb (D-7) $= 24\sqrt{(4000)}\,h_{ef}^{1.5}$ $= 1.52h_{ef}^{1.5}$ kips
	Determine required embedment length $h_{ef,req}$.	23.48 $= 1.0 \times 1.0 \times 1.0 \times 1.52h_{ef}^{1.5}$ $h_{ef,req}$ = 6.20 in. (required) **Use 5/8 x 6-3/4 in. long stud** $h_{ef,provided}$ = 6.75 – 0.312 – 0.187 + 0.375 in. = 6.63 in. > 6.20 in. OK
	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.	c_{a1} = 12 in. > $1.5h_{ef}$ = 1.5 × 6.63 = 9.94 in. Edge distance has no effect.
	Calculate N_{cb} using $h_{ef,provided}$.	$N_{cb} = 1.52 \times 6.63^{1.5}$ = 25.95 kips
STEP 3: Check pullout strength of stud.		
	Stud head is required to develop the concrete breakout strength N_{cb}.	
D.5.3	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.	$N_{pn} = \psi_{c,P}N_P$ (D-14) $N_p = 8A_{brg}f_c'$ (D-15) $= 8 \times A_{brg} \times 4$ $= 32A_{brg}$ 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 in.^{2} Table 6 N_{pn} = 1.0 × 32 × 0.92 = 29.44 kips
D.3.6.1	Design embedment as ductile, in accordance with D.3.6.1: $0.85N_{pn} \geq N_{sa}$ N_{sa} for this problem is calculated in Step 1.	$0.85N_{pn}$ = 0.85 × 29.44 = 25.02 kips > N_{sa} = 19.96 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 $c_{a,min} > 0.4h_{ef}$. In this example: h_{ef} = 6.63 in. $0.4h_{ef}$ = 0.4 × 6.63 = 2.6 in. $c_{a,min}$ = 12 in. > 2.6 in. Because $c_{a,min} > 0.4h_{ef}$, side-face blowout calculation is not required. The calculation will be done to illustrate the method.	$N_{sb} = 160c\sqrt{A_{brg}}\sqrt{f_c'}$ (D-16) $c_{a,min}$ = 12 in. A_{brg} = 0.92 in.^{2} Table 6 f_c' = 4000 psi $N_{sb} = 160 \times 12 \times 0.92^{0.5} \times 4000^{0.5}$ = 116.5 kips $0.85N_{sb}$ = 99.0 kips > N_{sa} = 19.96 kips 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: $0.85V_{cb} > A_{se} f_{uta}$	$0.85V_{cb} > A_{se} f_{uta}$ $V_{cb,req} = A_{se} f_{uta}/0.85$ $= 0.307 \times 65/0.85$ $= 23.48$ 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 $\ell_e/d_o = 8.0$ $V_b = 8 \times 8^{0.2} \times 0.625^{0.5} \times 4000^{0.5} \times c_{a1}^{1.5}$ $= 606c_{a1}^{1.5}$ lb $= 0.606c_{a1}^{1.5}$ 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.	$23.48 = 1.0 \times 1.0 \times 1.0 \times 0.606c_{a1,req}^{1.5}$ $c_{a1,req} = 11.45$ in. < 12.0 in. OK
Step 1	Steel strength in shear V_{sa}. Calculate V_{cb} using $c_{a1} = 12$ in. provided.	Strength controlled by steel $V_{sa} = 19.96$ kips $V_{cb} = 0.606c_{a1}^{1.5}$ $= 0.606(12)^{1.5}$ $= 25.19$ 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: $0.85V_{cp} > V_{sa}$	$0.85V_{cb} > V_{sa}$ $V_{cb,req} = V_{sa}/0.85$ $= 19.96/0.85$ $= 23.5$ kips
D.6.3	N_{cb} is calculated in Step 2	$V_{cp} = k_{cp}N_{cb}$ (D-28) $k_{cp} = 2.0$ $N_{cb} = 25.95$ kips $V_{cp} = 2 \times 25.95$ $= 51.9$ kips >> 23.5 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 OK
	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 OK
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 OK
	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 OK
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	$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2$	$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} = 0.50 + 0.40 = 0.90$ (D-3) $0.90 < 1.2$ OK
STEP 9: Calculate minimum plate thickness		
D.6.2.3	Select plate thickness equal to or grater 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 in., 0.5/2 = 0.25 in.) = 3/8 in. $t_{req} > 0.5/2.7 = 0.19$ in. $<$ 3/8 in. **3/8 in. thick plate** OK

*Goble, G. G., 1968, "Shear Strength of Thin Flange Composite Sections," *AISC Engineering Journal*, 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 in.
c_{a2} = 24 in.
h_a = 36 in.

Concrete

f_c' = 4000 psi

Bolt material (AB105)*

f_{ya} = 105 ksi
f_{uta} = 125 ksi

Plate (face mounted)

3/8 in. thick
F_y = 36 ksi

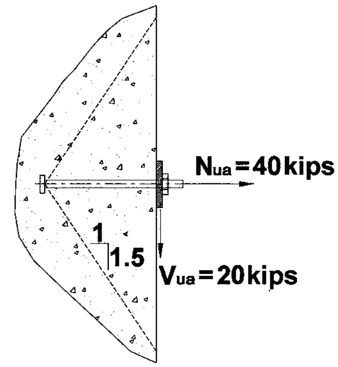
Loads

N_{ua} = 40 kips
V_{ua} = 20 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).



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

*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 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.

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 1: Determine required steel area of the bolt.		
		Equation no.
D.4.1.1	Equate the external factored load to the internal design strength and solve for the required steel area of the bolt.	$\phi N_n \geq N_{ua}$ (D-1) $N_n = N_{sa} = nA_{se,t} f_{uta}$ (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.	$A_{se,t} = N_{ua}/(\phi n f_{uta})$ $= (40/0.80 \times 1.0 \times 125) = 0.40$ 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, ϕ = 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.	$\phi V_n \geq V_{ua}$ (D-2) $V_n = V_{sa} = n0.6A_{se,v} f_{uta}$ (D-18)
	$A_{se,v}$ = required steel area for shear.	$A_{se,v} = V_{ua}/(n0.6\phi f_{uta})$ $= 20/(1.0 \times 0.6 \times 0.75 \times 125)$ $= 0.36$ in.^{2}
	Add the area of steel required for tension to the area of steel required for shear.	
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.	$A_{se,req} = (0.40 + 0.36)/1.2 = 0.63$ in.^{2}
	Anchor diameter, in. — Effective area A_{se}, in.^{2} 1.0 — 0.606 1.125 — 0.763 **controls**	**Use one 1-1/8 in. diameter headed bolt.** $A_{se} = 0.76$ in.^{2} > 0.64 in.^{2} OK
	Calculate the nominal steel strength N_{sa} in tension.	$N_{sa} = nA_{se} f_{uta}$ $= 1.0 \times 0.76 \times 125$ $= 95.0$ kips
	Calculate the nominal steel strength V_{sa} in shear.	$V_{sa} = n0.6A_{se} f_{uta}$ $= 1.0 \times 0.6 \times 0.76 \times 125$ $= 57.0$ 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: $N_{sa} = 95.0$ kips
D.3.6.1	The requirements for ductile design are given in D.3.6.1. For tension load, this requires that	
	$0.85N_{cb} \geq N_{sa}$	$0.85N_{cb,req} \geq N_{sa}$ $N_{cb,req} = N_{sa}/0.85$ $= 95/0.85$ $= 111.8$ kips
D.5.2.1	Calculate concrete breakout strength for a single bolt.	$N_{cb} = (A_{Nc}/A_{Nco})\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$ (D-4)
	For a single bolt away from edge:	$A_{Nc}/A_{Nco} = 1.0$

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	$\psi_{ed,N}$ = 1.0 $\psi_{c,N}$ = 1.0 $\psi_{cp,N}$ = N/A for bolts
	Basic concrete breakout strength: k_c = 24 for cast-in-place anchors.	$N_b = k_c\sqrt{f_c'}\,h_{ef}^{1.5}$ (D-7) $= 24\sqrt{(4000)}\,h_{ef}^{1.5}$ $= 1.52h_{ef}^{1.5}$ kips
		$N_{cb} = (A_{Nc}/A_{Nco})\psi_{ed,N}\psi_{c,N}N_b$ (D-4)
	Determine the required effective embedment depth $h_{ef,req}$.	$111.8 = 1.0 \times 1.0 \times 1.0 \times 1.52h_{ef}^{1.5}$ $h_{ef,req}$ = 17.57 in.
	Determine bolt length L.	L = 17.57 + 0.75 (thickness of head) = 18.32 in. **Use 18.5 in.** **Use 1-1/8 in. diameter, F 1554 Gr. 105 bolt embedded 18.5 in. into the concrete.**
	Calculate N_{cb} using $h_{ef,provided}$. Note: for 11 in. $\leq h_{ef} \geq$ 25 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.	$h_{ef,provided}$ = 18.5 – 0.75 = 17.75 in. > 17.57 in. OK $N_{cb} = 1.52 \times 17.75^{1.5}$ = 113.7 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.	$N_{pn} = \psi_{c,P}N_p$ (D-14)
	Calculate pullout strength.	$N_p = 8A_{brg}f_c'$ (D-15) $= 8 \times A_{brg} \times 4$ $= 32A_{brg}$
D.5.3.5	Concrete is cracked per problem statement. $\psi_{c,P}$ = 1.0 for cracked concrete.	$\psi_{c,P}$ = 1.0
	ASTM F 1554 recommends a heavy hex head and washer.	
	Appendix A, Table 4(c), A_{brg} = 1.85 in.^{2}	
	For this combination of steel strength and concrete strength, the anchor head alone is not sufficient to develop the required bearing strength.	$N_{pn} = 1.0 \times 32 \times 1.85$ = 59.2 kips $< N_{sa}$ = 95 kips (No good)
D.3.6.1	Find required net bearing area. Design embedment as ductile, in accordance with D.3.6.1: $0.85N_{pn} \geq N_{sa}$	$0.85N_{pn} = N_{sa}$ = 95 kips (N_{sa} from Step 1) $N_{pn,req}$ = 95/0.85 = 111.8 kips $111.8 = 32A_{brg}$
	See Appendix A, Table 4, for anchor head areas.	$A_{brg,req}$ = 111.8/32 = 3.49 in.^{2}
	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.	A_D = 0.99 in.^{2} anchor area $A_{total,req}$ = 3.49 + 0.99 = 4.48 in.^{2} For 1-1/8 in. bolt, hex head area is: A_H = 2.85 in.^{2} a_{req} = 4.48 – 2.85 = 1.63 in.^{2} Use a square plate 2-1/4 in. each side. $A_{washer} = 2.25^2$ = 5.06 in.^{2} > 4.48 in.^{2} OK

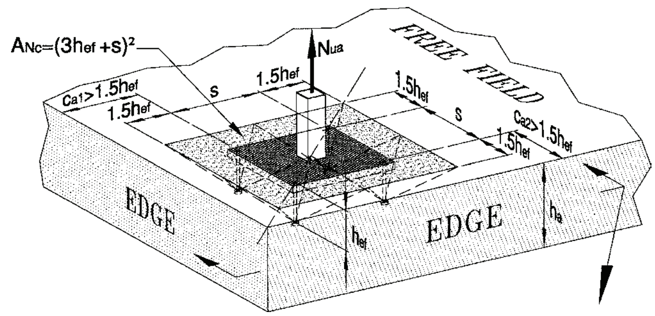
CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 3 (cont.)		
	Calculate N_{pn} using $A_{brg,provided}$.	$A_{brg,provided}$ = 5.06 – 0.99 = 4.07 in.^{2} > 3.49 in.^{2} OK
D.3.6.1	Check ductility: $0.85N_{pn} > N_{sa}$.	N_{pn} = 1.0 × 32 × 4.07 = 130.24 kips
	According to D.5.2.8, when adding a washer at 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.	$0.85N_{np}$ = 0.85 × 130.24 = 110.7 kips > N_{sa} = 95 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 × 0.6 × 0.76 × 215 = 57 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 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.	$V_b = 7(\ell_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f_c'}\,c_{a1}^{1.5}$ (D-23)
	See definition in D.6.2.2 for limits on ℓ_e.	ℓ_e/d_o = 17.75/1.125 = 15.78 > 8.0 Use ℓ_e/d_o = 8.0 $V_b = 7 \times 8^{0.2} \times 1.125^{0.5} \times 4000^{0.5} \times c_{a1}^{1.5}$ $= 712c_{a1}^{1.5}$ lb $= 0.712c_{a1}^{1.5}$ kips
D.6.2.6 D.6.2.7	Modification factors 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.	$67.06 = 1.0 \times 1.0 \times 1.0 \times 0.712c_{a1,req}^{1.5}$ $c_{a1,req}$ = 20.72 in. < 24.0 in. OK Strength controlled by steel
	Calculate V_{cb} using c_{a1} = 24 in. provided.	$V_{cb} = 0.712c_{a1}^{1.5}$ $= 0.712(24)^{1.5}$ = 83.7 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 kips V_{cp} = 2 × 113.7 = 227.4 kips >> 95 kips

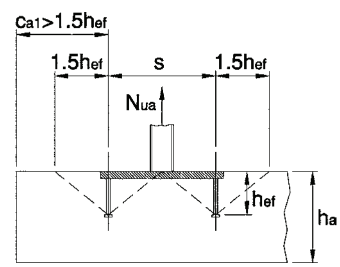
CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 7: Summary.		
D.4.1.2	TENSION	
	Applied load	N_{ua} = 40 kips
Step 1 D.4.5.a	Steel strength	$\phi N_{sa} = 0.8 \times 95.0 = 76.0$ kips
Step 2 D.4.5.c	Concrete breakout strength	$\phi N_{cb} = 0.75 \times 113.7 = 85.3$ kips
Step 3 D.4.5.c	Concrete pullout 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	$\phi N_n = \min(\phi N_{sa}, \phi N_{cb}, \phi N_{pn})$ $= \min(76.0, 85.3, 97.68)$ $= 76.0 \text{ kips} > N_{ua} = 40$ kips OK
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	$\phi V_n = \min(\phi V_{sa}, \phi V_{cb}, \phi V_{cp})$ $= \min(42.75, 62.78, 170.55)$ $= 42.75 \text{ kips} > V_{ua} = 20$ kips OK
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 OK
	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 OK
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.	$V_{ua}/\phi V_n = 20/42.75$ $= 0.47 > 0.2$
D.7.2	$N_{ua}/\phi N_n > 0.2$ Full strength in shear shall not be permitted.	$N_{ua}/\phi N_n = 40/76.0$ $= 0.53 > 0.2$ $\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2$ (D-30) $0.53 + 0.47 = 1.00 < 1.2$ OK
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

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

$h_a = 18$ in.

Concrete

$f_c' = 4000$ psi

Stud material (A29/A108)*

$f_{ya} = 51$ ksi

$f_{uta} = 65$ ksi

Plate

$F_y = 36$ ksi

Load

$N_{ua} = 28$ 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.

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*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.		
		Equation no.
D.4.1.1	Equate internal design strength ϕN_n to the external factored load N_{ua}.	$\phi N_n > N_{ua}$ (D-1)
	In ductile design, the internal strength N_n is controlled by the steel strength of the stud, N_{sa}.	$N_n = N_{sa} = nA_{se}f_{uta}$
D.4.5	The required steel strength of the stud, $N_{sa,req}$, is multiplied by ϕ = 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.	$0.80N_{sa,req} \geq 28$ $N_{sa,req} = 28/0.8$ $= 35.0$ kips
D.5.1	Solve for the required steel area $A_{se,req}$ for a single stud. Anchor diameter, in. — Effective area A_{se}, in.^{2} 3/8 — 0.110 1/2 — 0.196 — **controls** (Refer to Appendix A, Table 6)	$N_{sa,req} = nA_{se,req}f_{uta}$ (D-3) $35.0 = 4 \times A_{se,req} \times 65$ $A_{se,req} = 35.0/(4 \times 65)$ $= 0.13$ in.^{2} (required) **Use 1/2 in. diameter studs.** $A_{se} = 0.196$ in.$^2 > 0.13$ in.^{2} OK
D.5.1	Determine the nominal tensile strength N_{sa} of four 1/2 in. diameter studs	$N_{sa} = nA_{se}f_{uta}$ (D-3) $= 4 \times 0.196 \times 65$ $= 51.0$ kips
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}.	From Step 1: $N_{sa} = 51.0$ kips
	The design concrete breakout strength shall be taken as 0.85 times the nominal strength.	$0.85N_{cbg,req} > N_{sa}$ $N_{cbg,req} > 51/0.85$ $N_{cbg,req} > 60.0$ 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.	$A_{Nc} = 4 \times A_{Nco}$ $A_{Nc}/A_{Nco} = 4$
	Modification factors are 1.0 for:	
D.5.2.4	Eccentricity effects $\psi_{ec,N}$	$\psi_{ec,N} = 1.0$
D.5.2.5	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 $c_{a,min} \geq 1.5h_{ef}$ 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.	$\psi_{ed,N} = 1.0$
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.	$\psi_{cp,N}$ = 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 $k_c = 24$.	$N_b = 24(4000)^{0.5}h_{ef}^{1.5}$ $= 1.52h_{ef}^{1.5}$ kips
D.5.2	N_{cbg} is the nominal concrete breakout strength in tension of a group of anchors.	$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}N_b$ (D-5) $= 4 \times 1.0 \times 1.0 \times 1.0 \times 1.52 \times h_{ef}^{1.5}$ $= 6.08 \times h_{ef}^{1.5}$
	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.

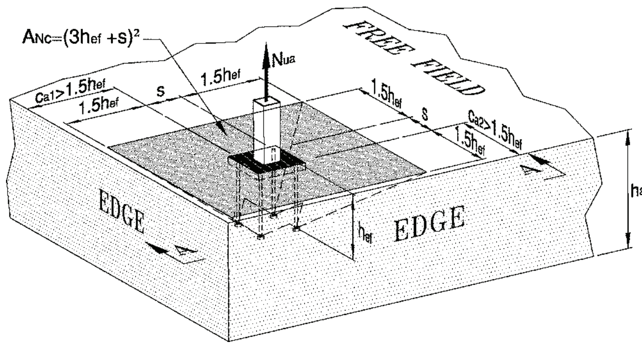
CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 2 (cont.)		
D.5.2	**Determine the total length L of the stud:** The total required length L of the stud is equal to the required effective embedment depth h_{ef} plus the head thickness, plus allowance for burnoff (minus the plate thickness, which is conservatively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

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}).	$c_{a1} > 0.4h_{ef}$ $= 0.4 \times 4.81$ $= 1.92$ in. $c_{a1} = 15$ in. > 1.92 in. OK
STEP 5: Summary.		
Given	Applied load	$N_{ua} = 28$ kips
Step 1 D.4.5.a	Design steel tensile strength	$\phi N_{sa} = 0.8 \times 51.0 = 40.8$ kips
Step 2 D.4.5.c	Design concrete breakout strength	$\phi N_{cbg} = 0.75 \times 64.1 = 48.1$ kips
Step 3 D.4.5.c	Design concrete pullout strength	$\phi N_{pn} = 0.75 \times 75.6 = 56.7$ kips
Step 4 D.4.5.c	Design concrete side-face blowout strength	ϕN_{sb} = N/A
D.4.1.2	Design strength of stud in tension	$\phi N_n = \min(\phi N_{sa}, \phi N_{cbg}, \phi N_{pn})$ $= \min(40.8, 48.1, 56.7)$ $= 40.8$ kips > $N_{ua} = 28$ kips OK
D.3.6.1	Ductility:	$\min(0.85N_{cbg}, 0.85N_{pn}) > N_{sa}$ $\min(0.85 \times 64.1, 0.85 \times 75.6)$ $\min(54.5, 64.3)$ $= 54.5$ kips > $N_{sa} = 51.0$ 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 in.

h_a = 18 in.

Base plate

8 x 8 in.

Spacing

s = 6 in.

Concrete

f_c' = 4000 psi

Stud material (A29/A108)[*]

f_{ya} = 51 ksi

f_{uta} = 65 ksi

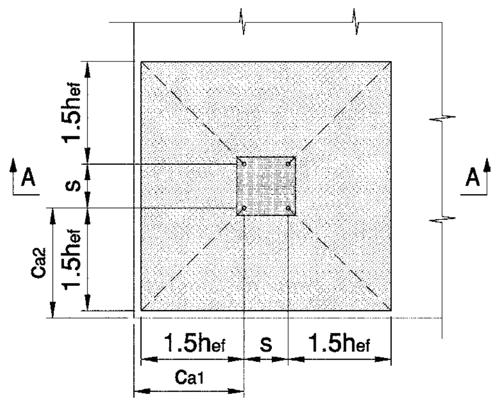
Plate

F_y = 36 ksi

Load

N_{ua} = 28 kips

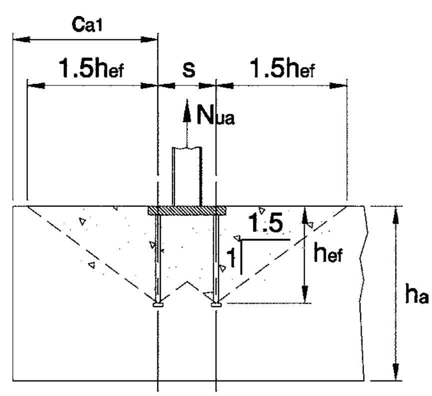
Where N_{ua} is the applied factored external load using load factors from Appendix C of the Code.



PLAN

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.



SECTION A-A

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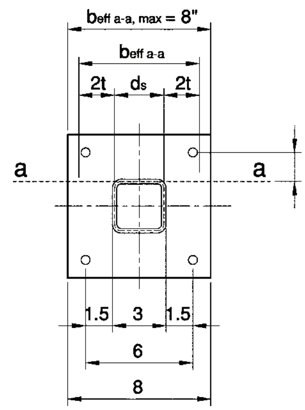
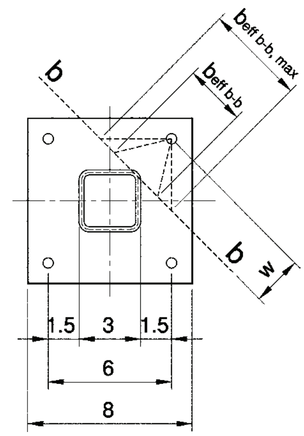
[*]Stud material is A29/A108, material properties per AWS D1.1, 2006, Table 7.1, Type B stud. Yield strength = 51 ksi; tensile strength = 65 ksi. It has elongation of 20% and reduction in area of 50%; meets the definition of a ductile steel element given in D.1, and meets the tensile strength requirements of D.5.1.2 and D.6.1.2: $f_{uta} \leq 1.9 f_{ya}$ ($65 \leq 1.9 \times 51 = 96.9$ 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}.	Equation no. $\phi N_n \geq N_{ua}$ (D-1)
	In ductile design, the internal strength N_n is controlled by the steel strength of the stud, N_{sa}.	$N_n = N_{sa} = nA_{se} f_{uta}$
D.4.5	The required steel strength of the stud, $N_{sa,req}$, is multiplied by ϕ = 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.	$0.80N_{sa,req} \geq 28$ $N_{sa,req} = 28/0.8$ $= 35.0$ kips
D.5.1	Solve for the required steel area $A_{se,req}$ for a single stud. Anchor diameter, in. — Effective area A_{se}, in.^{2} 3/8 — 0.110 1/2 — 0.196 **controls** (Refer to Appendix A, Table 6)	$N_{sa,req} = nA_{se,req} f_{uta}$ (D-3) $35.0 = 4A_{se,req} \times 65$ $A_{se,req} = 35.0/(4 \times 65)$ $= 0.13$ in.^{2} (required) **Use 1/2 in. diameter studs.** $A_{se} = 0.196$ in.$^2 > 0.13$ in.^{2} OK
D.5.1	Determine the nominal tensile strength N_{sa} of four 1/2 in. diameter studs.	$N_{sa} = nA_{se} f_{uta}$ (D-3) $= 4 \times 0.196 \times 65$ $= 51.0$ kips
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}.	From Step 1: $N_{sa} = 51.0$ kips
	The design concrete breakout strength shall be taken as 0.85 times the nominal strength.	$0.85N_{cbg,req} > N_{sa}$ $N_{cbg,req} > 51/0.85$ $N_{cbg,req} > 60.0$ 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 D.5.2.5	Modification factors are 1.0 for: 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 $c_{a,min} \geq 1.5h_{ef}$ 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.	$\psi_{cp,N}$ = 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 $k_c = 24$.	$N_b = 24(4000)^{0.5}h_{ef}^{1.5}$ $= 1.52h_{ef}^{1.5}$ kips
D.5.2.1	N_{cbg} is the nominal concrete breakout strength in tension of a group of anchors. **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.**	$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}N_b$ (D-5) $= (A_{Nc}/A_{Nco}) \times 1.0 \times 1.0 \times 1.0 \times 1.52 \times h_{ef}^{1.5}$ $= (A_{Nc}/A_{Nco}) \times 1.52 \times (h_{ef})^{1.5}$

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 2 (cont.)		
D.5.2.1	A_{Nc} is the projected area of the failure surface for the group of anchors. Refer to Fig. RD.5.2.1 in Commentary for guidance in calculating A_{Nc}. Spacing is 6 in. per problem statement.	$A_{Nc} = (3h_{ef} + s)^2$ $= [(3 \times 8) + 6]^2$ $= 900$ in.^{2}
D.5.2.1	A_{Nco} is the projected area of the failure surface of a single anchor remote from edges.	$A_{Nco} = 9 \times h_{ef}^2$ (D-6) $= 9 \times 8^2$ $= 576$ in.^{2}
D.5.2.1	Ratio of areas.	$A_{Nc}/A_{Nco} = 900/576$ $= 1.56$
D.5.2.2	Basic concrete breakout strength.	$N_b = 1.52(h_{ef})^{1.5}$ $= 1.52 \times 8^{1.5}$ $= 34.4$ kips
D.5.2.1	Nominal group concrete breakout strength N_{cbg}.	$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}N_b$ (D-5)
	Because N_{cbg} of 53.6 kips is less than required (60 kips), one needs to increase the effective embedment depth h_{ef} and try again.	$N_{cbg} = 1.56 \times 34.4$ $N_{cbg} = 53.6$ kips < 60 kips (No good)
	Second iteration: Try $h_{ef} = 9$ in.	
D.5.2.1	Determine A_{Nc}.	$A_{Nc} = (3h_{ef} + s)^2$ $= [(3 \times 9) + 6]^2$ $= 1089$ in.^{2}
D.5.2.1	Determine A_{Nco}.	$A_{Nco} = 9 \times h_{ef}^2$ (D-6) $= 9 \times 9^2$ $= 729$ in.^{2}
D.5.2.1	Determine the ratio of A_{Nc}/A_{Nco}.	$A_{Nc}/A_{Nco} = 1089/729$ $= 1.49$
D.5.2.2	Basic concrete breakout strength.	$N_b = 1.52(h_{ef})^{1.5}$ $= 1.52 \times 9^{1.5}$ $= 41.0$ kips
D.5.2.1	Nominal group concrete breakout strength N_{cbg}.	$N_{cbg} = 1.49 \times 41.0$ $N_{cbg} = 61.1$ kips ≥ 60 kips OK
	Because the concrete breakout strength N_{cbg} of 61.1 kips is greater than the required value of 60 kips, the embedment depth h_{ef} of 9 in. will produce a ductile design.	$h_{ef} = 9$ in. is OK
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.	$L_{req} = h_{ef,req}$ + head thickness + burnoff $= 9.0 + 0.312 + 0.125$ $= 9.4$ in. **Use four 1/2 in. diameter x 9-1/2 in. long studs.**
	Determine the concrete breakout strength N_{cbg} in tension of the anchor group using the final embedment and spacing.	
	Determine actual h_{ef}.	$h_{ef} = L$ – head thickness – burnoff $= 9.5 - 0.312 - 0.125$ $= 9.06$ in.
	Spacing is 6 in. as per problem statement.	$s = 6$ in.
D.5.2.1	Determine A_{Nc}.	$A_{Nc} = (3h_{ef} + s)^2$ $= [(3 \times 9.06) + 6]^2$ $= 1101$ in.^{2}

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 2 (cont.)		
D.5.2.1	Determine A_{Nco}.	$A_{Nco} = 9 \times h_{ef}^2$ (D-6) $= 9 \times (9.06)^2$ $= 739$ in.^{2}
D.5.2.1	Determine the ratio of A_{Nc}/A_{Nco}.	$A_{Nc}/A_{Nco} = 1100/739$ $= 1.49$
D.5.2	Determine basic concrete breakout strength. 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 above.	$N_b = 1.52(h_{ef})^{1.5}$ $= 1.52 \times (9.06)^{1.5}$ $= 41.5$ kips
	Nominal group concrete breakout strength N_{cbg}.	$N_{cbg} = 1.49 \times 41.5$ $= 61.7$ kips
	Check ductility. $0.85N_{cbg} > N_{sa}$	$0.85N_{cbg} = 0.85 \times 61.7$ $= 52.4$ kips > 51.0 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.	$N_{pn} = \psi_{c,P}N_p$ (D-14)
D.5.3.1	N_p is the pullout strength in tension of a single anchor in cracked concrete.	$N_p = 8A_{brg}f_c'$ (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).	$A_{brg} = \pi \times (1.00^2 - 0.50^2)/4$ $= 0.59$ in.^{2} $N_p = 8 \times 0.59 \times 4$ $= 18.9$ kips $N_{pn} = \psi_{c,P}N_p$ (D-14) $= 1.0 \times 18.9$ $= 18.9$ kips/stud $= 75.6$ kips (4 studs)
D.3.6.1	Check ductility. $0.85N_{pn} > N_{sa}$	$0.85N_{pn} = 0.85 \times 75.6$ $= 64.3$ kips > 51.0 kips (N_{sa}) OK
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}).	$c_{a1} \geq 0.4h_{ef}$ $\geq 0.4 \times 9.06$ ≥ 3.6 in. $c_{a1} = 15$ in. > 3.6 in. OK
STEP 5: Summary.		
Given	Applied load	$N_{ua} = 28$ kips
Step 1 D.4.5.a	Design steel tensile strength	$\phi N_{sa} = 0.8 \times 51.0 = 40.8$ kips
Step 2 D.4.5.c	Design concrete breakout strength	$\phi N_{cbg} = 0.75 \times 61.7 = 46.3$ kips
Step 3 D.4.5.c	Design concrete pullout strength	$\phi N_{pn} = 0.75 \times 75.6 = 56.7$ kips
Step 4 D.4.5.c	Design concrete side-face blowout strength	$\phi N_{sb} = $ N/A
D.4.1.2	Design strength of stud in tension	$\phi N_n = \min(\phi N_{sa}, \phi N_{cbg}, \phi N_{pn})$ $= \min(40.8, 46.3, 56.7)$ $= 40.8$ kips $> N_{ua} = 28$ kips OK

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 5 (cont.)		
D.3.6.1	Ductility:	min($0.85N_{cbg}$, $0.85N_{pn}$) > N_{sa} min(0.85 × 61.7, 0.85 × 75.6) > 51.0 min(52.4, 64.3) = 52.4 kips > N_{sa} = 51.0 kips OK
STEP 6: Check the required plate thickness.		
	The plate is 8 x 8 in. as per problem statement.	
D.3.1	The plate must transmit to the studs all loads used in the design of the attachment. Appendix C load factors are conservative compared with AISC. Evaluation Sections a-a and b-b to determine minimum load capacity. Yield of the plate material is 36 ksi. The plate is designed in accordance with the AISC-LRFD Code (ASD design is also acceptable). The design flexural strength is based on the limit state of yielding and is equal to $\phi_b M_n$ where, per Chapter F of AISC, ϕ_b = 0.90. $M_n = M_p = F_y Z \leq 1.5M_y$	
	Evaluate plate at Section a-a: Moment in plate at Section a-a: Use the midthickness of the tube as a point of fixity. Nominal capacity of the plate. The effective width is assumed to be equal to the width of the attachment plus 2t on either side. Use the plastic section modulus Z. Assume plate thickness is 1/2 in. Plastic moment: Nominal moment capacity M_n.	
	Required thickness on Section a-a. Evaluate Section b-b: Force in one bolt: Applied moment: The distance w is the distance from the corner anchor to the midthickness of the tube.	At face of tube (a-a): Tension in two studs† T_{2studs} = 28/2 = 14.0 kips a = 1.5 + 3/32 = 1.6 $M_{u,a\text{-}a} = T_{2studs}a$ = 14 × 1.6 = 22.4 in.-kips $M_{p,a\text{-}a} = Z \times F_y$ $Z = 1/4 \times b_{eff,a\text{-}a} \times t^2$ $b_{eff,a\text{-}a}$ = 3 + 2 × 0.5 + 2 × 0.5 = 5 in. F_y = 36 ksi $M_{p,a\text{-}a} = Z \times F_y$ $= 1/4 \times 5 \times t^2 \times 36$ $= 45.0t^2$ $\phi_b M_n = \phi_b Z F_y$ $= 0.9 \times 45t^2$ $= 40.5t^2$ $t_{min,a\text{-}a} = \sqrt{22.4/40.5}$ = 0.74 in. T = 28/4 = 7 kips $M_{u,b\text{-}b} = Tw$ $w \cong 1.5 \times 2^{0.5} + t_{tube}/2$ = 2.22 in. $M_{u,b\text{-}b}$ = 7 × 2.22 = 15 in.-kips

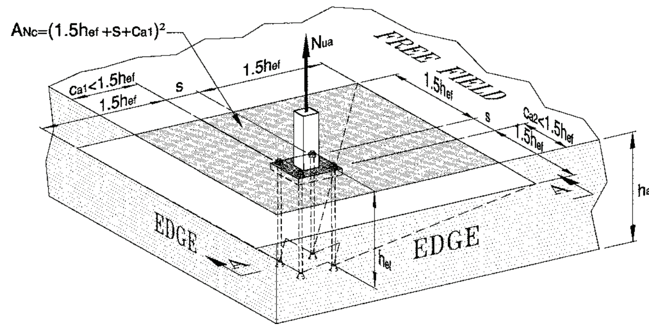
CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 6 (cont.)		
	The nominal moment capacity at Section b-b: The effective width at Section b-b is assumed to be equal to $4t$, $2t$ on either side of the corner of the tube, but not greater than $2w$. From Section a-a evaluation, t = 0.74 in. Use t = 0.75 in.	 $M_{p,b\text{-}b} = Z \times F_y$ $Z = 1/4 \times b_{eff,b\text{-}b} \times t^2$ $b_{eff,b\text{-}b}$ = 4 × (3/4) in. = 3.0 < $2w$ = 2 × 2.22 = 4.44 F_y = 36 ksi $M_p = Z \times F_y$ $= 1/4 \times 3.0 \times t^2 \times 36$ $= 27t^2$ $\phi_b M_n = 0.9 \times 27t^2$ $= 24.3t^2$ $t_{min,a\text{-}a} = \sqrt{15.5/24.3}$ = 0.80 in. **controls** Use t = 7/8 in. **Use 8 in. x 8 in. x 7/8 in. embedded plate.**

*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.

†In this problem, it is assumed that prying does not occur, and the force in individual anchors under the applied tension force is not increased by the prying effect. Prying may exist depending on the thickness of the plate, the location of the anchor, and the stiffness of the anchor.

Example B1(c)—Four-bolt surface-mounted plate, tension only, close spacing, close to a corner

Design an embedment with four post-installed undercut anchors and a surface-mounted plate for a 3 x 3 x 3/16 in. A501 structural tube attachment.



Given:

Concrete edges
$c_{a1} = c_{a2} = 12$ in.

Base plate
8 x 8 in.

Spacing
$s = 6$ in.

Concrete
$f_c' = 4000$ psi

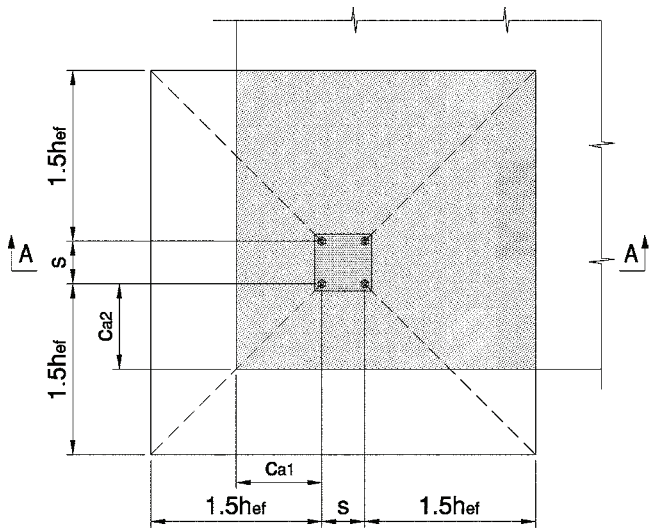
Anchor material (F 1554 Gr. 36)*
$f_{ya} = 36$ ksi
$f_{uta} = 58$ ksi

Anchor type
Threaded, undercut
$k_c = 24$ from product-specific tests

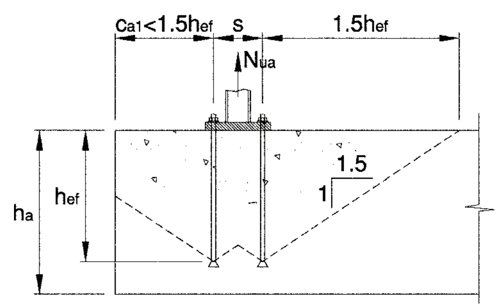
Plate
$F_y = 36$ ksi

Load
$N_{ua} = 28$ kips

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



PLAN



SECTION A-A

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.

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*Anchor material is ASTM F 1554 Gr. 36. It has elongation of 23% and reduction in area of 2 in., and meets the definition of a ductile steel element given in D.1 (f_{uta} = 58 ksi < $1.9f_{ya}$ = 1.9 × 36 = 64 ksi).

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 1: Determine the required anchor diameter.		
		Equation no.
D.4.1.1	Equate internal design strength ϕN_n to the external factored load N_{ua}.	$\phi N_n \geq N_{ua}$ (D-1)
	In ductile design, the internal strength N_n is controlled by the steel strength of the bolt, N_{sa}.	$N_n = N_{sa} = nA_{se} f_{uta}$
D.4.5	The required steel strength of the bolt, $N_{sa,req}$, is multiplied by ϕ = 0.80 for tension because the embedment and the steel bolt is ductile and the load factors are based on Appendix C of the Code.	$0.80 \times N_{sa,req} \geq 28$ $N_{sa,req} = 28/0.8$ $= 35.0$ kips
D.5.1	Solve for the required steel area, $A_{se,req}$, for a single bolt. (Refer to Appendix A, Table 3A) <u>Anchor diameter, in.</u> — <u>Effective area A_{se}, in.^{2}</u> 1/2 — 0.142 5/8 — 0.226 **controls**	$N_{sa,req} = nA_{se,req}f_{uta}$ (D-3) $35.0 = 4 \times A_{se,req} \times 58$ $A_{se,req} = 35.0/(4 \times 58)$ $= 0.15$ in.^{2} (required) **Use 5/8 in. diameter bolts** $A_{se} = 0.226$ in.^{2} > 0.15 in.^{2} OK
D.5.1.2	Determine the nominal tensile strength N_{sa} of four 5/8 in. diameter bolts. Note: It is assumed that prying will not occur. See † note in Example B1(b).	$N_{sa} = nA_{se} f_{uta}$ (D-3) $= 4 \times 0.226 \times 58$ $= 52.4$ kips
STEP 2: Determine the minimum embedment of the anchors 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}.	
D.5.2	The design concrete breakout strength shall be taken as 0.85 times the nominal strength.	$0.85N_{cbg} > N_{sa}$ $N_{cbg,req} \geq N_{sa}/0.85$ $N_{cbg,req} \geq 52.4/0.85$ $N_{cbg,req} = 61.6$ kips
D.5.2.1	N_{cbg} is the nominal concrete breakout strength in tension of a group of anchors.	$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$ (D-5)
	A_{Nc} is the projected area of the failure surface for the group of anchors. Refer to Fig. RD.5.2.1 in Commentary and the figure at the beginning of this problem for guidance in calculating A_{Nc}. Spacing s is 6 in. per problem statement. Edge distances c_{a1} and c_{a2} are 12 in. per problem statement. Because it is not known if the edge distance has an effect on A_{Nc} (that is, it needs to be checked that $c_{a,min} \leq 1.5h_{ef}$), it will be assumed it does; $\psi_{ed,N}$ and A_{Nc} will be calculated for each iteration.	$A_{Nc} = [(1.5 \times h_{ef}) + s + c_{a1}]^2$ $A_{Nc} = (1.5h_{ef} + 18)^2$
	A_{Nco} is the projected area of the failure surface of a single anchor remote from edges.	$A_{Nco} = 9 \times h_{ef}^2$ (D-6)
 D.5.2.4 D.5.2.5 D.5.2.6 D.5.2.7 D.8.6	Modification factors: Eccentricity effects $\psi_{ec,N}$. Assume no load eccentricity. Edge effects $\psi_{ed,N}$ Concrete cracking $\psi_{c,N}$. Concrete is cracked per problem statement. $\psi_{cp,N}$ depends on tests. $\psi_{cp,N} = 1.5h_{ef}/c_{ac}$. If no tests are done, then $c_{ac} = 2.5h_{ef}$. If reinforcement is included to control splitting or if analysis indicates cracking at service loads, then $\psi_{cp,N}$ is taken at 1.0. In this problem, we will assume adequate reinforcing exists to preclude splitting. Therefore, use $\psi_{cp,N} = 1.0$.	 $\psi_{ec,N} = 1.0$ $\psi_{ed,N} = [0.7 + 0.3 \times (c_{a,min}/(1.5h_{ef})]$ (D-11) $\psi_{c,N} = 1.0$ $\psi_{cp,N} = 1.0$
D.5.2.2	N_b is the basic concrete breakout strength in tension of a single anchor in cracked concrete. Assume the embedment will be less than 11 in., and use Eq. (D-7). Use k_c = 24 from product-specific test for this example problem. The k_c factor for post-installed anchors shall be permitted to be increased above 17 based on D.3.3 product-specific tests.	$N_b = k_c\sqrt{f'_c}\ h_{ef}^{1.5}$ (D-7) $N_b = 24(4000)^{0.5}(h_{ef})^{1.5}$ $= 1.52(h_{ef})^{1.5}$ kips

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 2 (cont.)		
	Use trial and error; increase h_{ef} until N_{cbg} is equal to or greater than the required $N_{cb,req}$. **First iteration:** Try h_{ef} = 9 in.—the same embedment used in Example B1(b).	
D.5.2.1	Determine A_{Nc} for the corner location. Note that the embedment dimensions are symmetric and that is located near a corner.	$A_{Nc} = (1.5h_{ef} + 18)^2$ $= [(1.5 \times 9) + 18]^2$ $= 992.3$ in.^{2}
D.5.2.1	Determine A_{Nco}.	$A_{Nco} = 9 \times h_{ef}^2$ (D-6) $= 9 \times 9^2$ $= 729$ in.^{2}
D.5.2.1	Determine the ratio of A_{Nc}/A_{Nco}.	$A_{Nc}/A_{Nco} = 992.3/729 = 1.36$
D.5.2.5	Determine edge effect factor $\psi_{ed,N}$.	$\psi_{ed,N} = 0.7 + 0.3 \times [c_{a,min}/(1.5h_{ef})]$ (D-11) $= 0.7 + 0.3 \times [12/(1.5 \times 9)]$ $= 0.7 + 0.26$ $= 0.97$
D.5.2.2	Basic concrete breakout strength.	$N_b = 1.52h_{ef}^{1.5}$ $= 1.52(9)^{1.5}$ $= 41.0$ kips
D.5.2.1	Nominal group concrete breakout strength N_{cbg}.	$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$ (D-5)
	Because N_{cbg} is less than $N_{cbg,req}$, the effective embedment depth needs to be increased.	$N_{cbg} = 1.36 \times 1.0 \times 0.97 \times 1.0 \times 1.0 \times 41.0$ $N_{cbg} = 54.1$ kips < 61.6 kips (No good)
	Second iteration: Try h_{ef} = 16 in.	h_{ef} = 16 in.
D.5.2.1	Determine A_{Nc} for the corner location.	$A_{Nc} = (1.5h_{ef} + 18)^2$ $= [(1.5 \times 16) + 18]^2$ $= 1764$ in.^{2}
D.5.2.1	Determine A_{Nco}.	$A_{Nco} = 9 \times h_{ef}^2$ (D-6) $= 9 \times 16^2$ $= 2304$ in.^{2}
D.5.2.1	Determine the ratio of A_{Nc}/A_{Nco}.	$A_{Nc}/A_{Nco} = 1764/2304 = 0.77$
D.5.2.5	Determine edge effect factor $\psi_{ed,N}$.	$\psi_{ed,N} = 0.7 + 0.3 \times [c_{a,min}/(1.5h_{ef})]$ (D-11) $= 0.7 + 0.3 \times [12/(1.5 \times 16)]$ $= 0.85$
D.5.2.1	Basic concrete breakout strength.	$N_b = 1.52(h_{ef})^{1.5}$ $= 1.52 \times 16^{1.5}$ $= 97.3$ kips
D.5.2.1	Nominal group concrete breakout strength N_{cbg}.	$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}\psi_{cp,N}N_b$ (D-5) $N_{cbg} = 0.77 \times 1.0 \times 0.85 \times 1.0 \times 1.0 \times 97.3$ $N_{cbg} = 63.7$ kips > 61.6 kips OK for ductility **Use 5/8 in. diameter bolt, with 16 in. embedment depth.**
	Check strength.	$\phi N_{cbg} = 0.75 \times 63.7$ $= 47.8$ kips > 28 kips OK

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 3: Check pullout strength of anchor.		
D.3.6.1	**Ensure steel failure:** To prevent concrete breakout failure in tension, the design concrete breakout strength $\phi n N_{pn}$ has to exceed the nominal tensile strength of the embedment steel, N_{sa}. To satisfy the ductility requirements of D.3.6.1, the design pullout strength shall be taken as 0.85 times the nominal strength.	$0.85N_{pn} > N_{sa}$
D.5.3.1	**Determine N_{pn}:** N_{pn} is the nominal pullout strength in tension of a single anchor.	$N_{pn} \geq N_{sa}/0.85$ $N_{pn} \geq 13.1/0.85$
D.5.3.2	For post-installed expansion and undercut anchors, the values of N_p shall be based on the 5% fractile of tests performed and evaluated according to D.3.3. It is not permissible to calculate the pullout strength in tension for such anchors. Therefore, testing for this specific anchor needs to show a result greater than $N_{pn,req}$, or the testing needs to show that pullout does not occur at all.	$N_{pn,req}$ = 15.4 kips for single anchor
STEP 4: Check concrete side-face blowout.		
D5.4.1	Check concrete side-face blowout. (Same is also true for c_{a2}).	$c_{a1} \geq 0.4h_{ef}$ $0.4 \times 16 = 6.4$ in. c_{a1} = 12 in. > 6.4 in. OK
STEP 5: Summary		
Given	Applied load	N_{ua} = 28 kips
Step 1 D.4.5.a	Design steel tensile strength	$\phi N_{sa} = 0.8 \times 52.4 = 41.9$ kips
Step 2 D.4.5.c	Design concrete breakout strength	$\phi N_{cbg} = 0.75 \times 63.7 = 47.8$ kips
Step 3 D.4.5.c	Design concrete pullout strength (testing)	ϕN_{pn} = check with manufacturer
Step 4 D.5.4.1	Design concrete side-face blowout strength	ϕN_{sb} = N/A
D.4.1.2	Design strength of bolt in tension	$\phi N_n = \min(\phi N_{sa}, \phi N_{cbg})$ $= \min(41.9, 47.8)$ = 41.9 kips > N_{ua} = 28 kips OK
D.3.6.1	Check ductility Plate design: same as Example B1(b).	$0.85N_{cbg} \geq N_{sa}$ $0.85 \times 63.7 \geq 52.4$ kips 54.1 > 52.4 kips OK

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

Design an embedment using welded studs and embedded plate for a 3 x 3 x 1/4 in. A501 structural tube attachment.

Given:

Concrete edges

c_{a1} = 18 in.

h_a = 18 in.

s = 5 in.

c_{a2} = 35 in.

Concrete

f_c' = 4000 psi

Stud material (A29/A108)*

f_{ya} = 51 ksi

f_{uta} = 65 ksi

Plate

F_y = 36 ksi

Loads

M_u = 70 in.-kips

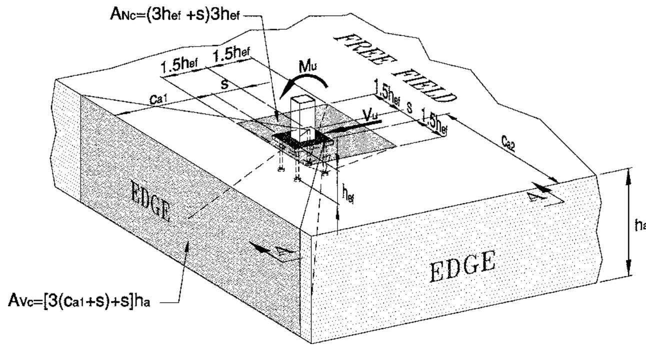
V_u = 12.4 kips

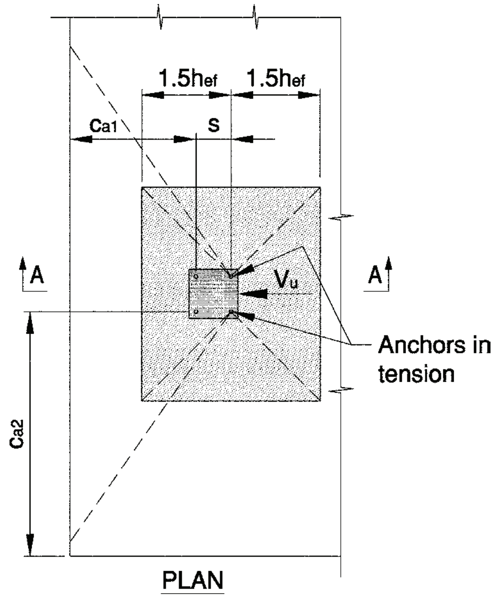
Where M_u and V_u are the applied factored external loads as defined in Appendix C of the Code.

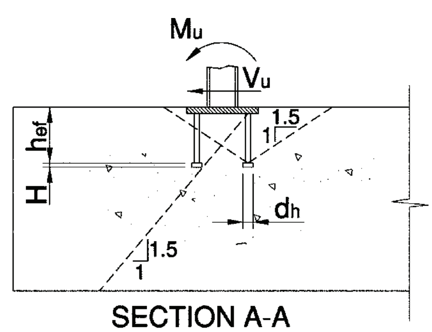
Note that the loads in this example have been selected to provide an example in which the anchors in tension must also carry shear.

Assumptions:

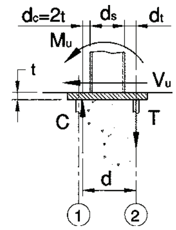
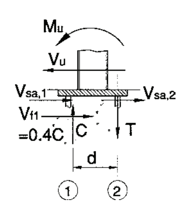
- Concrete is cracked.
- ϕ-factors are based on Condition B in D.4.5 of the Code (no supplementary reinforcement).





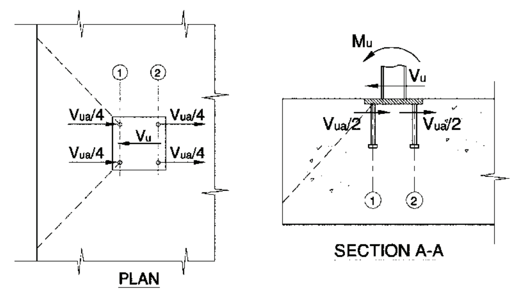
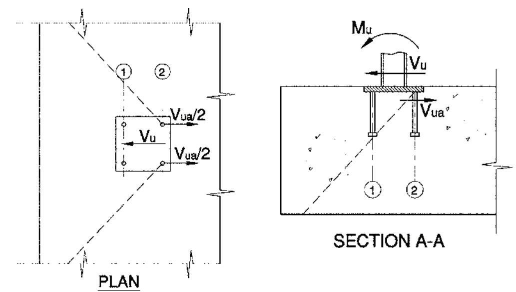
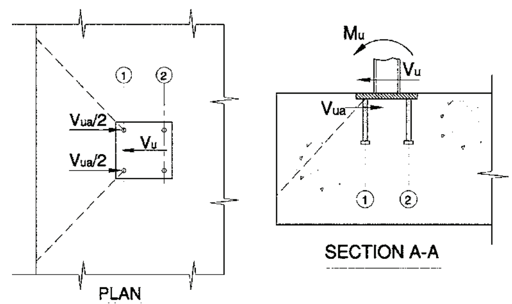


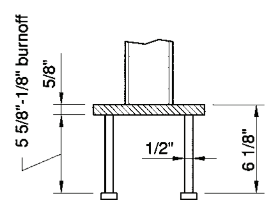
*Stud material is A29/A108, material properties per AWS D1.1, 2006, Table 7.1, Type B stud. Yield strength = 51 ksi; tensile strength = 65 ksi. It has elongation of 20% and reduction in area of 50%; meets the definition of a ductile steel element given in D.1, and meets the tensile strength requirements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

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 1: Determine required steel area of the studs.		
	BASE PLATE Determine the plate size and thickness and anchor bolt size for moment. Assume a 1/2 in. thick, 7 x 7 in. plate. Stud spacing s = 5 in., is given. Assume the center of compression force resultant is at a distance $d_c = 2t$ away from the outer edge of the supported member, the structural tube. The center of the tension force is at the center of the anchors in tension.	
	Determine the lever arm d between the center of compression and tension forces.	$d = d_t + d_s + d_c$ $= 1.0 + 3.0 + 1.0 = 5.0$ in.
	Calculate the tension in the anchors.	$T = M_u/d$ $= 70/5.0$ $= 14.0$ kips
AISC Chapter F	Design the plate using the AISC LRFD Code.† Use the plastic moment M_p to define the nominal strength of the plate in bending, and use $\phi_b = 0.9$ in accordance with the AISC Code. Assume that the full width of the base plate is effective.	$M_u = Td_t = 14.0 \times 1.0 = 14.0$ kip-in. $M_n = M_p = F_yZ$ $\phi_b M_p = 0.9F_yZ$
	Strength of steel plate is greater than the applied ultimate moment, but close to the limit. Use 5/8 in. plate.	$Z = 1/4(7)(0.5)^2 = 0.44$ in.^{3} $\phi_b M_p = 0.9(36)(0.44)$ $= 14.3 > M_u = 14$ kip-in. OK Use 7 x 7 x 5/8 in. thick plate. $Z = 1/4(7)(0.625)^2 = 0.68$ in.^{3} $\phi_b M_p = 0.9(36)(0.68) = 22.1 > M_u = 14$ kip-in. OK
	ANCHORS	
D.4.1.1	**Tension:**	
	Determine the required stud area $A_{se,t}$ for tension. Assume ductile design and use the corresponding ϕ factor in accordance with D.4.5. $\phi = 0.8$ for tension load. From Appendix A, Table 2: <u>Anchor diameter, in.</u> — <u>Effective area (gross area for stud), in.^{2}</u> 3/8 — 0.110 1/2 — 0.196 5/8 — 0.307	$T = N_{ua} = \phi N_{sa}$ $= \phi n_{Ase,t} f_{uta}$ $\phi = 0.80$ n = two anchors $f_{uta} = 65$ ksi $A_{se,t,req} = N_{ua}/\phi n f_{uta}$ $= 14.0/(0.8 \times 2 \times 65)$ $A_{se,t,req} = 0.135$ in.^{2} Use four 1/2 in. diameter studs. $A_{se} = A_{se,t,prov} = 0.196$ in.^{2} OK
	Shear: *Shear forces in anchors*: From the figure on the right, the forces in each anchor group and force due to friction can be calculated.	

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 1 (cont.)		
	From above, C_F and T are already calculated.	$C_F = T = 14.0$ kips
D.6.1.4	Determine friction force V_{f1}.	$V_{f1} = 0.4C_F = 0.4 \times 14 = 5.6$ kips
	Shear strength: Because this is an embedded base plate, shear strength may be calculated using either direct shear or shear friction; however, only the direct shear option will be shown herein.‡	
	Using the steel area A_{se} determined from tension above, calculate the available shear strength of the anchors.	
	Shear strength using direct shear and friction from compression: *A. Assume all four factors resist shear* (Concrete breakout strength needs to be checked; refer to Step 3).	$A_{se,v} = A_{se,v,1} + A_{se,v,2}$ $A_{se,v,1} = A_{se,v,2}$ $A_{se,v,1} = nA_{se} = 2 \times 0.196 = 0.392$ in.^{2}
D.6.1	Strength of front and back anchors, $V_{sa,1}$ and $V_{sa,2}$:	$V_{sa,1} = V_{sa,2} = nA_{se,1}f_{uta}$ (D-18) $= 2 \times 0.196 \times 65 = 25.5$ kips
	Total strength of front and back anchors:	$V_{sa} = V_{sa,1} + V_{sa,2}$ $V_{sa} = 2 \times 25.5 = 51.0$ kips
	Strength reduction factor:	$\phi = 0.75$
D.4.5.c	Design strength of anchors at Line 1, $\phi V_{sa,1}$ and Line 2, $\phi V_{sa,2}$ (Note: strength is based on f_{uta}, not f_{ya}).	$\phi V_{sa,1} = \phi V_{sa,2} = 0.75 \times 25.5 = 19.1$ kips
	Total design steel strength of four anchors:	$\phi V_{sa} = 2 \times 19.1$ $= 38.2$ kips
D.6.1.4	Additional shear strength is provided from friction on the compression side of the plate. ϕ is taken as 0.75 from Condition B.	$V_{f1} = 0.4C = 0.4 \times 14 = 5.6$ kips $\phi V_{f1} = 0.75 \times 5.6 = 4.2$ kips
	Design shear strength of the connection, four anchors plus frictional resistance:	$\phi V_{sa} + \phi V_{f1} = 38.2 + 4.2 = 42.4$ kips > 12.4 kips OK
	Note: It is prudent to ignore the shear strength provided by frictional resistance, especially for a new design. It is included herein, however, to illustrate the procedure as the Code permits it. It is best used to avoid rework if shear demand increases later due to a reanalysis or retrofit.	
	B. Assume all shear taken by back two anchors: (Line 2 in figure). Friction will not be included.	$\phi V_{sa,2} = 19.1$ kips
	Design strength of anchors $\phi V_{sa,2}$ at Line 2:	$\phi V_{sa,2} > V_{ua} = 12.4$ kips OK
STEP 2: Determine required embedment length for the studs breakout failure in tension.		
D.5.1	Calculate the steel strength of anchors in tension. Only anchors on Line 2 resist tension.	$N_{sa} = 2A_{se,t}f_{uta}$ $= 2(0.196)(65)$ $= 25.5$ kips
D.5.2.2 D.3.6.1	Calculate the required concrete breakout strength of anchors in tension to ensure that embedment is ductile.	$N_{sa} = 0.85N_{cbg}$ $N_{cbg,req} = 25.5/0.85 = 29.9$ kips
	Concrete breakout strength for 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)
	Try 5-5/8 in. long Nelson stud.	
D.5.2.1	Calculate A_{Nco}. Note that because the failure surface extends to the top of the concrete, the thickness of the plate is added to the anchor length.	$A_{Nco} = 9h_{ef}^2$ (D-6) $= 9 \times (6.125)^2$ $= 337.6$ in.^{2}
	$h_{ef} = 5.625 + t_{plate} -$ burnoff (See Appendix A, Table 6) $= 5.625 + 0.625 - 0.125$ $= 6.125$ in.	

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 2 (cont.)		
	Calculate A_{Nc}. There are no edge effects because 1.5 × 6.125 = 9.2 in. < c_{a1} = 23 in.	A_{Nc} = (2 × 1.5 × 6.125 + 5)(2 × 1.5 × 6.125) = 429.5 in.^{2} A_{Nc}/A_{Nco} = 429.5/337.6 = 1.3
	Modification factors are 1.0 for:	
D.5.2.4	Eccentricity effects $\psi_{ec,N}$	$\psi_{ec,N}$ = 1.0 (D-9)
D.5.2.5	Edge effects $\psi_{ed,N}$	$\psi_{ed,N}$ = 1.0 (D-10)
D.5.2.6	Concrete cracking $\psi_{c,N}$	$\psi_{c,N}$ = 1.0
D.5.2.7	Concrete corner splitting $\psi_{cp,N}$	$\psi_{cp,N}$ = N/A (not post-installed) (D-12)
D.5.2.2	N_b is the basic concrete breakout strength in tension of single anchor in cracked concrete. For cast-in-place anchors, use k_c = 24.	$N_b = k_c\sqrt{f_c'}\ h_{ef}^{1.5}$ (D-7) $N_b = 24(4000)^{0.5}(h_{ef})^{1.5}$ $= 1.52(6.125)^{1.5}$ = 23.0 kips
D.5.2.1	Concrete breakout strength for group.	$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$ (D-5) = 1.3 × 1.0 × 1.0 × 1.0 × 23.0 = 29.9 kips ≥ $N_{cbg,req}$ = 29.9 kips OK
	Embedment is ductile in tension. Strength controlled by steel. This satisfies shear friction requirement from Step 1.	Use four 1/2 in. diameter x 5-5/8 in. long anchors at 5 in. spacing.
STEP 3: Check pullout strength of anchor.		
D.5.3	Calculate the pullout strength of the anchor in tension in accordance with D.5.3. Design embedment as ductile in accordance with D.3.6.1.	$N_{pn} = \psi_{c,P}N_p$ (D-14) $N_p = 8A_{brg}f_c'$ (D-15) $= 8 \times 4A_{brg}$ $= 32A_{brg}$
D.5.3.4	Concrete is cracked per problem statement.	$\psi_{c,P}$ = 1.0
D.5.3.5	Calculate the bearing area. From manufacturer data, anchor head diameter is 1.0 in. for a 1/2 in. diameter stud (see Table 6 in Appendix A).	$A_{brg} = \pi \times (1.00^2 - 0.50^2)/4$ = 0.59 in.^{2}
D.3.6.1	Pullout strength to maintain ductile design in accordance with D.3.6.1.	N_{pn} = 1.0 × 32 × 0.59 = 18.9 kips $0.85N_{pn}$ = 0.85 × 18.9 = 16.1 kips each anchor = 32.2 kips for two anchors --> N_{sa} = 25.5 kips **Ductile** **1/2 in. diameter studs are OK for pullout.**
STEP 4: Check concrete side-face blowout.		
D.5.4.1	Check anchors closest to the edge for side-face blowout.	$c_{a1} > 0.4h_{ef}$ 18 in. > 0.4 × 6.125 = 2.45 in. **Side-face blowout N_{sb} need not be checked.**

CODE SECTION	DESIGN PROCEDURE	CALCULATION	
STEP 5: Check concrete breakout failure in shear.			
	To determine the critical concrete breakout strength, three modes of failure would be considered. They are shown to the right. The first mode places the failure cone at the front anchors, and a strength check is made against 1/2 of the applied shear V_{ua}. The second mode considers the failure cone initiating at the back anchors, and a strength check is made against the total shear V_{ua}. The final mode is a conservative check, which assumes that all of the shear is acting at the front anchors. This check might be considered if significantly oversized holes are used as in a column base plate. Per the note on Fig. RD.6.2.1(b) of the Commentary, the only check that is required for this problem is Mode 2, because the studs are welded to the anchor plate.	 Mode 1  **Mode 2** (only check required for welded studs)  Mode 3	
	Determine concrete breakout strength using Mode 2.		
D.6.2.1	For a plate with welded studs, Mode 2 can be used to calculate the concrete breakout strength. Therefore, c_{a1} = 18 + 5 = 23 in. The failure cone passes through the bottom of the slab, so the A_{Vc} equation uses the full slab thickness (h_a = 18 in. < 23 in.).	$V_{cbg} = (A_{Vc}/A_{Vco})\psi_{ec,V}\psi_{ed,V}\psi_{c,V}V_b$ $A_{Vc} = (3c_{a1} + s)(h_a)$ $= (3 \times 23 + 5)(18) = 1332 \text{ in.}^2$ $A_{Vco} = 4.5{c_{a1}}^2 = 4.5 \times 23^2 = 2381 \text{ in.}^2$ $A_{Vc}/A_{Vco} = 0.56$	(D-21) (D-22)
	Modification factors:		
D.6.2.5 D.6.2.6 D.6.2.7	Eccentricity on anchor group $\psi_{ec,V}$ Second edge effects $\psi_{ed,V}$ Assumed crack $\psi_{c,V}$	$\psi_{ec,V} = 1.0$ no eccentricity $\psi_{ed,V} = 1.0 c_2 > 1.5c_1$ $\psi_{c,V} = 1.0$	 (D-26)
D.6.2.3	Note: By definition, ℓ_e is limited to $8d_o$ = 4 in.	$V_b = 8(\ell_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f'_c}(c_{a1})^{1.5}$ $= 8(4/0.5)^{0.2}\sqrt{0.5}\sqrt{4000}(23)^{1.5}$ $= 59.8$ kips	(D-24)

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 5 (cont.)		
	Nominal concrete breakout strength:	$V_{cbg,2}$ = 0.56 × 1.0 × 1.0 × 1.0 × 59.8 = 33.5 kips
D.3.6.1	Check ductility, two anchors on Line 2:	$0.85V_{cbg,2}$ = 0.85 × 33.5 = 28.5 > $V_{sa,2}$ = 25.5 kips **Mode 2 is ductile.**
D.4.5	Check for strength:	
	Strength reduction factor:	ϕ = 0.75
D.6.2.1	Design group concrete breakout strength:	ϕV_{cbg} = 0.75 × 33.5 = 25.1 > V_{ua} = 8.2 kips **OK for strength**
STEP 6: Check group pryout.		
D.6.3	Concrete pryout of the anchors in shear must be checked.	$V_{cpg} = k_{cp}N_{cbg}$ (D-29) k_{cp} = 2 for h_{ef} > 2.5 in.
	Two anchors:	N_{cbg} = 29.9 kips V_{cpg} = 2 × 29.9 = 59.8 kips $0.85V_{cpg}$ = 0.85 × 59.8 = 50.8 > 25.5 kips **Ductile**
STEP 7: Summary		
	Stud: Diameter d_o = 1/2 in. Length L = 5-5/8 in. Effective depth h_{ef} = 6.13 in. Base plate thickness t = 5/8 in. Anchors are ductile.	
	TENSION	
Step 1 D.4.5.a	Applied load	N_{ua} = 14.0 kips
Step 3 D.4.5.c	Steel strength	ϕN_{sa} = 0.8 × 25.5 = 20.4 kips
Step 5 D.4.5.c	Concrete breakout strength	ϕN_{cbg} = 0.75 × 29.9 = 22.4 kips
Step 6 D.4.5.c	Concrete pullout strength	ϕN_{pn} = 0.75 × 2 × 18.9 = 28.4 kips
D.4.1.2	Concrete side-face blowout strength	$c_{a1} > 0.4h_{ef}$, so this is not applicable
	Design strength of stud in tension	$\phi N_n = \min(\phi N_{sa}, \phi N_{cbg}, \phi N_{pn})$ = min(20.4, 22.4, 28.4) = 20.4 kips > N_{ua} = 14.0 kips OK
	SHEAR: Mode 2 (frictional resistance counted)	
Step 1 D.4.5.a	Applied load, 12.4 – 4.2 = 8.2 kips	V_{ua} = 8.2 kips
Step 3 D.4.5.c	Steel strength	ϕV_{sa} = 0.75 × 25.5 = 19.1 kips
Step 7 D.4.5.c	Concrete breakout strength (nonductile)	ϕV_{cbg} = 0.75 × 33.5 = 25.1 kips
D.4.1.2	Concrete pryout strength	ϕV_{cpg} = 0.75 × 59.8 = 44.9 kips
	Design strength of stud in shear	$\phi V_n = \min(\phi V_{sa}, \phi V_{cbg}, \phi V_{cpg})$ = min(19.1, 25.1, 44.9) = 19.1 kips > V_{ua} = 8.2 kips OK

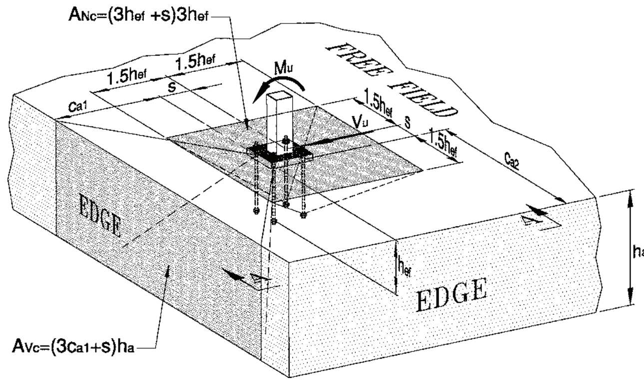
CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 8: Check for tension-shear interaction.		
D.7.3	Anchors subject to combined shear and tension forces must meet the tension-shear interaction requirements of D.7.	$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \le 1.2$ (D-30)
D.7.2	**Mode 2:** Design shear strength in shear: $V_{ua} > 0.2 \times$ design shear strength—interaction check is required.	$\phi V_n = 19.1$ kips Step 3 $(0.2)\phi V_n = 0.2 \times 19.1 = 3.8 < 8.2$ kips $14/20.4 + 8.2/19.1 = 0.69 + 0.43 = 1.12 < 1.2$ OK
	If the frictional resistance was ignored, then the aforementioned interaction equation becomes: and the anchors will have to be redesigned.	$14/20.4 + 12.4/19.1 = 0.69 + 0.65 = 1.34 > 1.2$
	Mode 3 assumption: An alternative assumption is that all shear is taken by only the anchors on the compression side. With this assumption, there is no interaction check because the anchors in tension are not in shear and the anchors in shear are not in tension. This approach, however, requires that Mode 3 failure for the concrete shear breakout strength (Step 5) be checked. Mode 3 will have lower concrete breakout strength, and is more likely to lead to a nonductile design.	

[*]**Notes on steel design:** The plate will be designed using the AISC-LRFD Code (American Institute of Steel Construction [1999] "Load Resistance Factor Design for Structural Steel Buildings," AISC, Chicago, IL). In applying it to this example, some conservative simplifying assumptions will be made: a) *Loads*: This example assumes that the loads are the same as used in the previous editions of the ACI 349 and therefore, the Appendix C ϕ-factors were used. The AISC Code uses the ASCE 7 (American Society of Civil Engineers, "Minimum Design Loads for Buildings and Other Structures," Reston, VA) load factors, and the strength reduction factors are determined accordingly. The loads used in this example are therefore conservative, and will be used with the LRFD design; b) *Strength reduction factors*: The strength reduction factors will be those of the AISC-LFRD Code (ϕ – 0.9 for bending); and c) *Strength design*: The nominal strength of a section in bending in the LRFD Code is based on a plastic section modulus Z and yield strength F_y of the steel material ($M_n = M_p = ZF_y$). This approach will be used in this example.

[†]D.4.3 of the Code requires the resistance to combined tensile and shear loads to be considered in design. In this problem, the tensile load on some of the anchors comes from the moment. The moment is assumed to be a result of the shear acting some distance from the face of the base plate. The anchorage, therefore, has no net externally applied tension force. The tension results in an equal and self-equilibrating compression force. The Code is not clear if the tension shear interaction equation is to be applied on an anchor-by-anchor basis or on the entire base plate, as was assumed in this example. Also, because Appendix D permits shear to be resisted by direct shear through individual anchors or by shear friction, either approach could be used; however, only the direct shear procedure is shown in this example problem.

Example B2(b)—Four-anchor surface-mounted plate, combined shear and uniaxial moment

Design an embedment using cast-in anchors and a flexible surface-mounted plate for a 3 x 3 x 1/4 in. A501 structural tube attachment.



Given:

Concrete edges

c_{a1} = 18 in.

h_a = 18 in.

s = 5 in.

c_{a2} = 35 in.

Concrete

f_c' = 4000 psi

Rod material (F 1554 Gr. 105)*

f_{ya} = 105 ksi

f_{uta} = 120 ksi

Plate

F_y = 36 ksi

Loads

M_u = 70 in.-kips

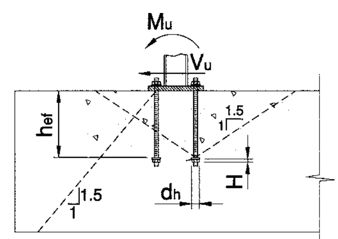
V_u = 12.4 kips

Where M_u and V_u are the applied factored external loads as defined in Appendix C of the Code.

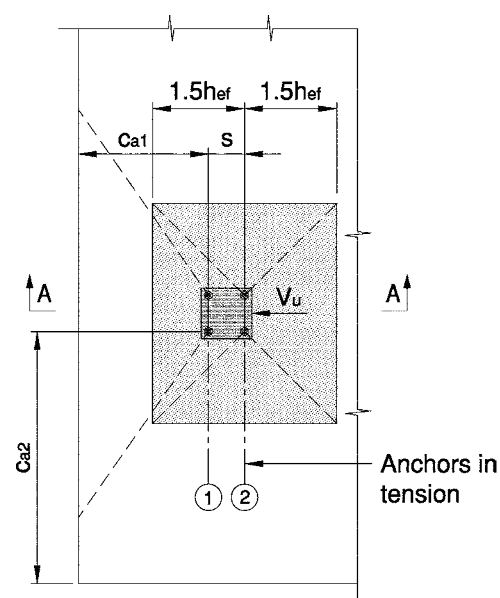
Refer to introduction for commentary on the distribution of stresses to the anchors for this problem.

Assumptions:

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

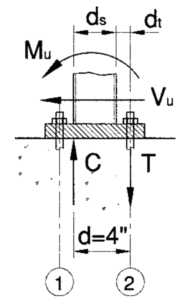


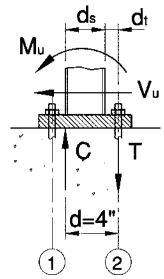
SECTION A-A



PLAN

*ASTM F 1554-00 specification, Grade 105, Class 1A, rod material will be used. Rod 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 to 3 in. diameters. Reductions in area requirements vary. For anchor diameters less then 2 in., elongation in 2 in. is 15%, 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 ≤ $1.9f_{ya}$ or 125 ksi. Refer also to Table 1 for other materials.

	DESIGN PROCEDURE	CALCULATION
STEP 1: Determine required steel area of the rod.		
	Try 7 x 7 in. plate with 5 x 5 in. rod spacing. Calculate the tension force in the anchors and compression reaction force in the concrete from the applied forces. Assume, conservatively, that the center of compression force resultant is at the outer edge of the supported member, the structural tube. (Alternately, it can be assumed that the center of the compression force is at a distance of 2*t* from the outer edge of the tube, as used in Example B2(a). This, however, requires the designer to estimate plate thickness and later verify that the thickness is no less than that assumed.) The center of the tensile force is at the center of the anchors in tension.	
D.5.1.2 D.4.5	Determined the required rod area $A_{se,t}$ for tension. Assume ductile design and use the corresponding ϕ-factor in accordance with D.4.5. Try 1/2 in. diameter rods. Gross cross-sectional area is 0.196 in.^{2} Tensile stress area is 0.142 in.^{2} (refer to Table 2, Appendix A) Anchor diameter, in. — Effective area (gross area for stud), in.^{2} 3/8 — 0.078 1/2 — 0.142 5/8 — 0.226	$N_u = C_F = M_u/d$ $= 70/4$ $= 17.5$ kips $N_{ua} = \phi N_{sa}$ $= \phi n A_{s,et} f_{uta}$ $\phi = 0.80$ n = two anchors $f_{uta} = 120$ ksi $A_{se,t,req} = N_{ua}/\phi n f_{uta}$ $= 17.5/(0.80 \times 2 \times 120)$ $= 0.091$ in.^{2} **Use 1/2 in. diameter rods.** $A_{se,t,prov} = 0.142 > 0.091$ in.^{2} OK
D.6.1.2(b)	Because this is a surface-mounted plate with cast-in headed bolts, direct shear is applicable.	
D.6.1.4	The nominal shear strength is the sum of the shear provided by the anchors and the friction force between the bases plate and concrete due to the compression reaction, taken as $0.40C_F$. Assume threads in shear plane.	Shear resistance $= V_{sa} + V_f$ $= 0.60 n A_{se,v} f_{uta} + 0.40 C_F$
	Shear strength of four anchors	$V_{sa} = 0.60 n A_{se,v} f_{uta}$ $= 0.60 \times 4 \times 0.142 \times 120$ $= 40.9$ kips
	Strength reduction factor	$\phi = 0.75$
	Design strength of four anchors	$\phi V_{sa} = 0.75 \times 40.9 = 30.68$ kips
D.6.1.4	Additional shear strength provided by friction between the base plate and concrete. (See note in Step 1 of Example B2(a) about the prudence of considering the shear strength from frictional resistance.)	$V_{f1} = 0.40C = 0.40T$ $= 0.40(17.5)$ $= 7$ kips
D.4.5.c	Strength reduction factor	$\phi = 0.75$
	Design strength provided by friction	$\phi V_{f1} = 0.75 \times 7 = 5.25$ kips $\phi V_{sa} + \phi V_{f1} = 30.68 + 5.25 = 35.93$ kips > 12.4 kips OK

	DESIGN PROCEDURE	CALCULATION
STEP 2: Design base plate.*		
ASIC LFRD Chapter F	Nominal flexural strength of base plate, per AISC LFRD Code, is $M_n = M_p = F_y Z$. The tension in the anchor is based on the applied moment, not the full tensile capacity of the anchor.	 $M_u = Td_t$ $= (17.5)(1.0)$ $= 17.5$ in.-kips
ASIC LFRD Chapter F	The ϕ-factor for flexure is ϕ_b is 0.90. Required plate thickness per AISC:	$\phi_b M_n = \phi_b F_y Z$ $= 0.9F_y(bt^2/4) = 0.9(36)(7)t^2/4$ $= 56.7t^2$ $56.7t^2 = 17.5$ $t \quad = 0.55$ in. **Use 7 x 7 x 5/8 in. plate.**
STEP 3: Determine required embedment length for the studs to prevent concrete breakout failure.		
D.5.1	Calculate design tension on anchors assuming two studs resist tensile loads.	$N_{sa} = nA_{se,t}f_{uta}$ (D-3) $= 2 \times 0.142 \times 120$ $= 34.1$ kips
D.3.6.1	Calculate the concrete breakout strength of anchors in tension so that embedment is ductile.	$N_{sa} = 0.85N_{cbg}$ $N_{cbg,req} = 34.1/0.85 = 42.6$ kips
D.5.2 D.3.6.1	Concrete breakout strength for a group of anchors Try 1/2 in. diameter headed rod with 8-1/2 in. effective length (h_{ef}).	$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}N_b$ (D-5) $A_{Nco} = 9h_{ef}^2$ (D-6) $= 9 \times 8.5^2$ $= 650$ in.^{2} $A_{Nc} = (2 \times 1.5 \times 8.5 + 5)(2 \times 1.5 \times 8.5)$ $= 778$ in.^{2} $A_{Nc}/A_{Nco} = 778/650$ $= 1.20$
D.5.2.4 D.5.2.5 D.5.2.6 D.5.2.7 D.5.2.2	Modification factors are 1.0 for: Eccentricity effects $\psi_{ec,N}$ Edge effects $\psi_{ed,N}$ Concrete cracking $\psi_{c,N}$. Concrete corner splitting $\psi_{cp,N}$	 $\psi_{ec,N} = 1.0$ $\psi_{ed,N} = 1.0$ $\psi_{c,N} = 1.0$ $\psi_{cp,N} =$ N/A
	N_b is the basic concrete breakout strength in tension of single anchor in cracked concrete. For cast-in-place anchors, use $k_c = 24$.	$N_b = k_c\sqrt{f_c'}\,h_{ef}^{1.5}$ (D-7) $= 24\sqrt{(4000)}\,h_{ef}^{1.5}$ $= 1518h_{ef}^{1.5}$ lb $= 1.52 \times 8.5^{1.5}$ kips $= 37.6$ kips
	Concrete breakout strength for group.	$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}N_b$ $N_{cbg} = 1.20 \times 1.0 \times 1.0 \times 1.0 \times 37.6 = 45.1 > 42.6$ kips OK Therefore, embedment is ductile. Strength controlled by steel. **Use four 1/2 in. diameter anchors with 8-1/2 in. embedment depth at 5 in. spacing.**

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 4: Check pullout strength of anchor.		
D.5.3	Calculate the pullout strength of the anchor in tension in accordance with D.5.3. Design embedment as ductile in accordance with D.3.6.1.	$N_{pn} = \psi_{c,P} N_p$ (D-14) $N_p = 8A_{brg} f_c'$ (D-15) $= 8 \times 4A_{brg}$ $= 32A_{brg}$
D.5.3.4	Concrete is cracked per problem statement.	$\psi_{c,P} = 1.0$
D.5.3.5	Calculate the bearing area. Assume heavy hex head for the rod. Appendix A, Table 4(c), for 1/2 in. diameter rod, F = 7/8 in., and C = 1.0 in. $A_H = (3F^2/2)$tan30 degrees $A_H = (3 \times 0.875^2/2)(0.577) = 0.663$ in.^{2} $A_D = \pi \times 0.5^2/4 = 0.196$ in.^{2}	$A_{brg} = A_h - A_D$ $= 0.663 - 0.196$ $= 0.47$ in.^{2}; see also Table 4(c)
	Pullout capacity for two anchors	$N_{pn} = 1.0 \times 32 \times 0.47$ $= 15.0$ kips each anchor
	Determine required bearing area	$0.85N_{pn} = 0.85 \times 2 \times 15.0$ $= 25.5 < N_{sa} = 34.1$ kips No good $A_{brg,req} = 34.1/(2 \times 32 \times 0.85)$ $= 0.625$ in.^{2} each anchor
	Try a hardened washer, with outside diameter (OD) (Table 5, SAE hardened washer)	$(OD)^2 = 4 \times (0.625)/\pi$ OD = 0.89 in. each anchor **Use a 1/2 in. washer with OD of 1.167 in.** $A_H = \pi \times 1.167^2/4 = 1.07$ in.^{2} $A_{brg} = A_H - A_D$ $= 1.07 - 0.196$ $= 0.874 > 0.625$ in.^{2} Check: $N_{pn} = 1 \times 32 \times 0.874$ $= 27.9$ kips each anchor $= 55.9$ kips for two anchors $0.85N_{pn} = 0.85 \times 55.9 = 47.5 > N_{sa} = 34.1$ kips Ductile, OK **Use 1/2 in. diameter rods with 1.167 in. OD SAE hardened washer on head.**
STEP 5: Check concrete side-face blowout.		
D.5.4	Check anchors closest to the edge for side-face blowout.	$c_{a1} > 0.4h_{ef}$ 18 in. > $0.4 \times 6.125 = 2.45$ in. **Side-face blowout N_{sb} need not be checked.**

CODE SECTION	DESIGN PROCEDURE	CALCULATION

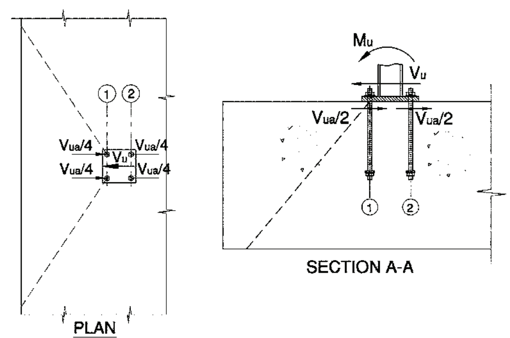
STEP 6: Check concrete shear breakout.

To determine the critical concrete breakout strength, three modes of failure should be considered. They are shown to the right. The first mode places the failure cone at the front anchors, and a strength check is made against 1/2 of the applied shear V_{ua}. The second mode considers the failure cone initiating at the back anchors, and a strength check is made against the total shear V_{ua}. The final mode is a conservative check, which assumes that all of the shear is acting at the front anchors. This check might be considered if significantly oversized holes are used as in a column base plate.

Concrete breakout		Steel strength		Applied load	
c_{a1}	$0.85V_{cbg,1}$	$V_{sa,1}$	$\phi V_{sa,1}$	$V_{ua,1}$	$V_{ua,2}$
18 in.	22.4 kips	20.5 kips	15.4 kips	3.6 kips‖	3.6 kips‖

Ductile—OK for strength; calculated below

‖Shear to be resisted = 12.4 – 5.25 = 7.15 kips
Each row resists, 0.5 × 7.15 = 3.6 kips

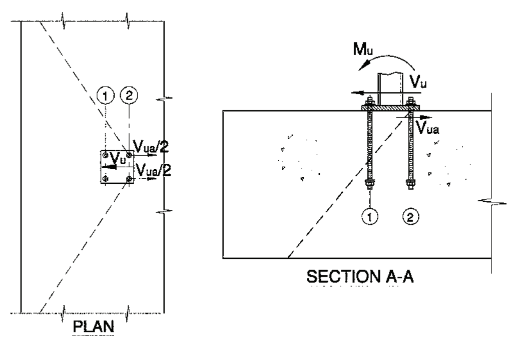


Mode 1

Concrete breakout		Steel strength		Applied load	
c_{a1}	$0.85V_{cbg,2}$	$V_{sa,2}$	$\phi V_{sa,2}$	$V_{ua,1}$	$V_{ua,2}$
23 in.	24.9 kips	20.5 kips	15.4 kips	3.6 kips‖	3.6 kips‖

Ductile—OK for strength; calculated below

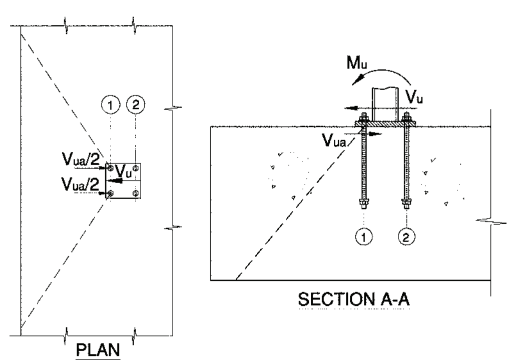
‖See note above.



Mode 2

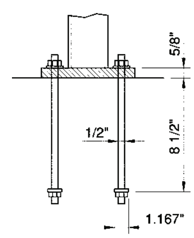
Concrete breakout		Steel strength		Applied load	
c_{a1}	$0.85V_{cbg,1}$	$V_{sa,1}$	$\phi V_{sa,2}$	$V_{ua,1}$	$V_{ua,2}$
18 in.	22.4 kips	20.5 kips	15.4 kips	7.2 kips	0 kips

Ductile—OK for strength.



Mode 3

	DESIGN PROCEDURE	CALCULATION
STEP 6 (cont.)		
D.6.2	**Mode 1:** Because the plate is not rigidly connected to the anchor, check the anchors nearest to the edge using 1/2 the applied shear.	$V_{cbg} = (A_{Vc}/A_{Vco})\psi_{ec,V}\psi_{ed,V}\psi_{c,V}V_b$ (D-21)
	Therefore, c_{a1} = 18 in.	
	Note that the depth of A_{Vc} is limited by h_a.	$A_{Vc} = (3c_{a1} + s)(h_a)$ $= (3 \times 18 + 5)(18) = 1062$ in.^{2}
		$A_{Vco} = 4.5c_{a1}^2 = 4.5 \times 18^2 = 1458$ in.^{2} (D-22)
		$A_{Vc}/A_{Vco} = 1062/1458 = 0.73$
D.6.2.5 D.6.2.6	Eccentricity on anchor group $\psi_{ec,V}$ Second edge effect $\psi_{ed,V}$	$\psi_{ec,V} = 1.0$ (no eccentricity) $\psi_{ed,V} = 1.0$ $c_{a2} > 1.5c_{a1}$ (D-26)
D.6.2.7	Second edge effect, $\psi_{c,V}$ assumed cracked.	$\psi_{c,V} = 1.0$
	Note: By definition, ℓ_e is limited to $8d_o$ = 4 in.	$V_b = 7(\ell_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f_c'}(c_{a1})^{1.5}$ (D-23)
		$V_b = 7(4/0.5)^{0.2}\sqrt{0.5}\sqrt{4000}(18)^{1.5}$ V_b = 36.2 kips
		$V_{cbg} = 0.73 \times 1.0 \times 1.0 \times 1.0 \times 36.2$
		V_{cbg} = 26.4 kips
	Ductility check:	$0.85V_{cbg} = 0.85(26.4)$ $= 22.4 > V_{sa} = 20.5$ kips
		Ductile, OK
	Strength: From Step 1, V_{sa} = 40.9/2 = 20.5 kips (two anchors)	$V_{sa} = 0.75 \times 20.5 = 15.40$ kips
D.6.2	**Mode 2:**	$V_{cbg} = (A_{Vc}/A_{Vco})\psi_{ec,V}\psi_{ed,V}\psi_{c,V}V_b$ (D-21)
	Next, check against failure at the back anchor under the full shear. Therefore, c_{a1} = 23. Note that the depth of A_{Vc} is limited by h_a.	$A_{Vc} = (3c_{a1} + s)(h_a)$ $= (3 \times 23 + 5)(18) = 1332$ in.^{2}
		$A_{Vco} = 4.5c_{a1}^2 = 4.5 \times 23^2 = 2381$ in.^{2} (D-22)
		$A_{Vc}/A_{Vco} = 0.56$
D.6.2.5 D.6.2.6	Eccentricity on anchor group $\psi_{ec,V}$ Second edge effect $\psi_{ed,V}$	$\psi_{ec,V} = 1.0$ (no eccentricity) $\psi_{ed,V} = 1.0$ $c_{a2} > 1.5c_{a1}$ (D-26)
D.6.2.7	Second edge effect $\psi_{c,V}$ assumed cracked.	$\psi_{c,V} = 1.0$
	Note: By definition, ℓ_e is limited to $8d_o$ = 4 in.	$V_b = 7(\ell_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f_c'}(c_{a1})^{1.5}$ (D-23)
		$V_b = 7(4/0.5)^{0.2}\sqrt{0.5}\sqrt{4000}(23)^{1.5}$ V_b = 52.3 kips
		$V_{cbg} = 0.56 \times 1.0 \times 1.0 \times 1.0 \times 52.3$
		V_{cbg} = 29.2 kips
	Ductility check:	$0.85V_{cbg} = 0.85(29.2)$ $= 24.9 > V_{sa} = 20.5$ kips
		Ductile, OK
	Strength: From Step 1, V_{sa} = 40.9/2 = 20.5 kips (two anchors)	$\phi V_{sa} = 0.75 \times 20.5 = 15.4$ kips

	DESIGN PROCEDURE	CALCULATION
STEP 6 (cont.)		
	Note to reader: See Problem B2(a) for discussion of the assumptions regarding the distribution of shear stresses in the steel anchors in a Mode 2 failure. **Mode 3** assumption: An alternate assumption is that all of the shear is taken by only the anchors on the compression side. With this assumption, there is no interaction check because the anchors in tension are not in shear and the anchors in shear are not in tension. This approach, however, requires that Mode 3 failure for the concrete shear breakout strength (as explained in the beginning of this Step 6) be checked. Mode 3 will have lower concrete breakout strength and is more likely to lead to a nonductile design, although in this particular example, it is ductile.	
Step 7: Check group pryout.		
D.6.3	Concrete pryout of the anchors in shear must be checked: Mode 1 is checked herein; Mode 2 can be similarly checked. (Note: the Code states that N_{cbg} is taken from Eq. (D-5).)	$V_{cpg} = k_{cp} \times N_{cbg}$ (D-29) k_{cp} = 2 for h_{ef} > 2.5 in. N_{cbg} = 45.1 kips V_{cpg} = 2 × 45.1 = 90.2 kips ϕV_{cpg} = 0.75 × 90.2 = 67.7 kips > 12.4 kips OK
STEP 8: Summary		
	Rod diameter d_o = 1/2 in. Plate thickness t = 5/8 in. Effective length h = 8-1/2 in.	
	TENSION	
Step 1	Applied load	N_{ua} = 17.5 kips (from applied moment)
Step 3 D.4.5.a	Steel strength	ϕN_{sa} = 0.8 × 34.1 = 27.3 kips
Step 3 D.4.5.c	Concrete breakout strength	ϕN_{cbg} = 0.75 × 45.1 = 33.8 kips
Step 4 D.4.5.c	Concrete pullout strength	ϕN_{pn} = 0.75 × 55.9 = 41.9 kips
Step 5 D.4.5.c	Concrete side-face blowout strength	$c_{a1} > 0.4h_{ef}$ so this is not applicable
D.4.1.2	Design strength of stud in tension	$\phi N_n = \min(\phi N_{sa}, \phi N_{cbg}, \phi N_{pn})$ = min(27.3, 33.8, 41.9) = 27.3 kips > N_{ua} = 17.5 kips OK
	SHEAR: Mode 2 (frictional resistance considered)	
Step 1 D.4.5.a	Applied load	V_{ua} = 12.4 – 5.25 = 7.2 kips
Step 6 D.4.5.c	Steel strength, two anchors	ϕV_{sa} = 0.75 × 20.5 = 15.4 kips
Step 7 D.4.5.c	Concrete breakout strength (nonductile)	ϕV_{cbg} = 0.75 × 29.2 = 21.9 kips
	Concrete pryout strength	ϕV_{cpg} = 0.75 × 90.2 = 67.7 kips
D.4.1.2	Design strength of stud in shear	$\phi V_n = \min(\phi V_{sa}, \phi V_{cbg}, \phi V_{cpg})$ = min(15.4, 21.9, 67.7) = 15.4 kips > V_{ua} = 7.2 kips OK

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 9: Check for tension-shear interaction.		
D.7.3	Tension-shear interaction, Mode 2.	17.5/27.3 + 7.2/15.4 = 1.11 < 1.2 OK

*__Notes on steel design:__ The plate will be designed using the AISC-LRFD Code (American Institute of Steel Construction [1999] "Load Resistance Factor Design for Structural Steel Buildings," AISC, Chicago, IL). In applying it to this example, some conservative simplifying assumptions will be made:
a) *Loads*: This example assumes that the loads are the same as used in the previous editions of the ACI 349 and therefore, the Appendix C ϕ-factors were used. The AISC Code uses the ASCE 7 (American Society of Civil Engineers, "Minimum Design Loads for Buildings and Other Structures," Reston, VA) load factors, and the strength reduction factors are determined accordingly. The loads used in this example are therefore conservative, and will be used with the LRFD design; b) *Strength reduction factors*: The strength reduction factors will be those of the AISC-LFRD Code (ϕ – 0.9 for bending); and c) *Strength design*: The nominal strength of a section in bending in the LRFD Code is based on a plastic section modulus Z and yield strength F_y of the steel material ($M_n = M_p = ZF_y$). This approach will be used in this example.

†AISC recommends oversizing holes for base plates. AISC *Design Guide 1*, 2nd Edition, "Base Plate and Anchor Rod Design," provides guidance for recommended rod hole size. In cases such as this, it is possible to have the anchors closest to the edge make contact with the base plate before the back anchors contact. The resulting breakout cone shown in Mode 3 would need to be evaluated.

Example B3—Four-threaded anchors and surface-mounted plate, combined axial, shear, and moment

Design a group of four-threaded headed anchors to resist seismic loads given as follows. The supported member is a W10 x 15 stub column. Design parameters are provided as follows.

Given:

Concrete edges
- c_{a1} = 18 in.
- c_{a2} > 24 in.
- h_a = 18 in.

Base plate
- 12 x 12 in.

Bolt spacing
- s = 8 in.

Concrete
- f_c' = 4000 psi (concrete)
- f_c' = 9000 psi (grout)

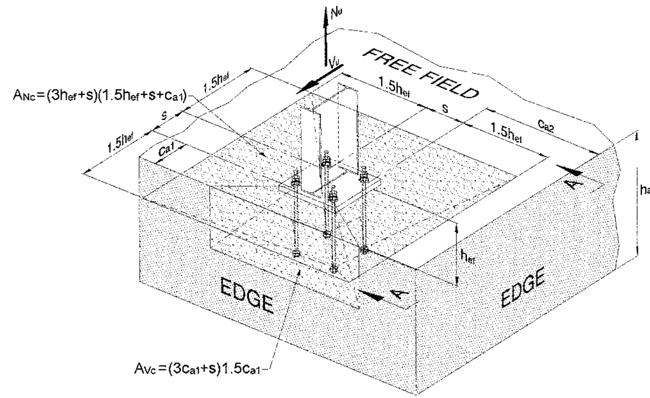
Bolt material (F 1554 Gr. 36 anchor rods[*])
- f_{ya} = 36 ksi
- f_{uta} = 58 ksi

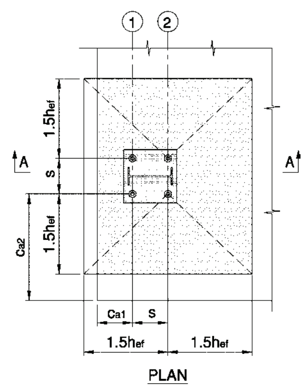
Plate
- F_y = 36 ksi

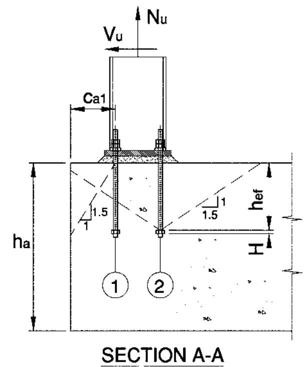
Loads
- V_{ua} = ±7 kips
- e = 18 in. (height of stub column above concrete surface)
- $M_u = V_{ua} \times e$ = 126.0 in.-kips
- P_u = 3.0 kips

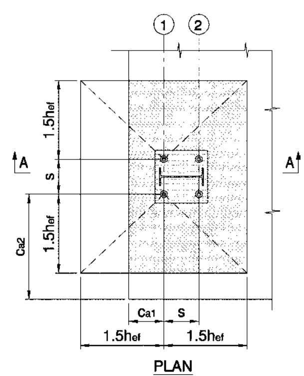
Where M_u, P_u, and V_{ua} are the required factored external loads using load factors from Chapter 9 of the Code.

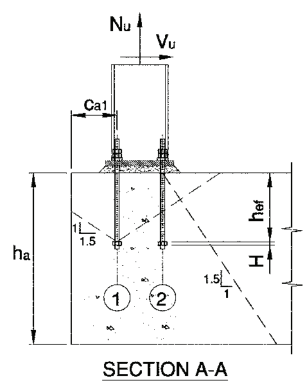
Assumptions:
- Concrete is cracked.
- ϕ-factors are based on Condition B in D.4.4 of the Code (no supplementary reinforcement).
- Ductile embedment design is in accordance with D.3.6.1.



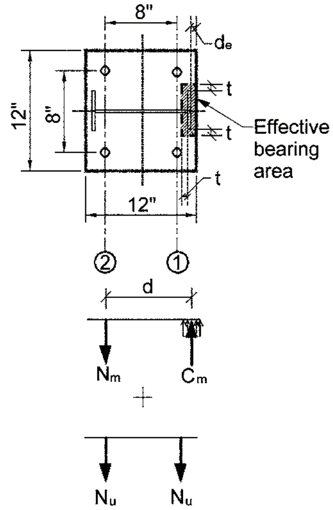








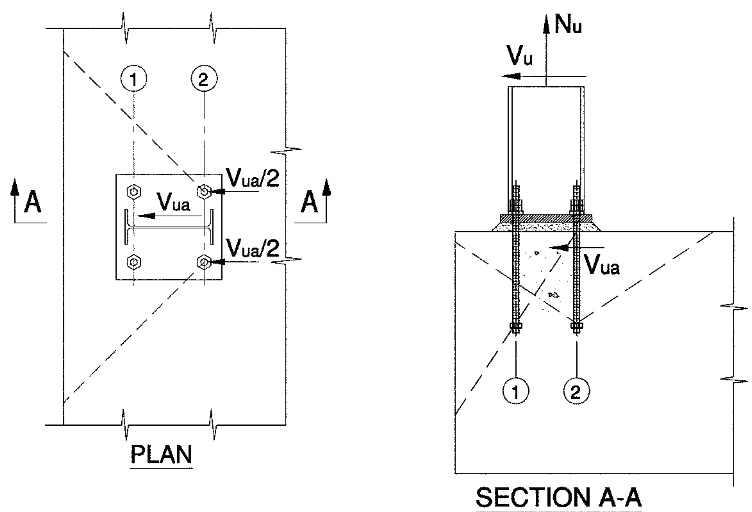
*Anchor material is ASTM F 1554 Gr. 36. It has a tensile elongation of 23%, reduction in area of 40%, and meets the definition of a ductile steel element given in D.1 (f_{uta} = 58 ksi < $1.9f_{ya}$ = 1.9 × 36 = 64 ksi).

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 1: Design for moment and tension.		
AISC	The plate size is 12 x 12 in. and the spacing *s* of the anchors is 8 x 8 in. Assume plate thickness *t* is 1 in. The first step is to calculate the tension force in the anchors and compression reaction force in the concrete from the applied forces. The base plate is just large enough in area to accommodate the column profile, that is, small base plate (per AISC). For the shear toward the edge of the slab, the resultant tension from the moment is taken by the two left-hand side (Line 2) bolts, and the compression is taken in bearing on the effective bearing area. The bearing area is taken to the edge of the plate. The effective bearing area is taken as a distance *t* around the compression flange. This is an approximation. The error introduced into the calculation is negligible. The exact location of the compression resultant is difficult to determine. For design, take the resultant to act at the outside edge of the compression flange. The approximate analysis is deemed adequate.	
	Definite the moment arm *d*.	
	Determine the tension in the bolts and compression in the concrete.	$d_e = \dfrac{12\text{ in.} - 10\text{ in.}}{2} = 1.0\text{ in.}$ $d = 12\text{ in.} - d_e - \dfrac{12\text{ in.} - 8\text{ in.}}{2}$ $d = 9$ in.
10.17.1 9.3.2.5	Check the effective bearing area based on the location of the resultant compression assumed previously. The bearing capacity is as given in the noted Code section. An assumption has been made in this example that the grout under the plate, though unconfined, does not control because it is at least 9000 psi in compressive strength. Check the concrete instead. The ϕ-factors are as given in Chapter 9. ϕ is 0.65 for bearing. Due to confinement, use the maximum allowed factor $\sqrt{A_2/A_1}$, 2.	$N_{ua,M} = M_u/d$ $N_{ua,M}$ = 126 in.-kips/9 in. $N_{ua,M}$ = 14 kips $C_m = N_{ua,M}$ $C_m = 2\phi(0.85f_c')A_{brg,pl}$ $A_{brg,pl} = (b_f + 2t)(t_f + t + d_e)$ $A_{brg,pl} = (3.96 + 2)(0.27 + 1 + 1)$ $A_{brg,pl} = 13.53\text{ in.}^2$ $C_m \leq 2(0.65)(0.85 \times 4\text{ ksi})A_{brg,pl}$ $C_m \leq 2(0.65)(0.85 \times 4\text{ ksi})13.53\text{ in.}^2$ $C_m \leq 59.8$ kips 14 kips $\leq$ 59.8 kips OK
D.4.4	The effect of tension force in shifting the location of the compression resultant is deemed negligible and, hence, it is conservative to algebraically add the bolt force distribution from moment to that from tension. P_u is the tension in the columns, and $N_{au,2}$ is the tension in the two bolts. Likewise, the compression on the bearing area can be reduced directly by the tension force even though forces are centered at different locations. ϕ is 0.75 for tension strength in steel.	$N_{ua,2} = N_{ua,M} + P_u/2$ $N_{ua,2}$ = 14 kips + (3 kips/2) (in two bolts) $N_{ua,2}$ = 15.5 kips $C_F = C_m - P_u/2$ C_F = 14 kips – (3 kips/2) (on bearing area) C_F = 12.5 kips $N_{ua,2} = \phi N_{sa}$ $N_{ua,2} = \phi_n A_{se,t} f_{ut,a}$ (D-3)

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 1 (cont.)		
D.5.1.2	Determine the required bolt area $A_{s,et,req}$ for tension. F 1554 Gr. 36 is a ductile steel element. The ϕ-factors are as given in D.4.4. Note that $A_{s,et,req}$ is the effective tensile area required, and n is the number of bolts resisting the tension force. Note that the bolts resist shear, as well; hence, the margin in area of bolt provided ($A_{s,et}$). Refer to Step 2 for consideration of shear. Net tensile area of threaded bolts can be found in Table 2 of Appendix A.	$A_{s,et,req} = N_{u,2}/\phi_n f_{uta}$ $A_{s,et,req} = 15.5/0.75(2)58$ $A_{s,et,req} = 0.18$ in.^{2} **Use 3/4 in. diameter threaded rods.** $A_D = 0.44$ in.^{2} (nominal area) $A_{se,t} = 0.334$ in.^{2} (effective area) $\phi N_{sa} = \phi A_{se,t} n f_{uta}$ (D-3) $\phi N_{sa} = 0.75(0.334)(2)58$ $\phi N_{sa} = 29.06$ (two bolts) $\phi N_{sa} > N_{ua,2}$ 29.06 kips > 15.5 kips OK

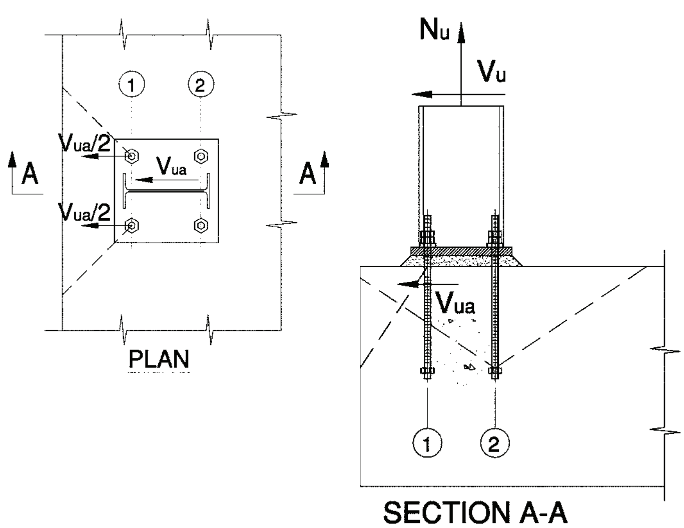
STEP 2: Design for shear.

Concrete breakout modes (options) that will be used in design.



Note: Due to the reversibility of seismic loads, both cases of shear toward the edge and shear away from the edge will be considered.

a) Shear resisted by two anchors farthest from edge:

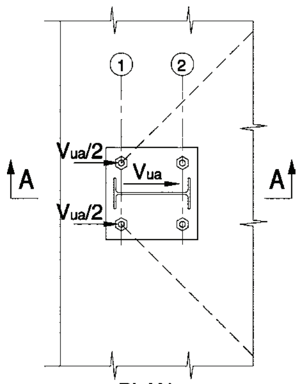


CODE SECTION	DESIGN PROCEDURE	CALCULATION

STEP 2 (cont.)

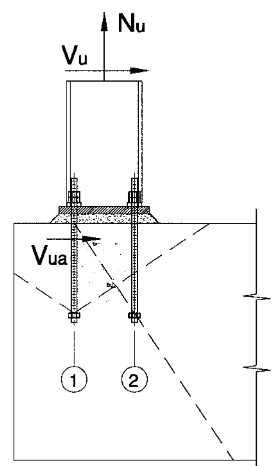
b) Shear resisted by two anchors closest to the edge:

SHEAR FORCE TOWARD EDGE



PLAN

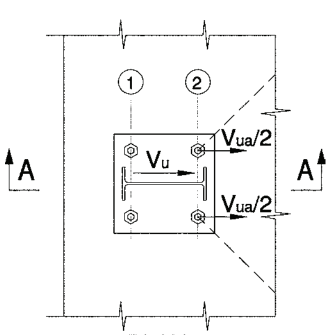
c) Shear resisted by two anchors closets to edge:



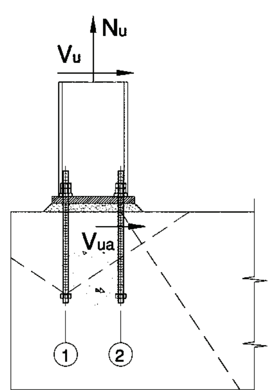
SECTION A-A

d) Shear resisted by two anchors farthest from edge

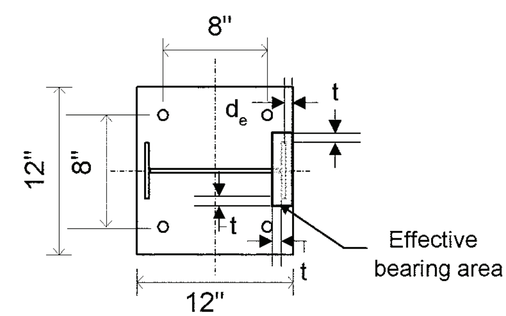
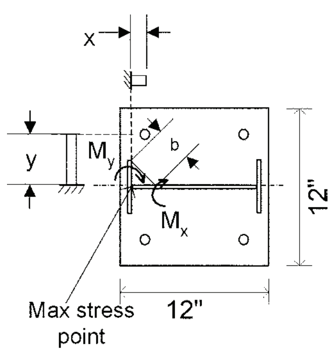
SHEAR FORCE AWAY FROM EDGE

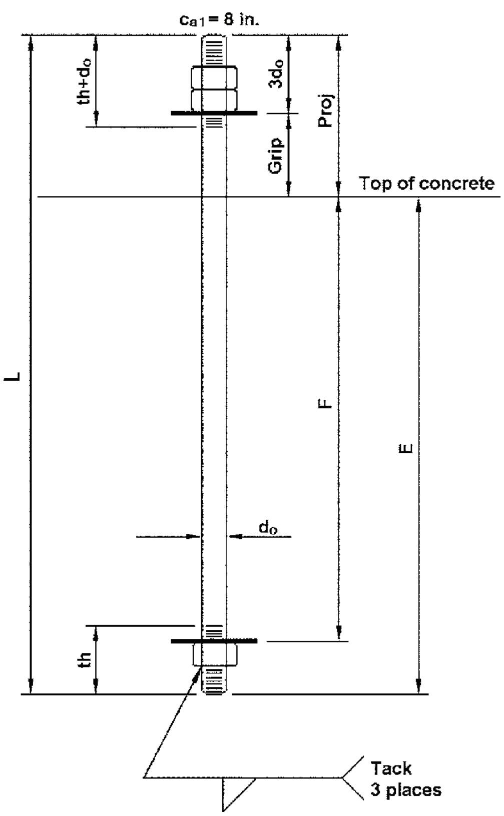


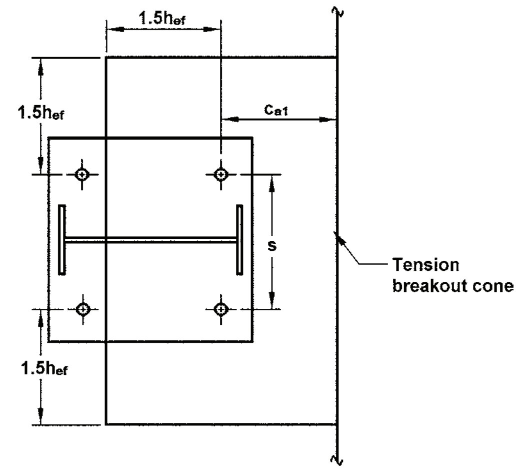
PLAN

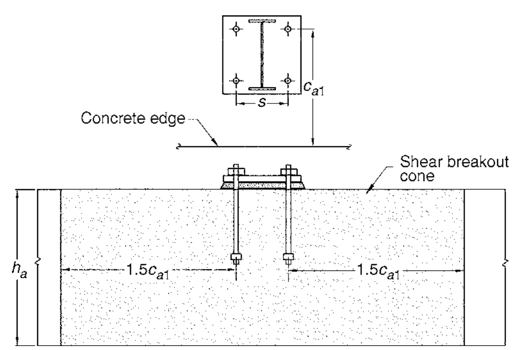


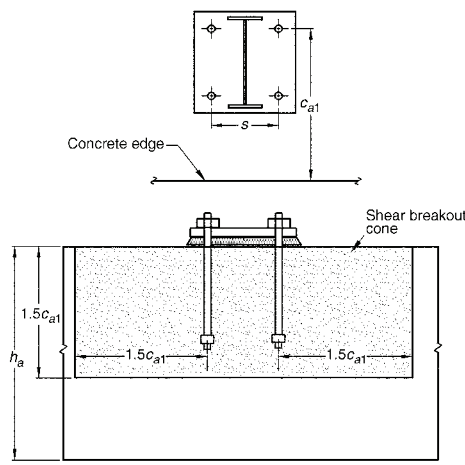
SECTION A-A

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 2 (cont.)		
D.6.1.2	This is a base plate on grout, therefore Section D.6.1 is applicable. Note that surface-mounted plates with grout often come with oversized bolt holes.* Therefore, in this example, it will be assumed that only two anchors are engaged in resisting shear. There are two options for analysis. The first is to assume the bolts in tension also carry shear and compute the available shear strength from steel based on that assumption. Alternatively, assume that only the bolts in compression take the shear and compute the shear strength based on that assumption. Both will be checked in the solution presented.	
D.6.1.2 D.6.1.3	**Compute the steel shear capacity:** In Options (a) and (c), the bolts in tension also resist shear. This is critical case for steel strength of anchor. Note that the 0.8 factor for grout (D.6.1.3) applies to Eq. (D-19)	Options (a) and (c) $V_{ua} \leq \phi V_n$ $\phi V_n = \phi(0.8)V_{sa} + \phi(0.4C_F)$ $\phi V_n = \phi(0.8)0.6nA_{se}f_{uta} + \phi(0.4C_F)$ (D-19) $\phi(0.8)0.6nA_{se}f_{uta} = 0.65(0.8)0.6(2)(0.334)58$ $\phi(0.8)0.6nA_{se}f_{uta} = 12.09$ kips $\phi(0.4C_F) = 0.70(0.4(12.5)) = 3.50$ kips $V_{ua} \leq \phi(0.8)0.6nA_{se}f_{uta} + \phi(0.4C_F)$ (D-19) $7 \leq 12.09 + 3.50$ $7 \leq 15.39$ kips OK
STEP 3: Design for base plate.		
	The plate thickness was assumed at the beginning of the example problem to be 1 in. To check the required plate thickness, there are two possible failure modes: 1. Yielding of the plate in the tension region around the two tension bolts. 2. Yielding of the plate in the compression region. Pryout of the bolts in the tension region is ignored. Failure Mode 1: Tension yielding of the plate around the bolts in tension. Plate bending approximation: Assume that the plate is fixed along the web and the flange of the wide flange shape in tension and that the plate acts as a cantilever between the bolts and the web and flange of the wide flange. Also assume the effective width *b* of plate for stress computation is 2*t* each side of the point of maximum stress. Therefore, $b_{eff} = 4t$, where *t* is the thickness of the plate. This approximation is conservative because it maximizes the moment arm for moment computation and minimizes the effective width of the plate resisting this moment. It also ignores the clamping effect at the bolt location.	 $N_{ua,bolt} = 15.5/2 = 7.8$ kips per bolt  $x = (b - s)/2 - d_e - t_f$ $x = (12 \text{ in.} - 8 \text{ in.})/2 - 1 \text{ in.} - 0.27 \text{ in.} = 0.73$ in. $y = (s/2) - (t_w/2)$ $y = 8 \text{ in.}/2 - 0.115 \text{ in.} = 3.88$ in. $b_{eff} = 4t = 4(1 \text{ in.}) = 4$ in. $M_y = N_{ua,bolt}(x) = 7.8(0.73 \text{ in.})$ $M_y = 5.7$ kip-in. $M_x = N_{ua,bolt}(y) = 7.8(3.88 \text{ in.})$ $M_x = 30.1$ kip-in.

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 3: Design for base plate.		
		$M_u = \sqrt{M_x^2 + M_y^2} = \sqrt{5.7^2 + 30.1^2}$ M_u = 30.6 kip-in. $\phi M_n = 0.9F_yZ = 0.9F_y(b_{eff}t^2/4)$ $\phi M_n = 0.9(36\text{ ksi})(4(1)^2/4) = 32.4$ kips $\phi M_n \geq M_u$ 34.2 kip.-in. ≥ 30.6 kip-in. OK **Use 1 in. thick plate.**
	Failure mode 2:‡ The compression is acting in the effective bearing area. The bearing area is taken as fixed along the column web and compression flange. The maximum cantilever distance of the area loaded in bearing relative to the fixed axis is the maximum of t or d_e. The bearing area can be taken to be under a uniform pressure equivalent to $C_F/A_{brg,pl}$.	C_F = 12.5 kips $t = \max(t,d_e)\sqrt{\dfrac{2C_F}{0.9F_yA_{brg,pl}}}$ $t = 1.0\sqrt{\dfrac{2(12.5)}{0.9(36)13.53}}$ t = 0.24 in. < 1.0 in. OK
STEP 4: Determine required embedment length for the bolts to prevent concrete breakout failure.		
D.5.2	The required embedment length for the anchors will be determined using concrete breakout strength of the anchors in tension. Section D.5.2 applies. Steel strength remains as previously calculated. The shear force is seismic and, hence, has a reversible direction. The critical tension breakout cone is that closest to the edge. This is depicted in Options (c) and (d) in Step 2. If the steel strength is less than 85% of the nominal concrete tension breakout strength, then the connection will be ductile for tension load; otherwise, it is nonductile. Notations are consistent with Section RD.5.2.1.	$N_{sa} = 0.334 \times 2 \times 58 = 38.7$ kips $N_{ua,2}$ = 15.5 kips $N_{sa} < 0.85N_{cbg}$ (for steel to control) $N_{cbg,req}$ > 38.7 kips/0.85 = 45.5 kips
D.4.4	Assume Condition B exists. Also assume that the anchors use standard A563 nut and a F436 washer at the end of the anchors to serve as the anchor head. Assume embedment depth h_{ef} (F in sketch) = 12 in. Bolt dimensions: d_o = 3/4 in. th = 2 in. E = 13.25 in. L = 18.5 in. (for a double nut)	

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 4 (cont.)		
		h_{ef} = 12 in. $N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}N_b$ (D-5)  $A_{Nc} = (1.5h_{ef} + c_{a1})(1.5h_{ef} + s_1 + 1.5h_{ef})$ c_{a1} = 8 in. A_{Nc} = (1.5(12) + 8)(1.5(12) + 8 + 1.5(12)) = 1144 in.^{2} $A_{Nco} = 9h_{ef}^2 = 9(12)^2$ = 1296 in.^{2} A_{Nc} = 1296 in.$^2 \le nA_{Nco}$ = 2(1296) = 2592 in.^{2} OK
 D.5.2.4 D.5.2.5 D.5.2.6 D.5.2.7	Modification factors are computed for: Eccentricity effects $\psi_{ec,N}$ Edge effects $\psi_{ed,N}$ Concrete cracking $\psi_{c,N}$. Post-installed anchor effect $\psi_{cp,N}$ (Not required in this case.)	$\psi_{ec,N}$ = 1 (no eccentricity) $\psi_{ed,N} = 0.7 + 0.3(c_{a,min}/1.5h_{ef})$ (D-11) $\psi_{ed,N}$ = 0.7 + 0.3(8/18) = 0.83 $\psi_{c,N}$ = 1.0 (cracked concrete)
D.5.2.1	Basic concrete breakout strength N_b of a single anchor in tension in cracked concrete.	$N_b = 16\sqrt{f'_c}\ h_{ef}^{5/3}$ 11 in. < h_{ef} < 25 in. (D-8) $N_b = 16\sqrt{4000}\ (12)^{5/3}$ = 63.65 kips N_{cbg} = (1144/1296)(1.0)(0.833)(1.0)(1.0)63.65
	Tensile strength controlled by steel.	N_{cbg} = 46.8 kips > (N_{sa}/0.85) = 45.5 kips
	Check strength:	Ductile. OK.
D.4.4	ϕ is 0.70 for tension strength in concrete against tension breakout.	Embedment is ductile for tension
	Tension-shear interaction remains as computed in Step 9.	ϕN_{cbg} = 0.7(46.80 kips) $\ge \phi N_{sa}$ ϕN_{cbg} = 32.76 kips > 29.06 kips **Use four rods 3/4 in. diameter x 18.5 in. long with h_{ef} = 12 in.**
STEP 5: Check pullout strength of anchor.		
D.5.3	Calculate the pullout strength of the anchor in tension in accordance with D.5.3.	$N_{pn} = \psi_{c,P}N_p$ (D-14) $N_p = 8A_{brg}f'_c$ (D-15)
D.5.2.8 D.5.3.4 D.5.3.5	To obtain a ductile design, it can be shown that either a heavy hex nut or washer is required. For this example problem, a washer is provided.	
	Width across the flat for an A563 hex nut is 1.125 in. Use the outside diameter (OD) of the washer and the diameter of the anchor, d_o, to compute the bearing area of the head.	$\psi_{c,P}$ = 1.0 (cracked concrete) OD = min(1.468, 1.125 + 2t_w) OD = min(1.468, 1.125 + 2(0.136)) = 1.4
D.3.6.1	The OD of a F463 (U.S. dimensions) circular washer is 1.468 in. for a 3/4 in. bolt. From D.5.2.8, check that the OD is less than diameter for the nut plus two time thickness of washer. Pullout capacity to maintain ductile design in accordance with D.3.6.1. Number of bolts in tension is used in the computation of pullout.	$A_{brg} = \pi\dfrac{(\text{OD}^2 - d_o^2)}{4}$ $A_{brg} = \pi\dfrac{(1.4^2 - 0.75^2)}{4}$ = 1.1 in.^{2}

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 5 (cont.)		
D.4.4	ϕ is 0.70 for pullout strength and 0.75 for tension strength of steel.	$N_{pn} = \psi_{c,P} A_{brg} 8 f_c'$ N_{pn} = 1.0(1.1)8(4000) = 35.2 kips $nN_{pn} > N_{sa}/0.85$ (for ductile response) nN_{pn} = 2(35.2) = 70.4 kips > 45.5 kips **Ductile, OK** ϕN_{png} = 0.7(70.4) = 49.3 kips 49.3 kips > ϕN_{sa} = 29.06 kips **3/4 in. diameter bolts are OK for pullout.**
STEP 6: Check concrete side-face blowout.		
D.5.4	Check if side-face blowout needs to be investigated using the Code limits given in Section D.5.4.	c_{a1} = 8 in. > $0.4h_{ef}$ = 0.4(12) = 4.8 in. **OK. Ignore side-face blowout.**
STEP 7: Check concrete shear breakout.		
D.6.2	Because the base plate if not rigidly attached to the anchor bolts, two shear failure cones need to be checked. Note that these two shear breakout cones need to be checked even if all the bolts resist shear. This is done to prevent the zipper effect in which the concrete supporting the two bolts closest to the edge fails first and causes the failure of the concrete around the two bolts farther from the edge.	$V_{sa} = 0.6nA_{se}f_{uta}$ V_{sa} = 0.6(2)(0.334)(58) V_{sa} = 23.2 kips (two bolts) $V_{cbg,req} \geq$ 23.2 kips/0.85 = 27.3 kips (two bolts)
D.6.2.1 D.6.2.2	**Two bolts failure cone: Option (a)** Note: For Option (a) where the shear is toward the edge and tension bolts resist shear, $c_{a1} = c_{a1} + s$ = 8 + 8 = 16 in. Section D.6.2.2 applies. Check for slab depth limitations. h_a = 16 in. < $1.5c_{a1}$ = 1.5 × 16 = 24 in. Limit cone depth to 16 in.	
	Compute areas. All factors are set to 1 because there isn't any eccentricity for the shear load, no perpendicular edge effects, and concrete is cracked. Shear strength is controlled by steel for Option (a) where the shear load is taken by the bolts farthest from the edge.	$V_{cbg} = (A_{Vc}/A_{Vco})\psi_{ec,V}\psi_{ed,V}\psi_{c,V}V_b$ (D-21) $V_b = 7(\ell_e/d_o)^{0.2}\sqrt{d_o}\sqrt{f_c'}\,c_{a1}^{1.5}$ (D-23) $A_{Vc} = (2(1.5c_{a1}) + s_1)h_a$ A_{Vc} = (2(1.5{16} + 8){16} = 896 in.^{2} $A_{Vco} = 4.5c_{a1}^2$ = 4.5{16}^{2} = 1152 in.^{2} $\psi_{ec,V}$ = 1.0 $\psi_{ed,V}$ = 1.0 $\psi_{c,V}$ = 1.0 $\ell_e = \min(h_{ef}, 8d_o)$ = min(12,8{0.75}) ℓ_e = min(12,6) = 6 in. $V_b = 7(6/0.75)^{0.2}\sqrt{0.75}\sqrt{4000}\,(16)^{1.5}$ V_b = 37.2 kips $V_{cbg} = \dfrac{896}{1152}(1.0)(1.0)(1.0)37.2$ V_{cbg} = 28.9 kips > 27.3 kips **Ductile**

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 7 (cont.)		
D.6.2.1 D.6.2.2	**Two bolts failure cone: Option (b)** c_{a1} = 8 in.with shear toward the edge and compression bolts loaded in shear. Compute areas. Check for slab depth limitations. h_a = 16 in. > $1.5c_{a1}$ = 12 in. Cone depth OK.	 c_{a1} = 8 in. $A_{Vc} = (2(1.5c_{a1}) + s_1)1.5\{c_{a1}\}$ $A_{Vc} = (2(1.5\{8\} + 8)12 = 384$ in.^{2} $A_{Vco} = 4.5c_{a1}^2 = 4.5\{8\}^2 = 288$ in.^{2} $V_b = 7(6/0.75)^{0.2}\sqrt{0.75}\sqrt{4000}\,(8)^{1.5}$ V_b = 13.2 kips $V_{cbg} = (384/288) \times 1.0 \times 1.0 \times 1.0 \times 13.2$ V_{cbg} = 17.5 kips < 27.3 kips **Nonductile** $\phi V_{cbg} = \phi(0.6)V_{cbg}$ ϕV_{cbg} = 0.7(0.6)17.5 kips ϕV_{cbg} = 7.4 kips > V_{ua} = 7.0 kips **OK**
STEP 8: Check concrete shear pryout.		
D.6.3 D.6.3.1	For pryout, check the tension cone for the two bolts closest to the edge. Note N_{cbg} computed in Step 4. Two bolts failure cone: Option (c): From Section D.6.3.1, k_{cp} is 2 because h_{ef} > 2.5 in. **This concludes the checks for the connection design. This connection is a ductile design for tension, and nonductile for shear.**	N_{cbg} = 46.8 kips (two bolts) $V_{cpg} = k_{cp}N_{cbg}$ (D-28) V_{cpg} = 2(46.8) = 93.6 kips > 27.3 kips **OK for pryout, but because nonductile in shear, apply the 0.6 penalty.** $\phi V_{cpg} = \phi(0.6)V_{cbg}$ ϕV_{cpg} = 0.7(0.6)93.6 kips ϕV_{cpg} = 39.3 kips > V_{ua} = 7.0 kips
STEP 9: Check for tension-shear interaction.		
D.7 D.7.3	Note that when concrete failure controls (that is, nonductile failure), there is no interaction between tension and shear (see Step 7). The tension-shear interaction is therefore checked on the steel strength and loads on two bolts. $N_{ua,2}$ is the tension taken by two bolts. Note that, from observation, Section D.7.1 and D.7.2 do not govern. Also note that the friction between the base plate may be used to directly reduce the shear load on the connections, or may be neglected, preferably for a new design interaction ratio considering both scenarios presented.	V_{ua} = 7.0 kips (applied load) V_{ua} = 7.0 kips – 3.5 kips = 3.5 kips (if applied load is reduced by friction) $N_{ua,2}$ = 15.5 kips ϕV_n = 15.39 kips (including friction) ϕV_n = 12.09 kips (excluding friction) $\phi N_n = \phi N_{sa}$ = 29.06 kips $(N_{ua,2}/\phi N_n) + (V_{ua}/\phi V_n) = (15.5/29.06) + (7/12.09) = 1.11 < 1.20$ OK for the case where friction is not considered. $(N_{ua,2}/\phi N_n) + (V_{ua}/\phi V_n) = (15.5/29.06) + (3.5/12.09) = 0.82 < 1.20$ OK for the case where friction is considered.

	DESIGN PROCEDURE	CALCULATION
STEP 10: Summary		
Given	Applied load on the embed attachment:	V_{ua} = 7 kips e = 18 in. (height of stub column above surface of concrete) $M_u = V_{ua} \times e$ = 126.0 in.-kips P_u = 3.0 kips
	Resulting applied load on critical two bolts:	$N_{ua,2}$ = 15.5 kips V_{ua} = 7 kips
Step 1	Design steel tensile strength	ϕN_{sa} = 29.06 kips (two bolts)
Step 4	Design concrete tension breakout strength	ϕN_{cbg} = 32.76 kips
Step 5	Design concrete tension pullout strength	$\phi n N_{pn}$ = 49.3 kips
Step 6	Concrete side-face blowout strength	ϕN_{sb} = N/A
Step 2 D.4.1.2	Design strength of stud in tension	$\phi N_n = \min(\phi N_{sa}, \phi N_{cbg}, \phi n N_{pn})$ $= \min(29.06, 32.76, 49.3)$ $= 29.06$ kips $> N_{ua,2} = 15.5$ kips OK
Step 7	Design steel shear strength (includes grout and compression effects)	ϕV_n = 15.39 kips
	Design steel shear strength (excludes compression effects)	ϕV_n = 12.09 kips
	Design concrete shear breakout strength (includes the 0.6 penalty because concrete controls)	ϕV_{cbg} = 7.4 kips
	Design concrete shear pryout strength (includes the 0.6 penalty because concrete control)	ϕV_{cpg} = 39.3 kips
D.4.1.2	Design strength of anchor, shear (conservatively considering the case that excludes compression effects)	$\phi V_n = \min(\phi V_n, \phi V_{cbg}, \phi V_{cpg})$ $= \min(12.09, 7.4, 39.3)$ $= 7.4$ kips $> V_{ub} = 7$ kips OK

[*]Recommended hole sizes for base plates taken from AISC *Design Guide 1*. (American Institute of Steel Construction, "Column Base Plates," *Design Guide 1*, AISC, Chicago, IL.)
[†]ϕ-factor for concrete shear breakout, specified in Section D.4.4c Condition B, is assumed to be convert nominal shear strength resulting from friction between the base plate and the concrete to design strength.
[‡]American Institute of Steel Construction, *AISC-LRFD Manual of Steel Construction*, Load Resistance Factor Design, V. 2, 2nd Edition, Chicago, IL, pp. 11-59.

Example B4(a)—Four-stud embedded plate in thin slab, tension only

Objective: Describe the additional cracks required to assure splitting failure does not occur.

Given:

Concrete edges

c_{a1} = 10 in.
c_{a2} = 10 in.
h_a = 7-1/2 in.
s_1, s_2 = 6 in.
d_o = 1/2 in.
h_{ef} = 5 in.

Slab reinforcement: No. 5 bars

Concrete
f_c' = 4000 psi

Stud material (A29/A108)*
f_{ya} = 51 ksi
f_{uta} = 65 ksi

Plate
F_y = 36 ksi

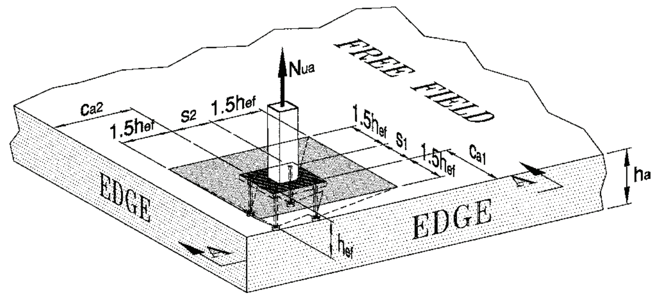
Load
N_{ua} = 18 kips

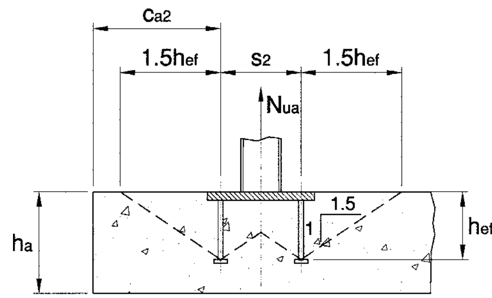
Where N_{ua} is the applied factored external load using load factors from Appendix C of the Code.

Assumptions:

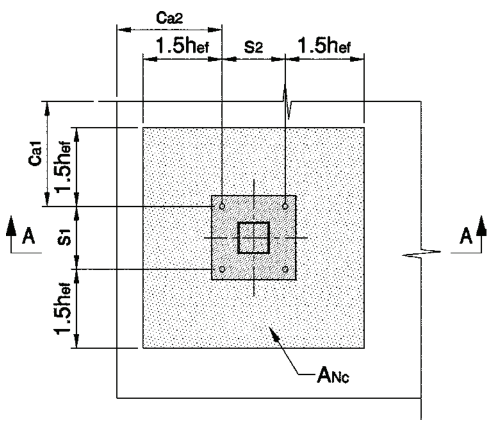
- Concrete is cracked.
- ϕ-factors are based on Condition B in D.4.5 of the Code (no supplementary reinforcement).

*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%, and 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: $f_{uta} \leq 1.9 f_{ya}$ (65 ≤ 1.9 × 51.0 = 96.9 ksi).





SECTION A-A



CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 1: Check the required spacing to preclude splitting failure.		
D.8.1	Minimum center-to-center spacing for cast-in anchors (anchors are not torqued)	$s \geq 4d_o$ d_o = 1/2 in. Given $4d_o = 4 \times (1/2)$ = 2.0 in. s = 6 in. > 2.0 in. OK
STEP 2: Check for minimum edge distance to preclude splitting failure.		
D.8.2 7.7	Minimum edge distance for cast-in anchors Minimum cover for No. 5 bar and smaller	$c_{a,min} = c_{a1} = c_{a2}$ = 10 in. Cover required = 1.5 in. $c_{a,min}$ = 10 in. > 1.5 in. OK
STEP 3: Check for minimum slab thickness to preclude splitting failure.		
D.8.5	"The value of h_{ef} for an expansion or undercut post-installed anchor shall not exceed the greater of either 2/3 of the member thickness or the member thickness less 4 in." No guidance is given for cast-in anchors. Therefore, assume no additional check is required for this example.	7-1/2 in. slab OK for cast-in stud
STEP 4: Summary.		
Step 1 D.8.1	Minimum spacing for cast-in anchors	s = 6 in. $\geq 4d_o = 4 \times 1/2$ = 2.0 in.
Step 2 D.8.2	Minimum edge distance	$c_{a,min}$ = 10 in.
Step 2 7.7	Minimum cover for No. 5 bar and smaller	Cover required = 1.5 in. < 10 in.
Step 3 D.8.5	h_{ef}	5 in.
Step 3 D.8.5	Minimum slab thickness: No Code requirement for cast-in anchors	

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

Given:

Concrete edges

$c_{a1} = c_{a2} = 10$ in.

$s_1 = s_2 = 6$ in.

$h_a =$ 7-1/2 in.

Concrete

$f_c' = 4000$ psi

Stud material (A29/A108)*

$f_{ya} = 51$ ksi

$f_{uta} = 65$ ksi

Plate

3 x 3 x 5/8 in. thick

$F_y = 36$ ksi

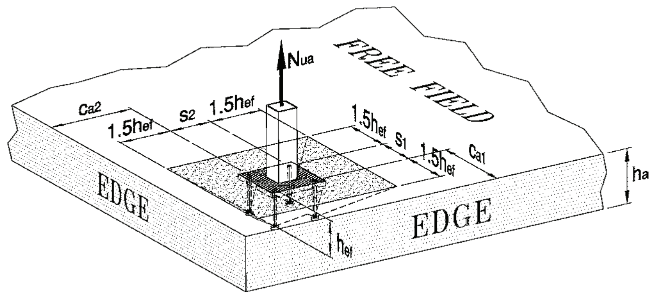
Load

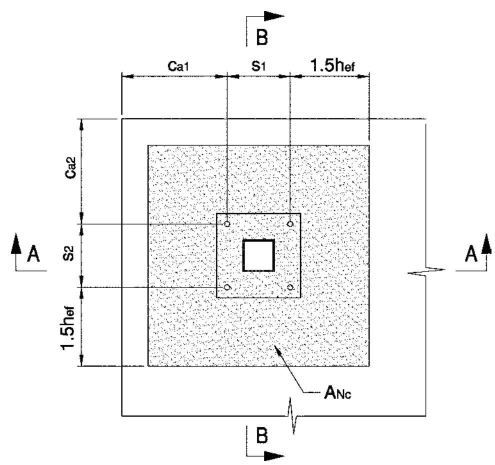
$N_{ua} = 18$ kips

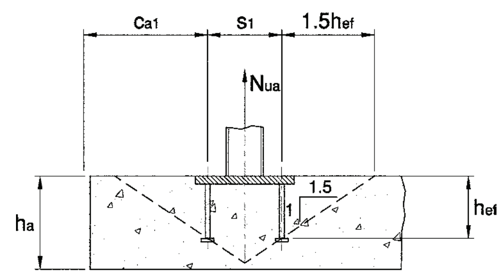
Where N_{ua} is the applied factored external load using load factors from Appendix C of the Code.

Assumptions:

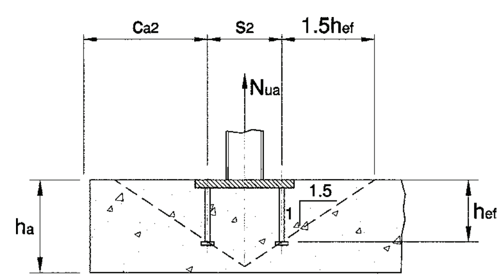
- Concrete is cracked.
- ϕ-factors are based on Condition B in D.4.5 of the Code (no supplementary reinforcement).







SECTION A-A



SECTION B-B

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*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%, and 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: $f_{uta} \le 1.9 f_{ya}$ ($65 \le 1.9 \times 51.0 = 96.9$ ksi).

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 1: Determine required steel area of the anchor.		
		Equation no.
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.	$\phi N_n \geq N_{ua}$ (D-1) $N_n = N_{sa} = nA_{se} f_{uta}$ (D-3) $N_{ua} = 18 = \phi nA_{se} f_{uta}$ $= 0.85 \times 4.0 \times A_{se} \times 65$ kips
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, ϕ = 0.85 for tension.	
	Calculate nominal steel strength of selected stud.	$A_{se} = N_{ua}/\phi nf_{uta}$ = 18/(0.85 × 4 × 65) min A_{se} = 0.081 in.^{2} (required)
	Anchor diameter, in. / Effective area A_{se}, in.^{2} 1/4 — 0.049 3/8 — 0.110	**Use four 3/8 in. diameter studs,** A_{se} = 0.110 in.^{2} > 0.0.81 in.^{2}
	Calculate the nominal steel strength N_{sa}.	$N_{sa} = nA_{se} f_{uta}$ (D-3) = 4.0 × 0.110 × 65 = 28.6 kips
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.)	$f_{uta} = 65 \leq 1.9f_{ya}$ or 125 ksi $\leq 1.9f_{ya}$ = 1.9 × 51 = 96.9 ksi OK
STEP 2: Determine required embedment length for the anchor to prevent concrete breakout failure in tension.		
D.5.2	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.	From Step 1: N_{sa} = 28.6 kips
D.3.6.1	The requirements for a ductile design are given in D.3.6.1. To prevent concrete breakout for tension load requires that $0.85N_{cbg} > N_{sa}$.	$0.85N_{cbg} = N_{sa}$ $N_{cbg} = N_{sa}/0.85$ = 28.6/0.85 = 33.65 kips
	Concrete breakout strength for an anchor group:	$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$ (D-5)
D.5.2.1	For a four-stud group away from edge, assume h_{ef} = 5.0 in., $A_{Nco} = 9nh_{ef}^2$ = 225 in.^{2}, A_{Nc} = [1.5(5) + 6 + 1.5(5)]^2 = 441 in.^{2}	A_{Nc}/A_{Nco} = 1.96
D.5.2.4 D.5.2.5 D.5.2.6 D.5.2.7	Modification factors for: Eccentricity factor $\psi_{ec,N}$ Edge effects $\psi_{ed,N}$ Concrete cracking $\psi_{c,N}$ Modified post installed $\psi_{cp,N}$	$\psi_{ec,N}$ = 1.0 $\psi_{ed,N}$ = 1.0 (D-10) $\psi_{c,N}$ = 1.4 $\psi_{cp,N}$ = N/A
D.5.2.2	k_c = 24 for post-installed anchors	$N_b = k_c\sqrt{f_c'}\,h_{ef}^{1.5}$ lb (D-7) $= 24\sqrt{(4000)}\,h_{ef}^{1.5}$ $= 1518h_{ef}^{1.5}$ lb $= 1.52h_{ef}^{1.5}$ kips
	Determine required embedment length* $h_{ef,req}$.	N_{cbg} = 33.65 kips 33.65 = 1.96 × 1.0 × 1.0 × 1.4 × 1.0 × $1.52h_{ef}^{1.5}$ $h_{ef,req}$ = 4.02 in.
Table 6, Appendix A		**Use 3/8 x 5 in. long stud.**
	Total length of a a stud, L, before weld, is equal to the embedment length plus the head thickness plus allowance for burnoff. Standard length and head dimensions are given by the manufacturer. Typical values are given in Table 6, Appendix A.	

CODE SECTION	DESIGN PROCEDURE	CALCULATION
STEP 2 (cont.)		
D.3.6.1	Calculate N_{cbg} using $h_{ef,provided}$.	$h_{ef,provided}$ = 5 – 0.187 + 0.375 – burnoff (0.125 in.) = 5.06 in. > 4.02 in. OK N_{cbg} = 1.96 × 1.4 × 1.52 × $(5.06)^{1.5}$ = 47.47 kips
D.4.1.2	Final check: a) Ductility: b) Strength:	$0.85N_{cbg} \geq N_{sa}$ 0.85 × 47.47 = 40.35 > 28.6 kips OK $\phi N_{cbg} \geq N_{ua}$ 0.75 × 47.47 = 35.6 > 18 kips OK
STEP 3: Check pullout strength of anchor.		
D.5.3 D.3.6.1	Calculate the pullout strength N_{pn} of the stud in tension in accordance with D.5.3. Design embedment as ductile in accordance with D.3.6.1	$N_{pn} = \psi_{c,P}N_P$ (D-14)
D.5.3.1	Concrete is cracked per problem statement	
D.5.3.4	Calculate pullout strength of anchor.	$4N_p = 4A_{brg}f'_c$ (D-15) $= 4A_{brg} \times 8 \times 4$ $= 128A_{brg}$
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 0.75 in. for a 3/8 in. diameter stud (see also Table 6 in Appendix A).	$A_{brg} = \pi \times (0.75^2 - 0.375^2)/4$ = 0.33 in.^{2}
D.3.6.1	Design embedment as ductile, in accordance with D.3.6.1: $0.85N_{pn} \geq N_{sa}$	N_{pn} = 1.0 × 128 × 0.33 = 42.4 kips $0.85N_{pn}$ = 0.85 × 42.4 = 36.04 > N_{sa} = 28.6 kips Therefore, ductile OK **Use 3/8 in. diameter x 5 in. long stud.** **Use standard length stud.** OK
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} = 18 kips
Step 1	Steel strength	ϕN_{sa} = 0.8 × 28.6 = 22.28 kips
Step 2	Concrete breakout strength	ϕN_{cbg} = 0.75 × 47.47 = 35.6 kips
Step 3	Concrete pullout strength	ϕN_{pn} = 0.75 × 42.4 = 36.04 kips
Step 4	Concrete side-face blowout strength	ϕN_{sb} = N/A
D.4.1.2	Design strength of stud	$\phi N_n = \min(\phi N_{sa}, \phi N_{cbg}, \phi N_{pn})$ = min(22.88, 35.6, 36.04) = 22.88 kips > N_{ua} = 18 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 satisfied the minimum thickness required for stud welding.	Stud welding of 3/8 in. diameter studs is acceptable on 5/8 in. thick plate per D.6.2.3. OK

*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.

APPENDIX A—TABLES

Table 1—Materials for headed and threaded anchors*

Material	Grade or type	Diameter, in.	Tensile strength, minimum, ksi	Yield strength, minimum		Elongation, minimum		Reduction of area, minimum, %	ACI 349 ductility criterion	Comments
				ksi	Method	%	Length			
Welded studs AWS D.1.1:2006 ASTM A 29-05/ A108-03	B 1010 B 1020	1/4 to 1	65	51	0.2%	20	2 in.	50	Ductile	Structural Welding Code—Steel, Section 7, covers welded headed or welded bent studs. AWS D1.1 requires studs to be made from cold drawn bar stock conforming to requirements of ASTM A 108.
ASTM F 1554-04 (HD, T)†	36	1/4 to 4	58	36	0.2%	23	2 in.	40	Ductile	ASTM F 1554, "Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength," is the preferred material specification for anchors.
	55	≤2‡	75	55	0.2%	21	2 in.	30	Ductile‡	
	105	1/4 to 3	125	105	0.2%	15	2 in.	45	Ductile	
ASTM A 193-06a (T)	B7	≤2-1/2	125	105	0.2%	16	4D	50	Ductile	ASTM A 193, "Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for High-Temperature Service": Grade B7 is an alloy steel for use in high-temperature service.
		Over 2-1/4 to 4	115	95	0.2%	16	4D	50	Ductile	
		Over 4 to 7	100	75	0.2%	18	4D	50	Ductile	
ASTM A 307-04 (Gr. A: HD) (Gr. C: T)	A	1/4 to 4	60	—	—	18	2 in.	—	Ductile	ASTM A 307, "Standard Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength": ACI 349 specifies that elements meeting ASTM A 307 shall be considered ductile. Note that Grade C conforms to tensile properties for ASTM A 36.
	C	1/4 to 4	58	36	—	23	2 in.	—	Ductile	
ASTM A 36-05 (T)	—	To 8	58	36	—	23	2 in.	—	Ductile	ASTM A 36, "Standard Specification for Carbon Structural Steel": Because ACI 318 considers ASTM A 307 to be ductile, A 36 will also qualify because it is the basis for ASTM A 307 Grade C.
ASTM A 449-04b (HD, T)	1	1/4 to 1	120	92	0.2%	14	4D	35	Ductile	ASTM A 449, "Standard Specification for Quenched and Tempered Steel Bolts and Studs": this specification is for general high-strength applications.
		Over 1 to 1-1/2	105	81	0.2%	14	4D	35	Ductile	
		Over 1-1/4 to 3	90	58	0.2%	14	4D	35	Ductile	

*The materials listed are commonly used for concrete fasteners (anchors). Although other material may be used (for example, ASTM A 193 for high-temperature applications, ASTM A 320 for low-temperature applications), those listed are preferred for normal use. Structural steel bolting materials such as ASTM A 325 and A 490 are not typically available in the lengths needed for concrete fastening applications.

†Anchor type availability is denoted as follows: HD = headed bolt; and T = threaded bolt.

‡Diameters larger than 2 in. (up to 4 in.) are available, but the reduction of area will vary for Grade 55.

Table 2—Threaded fastener dimensions*

Diameter† d_o, in.	Threads per in., n_{th}	Effective area		Nominal steel strength N_{sa}	
		Gross,‡ A_D, in.^{2}	Tensile,§ A_{se}, in.^{2}	Stud,‖ kips	Threaded,# kips
0.250	20	0.049	0.032	3.2	1.8
0.375	16	0.110	0.078	7.2	4.5
0.500	13	0.196	0.142	12.8	8.2
0.625	11	0.307	0.226	19.9	13.1
0.750	10	0.442	0.334	28.7	19.4
0.875	9	0.601	0.462	39.1	26.8
1.000	8	0.785	0.606	51.1	35.1
1.125	7	0.994	0.763	64.6	44.3
1.250	7	1.227	0.969	79.8	56.2
1.375	6	1.485	1.16	96.5	67.3
1.500	6	1.767	1.41	114.9	81.5
1.750	5	2.405	1.90	156.3	110.2
2.000	4.5	3.142	2.50	204.2	144.9

*Table taken from AISC *Manual of Concrete Construction*.

†Concrete breakout strength limited to anchor diameter no greater than 2 in. and length no greater than 25 in. (D.4.2.2).

‡Use gross area A_D for studs.

§Use tensile stress area A_{se} for threaded anchors (Note: A_{se} is the same as tensile stress area in AISC). $A_{se} = 0.7854 \times (d_o - (0.9743/n_{th}))^2$.

‖$N_{sa} = A_D \times f_{uta}$; f_{uta} = 65 ksi; see Table 1 for other materials.

#$N_{sa} = A_D \times f_{uta}$; f_{uta} = 58 ksi; see Table 1 for other materials.

Note: f_{uta} = 65 ksi for A108, and f_{uta} = 58 ksi for F 1554 Gr. 36.

Table 3(a)—Required embedment for ductile behavior: Free field—single threaded cast-in headed bolt anchor (F 1554 Grade 36, Reference D.3.6.1, 0.85 factor)

Nominal anchor diameter d_o, in.	Anchor areas		$N_{sa} = A_{se} \times f_{uta}$	Required embedment depth h_{ef} for ductile behavior, in.*				
				Concrete strength, psi				
	Gross area of bolt, A_D, in.^{2}	Tensile area, A_{se},† in.^{2}	f_{uta} = 58 ksi	3000	4000	5000	6000	8000
0.250	0.049	0.032	1.8	1.4	1.3	1.2	1.1	1.0
0.375	0.110	0.078	4.5	2.5	2.3	2.1	2.0	1.8
0.500	0.196	0.142	8.2	3.8	3.4	3.2	3.0	2.7
0.625	0.307	0.226	13.1	5.2	4.7	4.4	4.1	3.7
0.750	0.442	0.334	19.4	6.7	6.1	5.7	5.3	4.8
0.875	0.601	0.462	26.8	8.3	7.6	7.0	6.6	6.0
1.000	0.785	0.606	35.1	10.0	9.1	8.4	7.9	7.2
1.125	0.994	0.763	44.3	11.6	10.6	9.8	9.2	8.4
1.250	1.227	0.969	56.2	13.6	12.4	11.5	10.8	9.8
1.375	1.485	1.16	67.3	15.4	14.0	13.0	12.2	11.1
1.500	1.767	1.41	81.5	17.5	15.9	14.7	13.9	12.6
1.750	2.405	1.90	110.2	21.3	19.4	18.0	16.9	15.4
2.000	3.142	2.50	144.9	25.6	23.3	21.6	20.3	18.5

*$0.85N_b = N_{sa}$; $N_b = 24(f_c')^{0.5}h_{ef}^{1.5}$; $h_{ef} = (N_{sa}/(0.85 \times 24 \times f_c'^{0.5}))^{2/3}$.
†A_{se} taken from Table 2.

Table 3(b)—Required embedment for ductile behavior: Free field—single threaded cast-in headed bolt anchor (F 1554 Grade 105, Reference D.3.6.1, 0.85 factor)

Nominal anchor diameter d_o, in.	Anchor areas		$N_{sa} = A_{se} \times f_{uta}$	Required embedment depth h_{ef} for ductile behavior, in.*				
				Concrete strength, psi				
	Gross area of bolt, A_D, in.^{2}	Tensile area, A_{se},† in.^{2}	f_{uta} = 105 ksi	3000	4000	5000	6000	8000
0.250	0.049	0.032	3.3	2.1	1.9	1.8	1.6	1.5
0.375	0.110	0.078	8.2	3.8	3.4	3.2	3.0	2.7
0.500	0.196	0.142	14.9	5.6	5.1	4.7	4.5	4.1
0.625	0.307	0.226	23.7	7.7	7.0	6.5	6.1	5.5
0.750	0.442	0.334	35.1	10.0	9.0	8.4	7.9	7.2
0.875	0.601	0.462	48.5	12.3	11.2	10.4	9.8	8.9
1.000	0.785	0.606	63.6	14.8	13.4	12.5	11.7	10.7
1.125	0.994	0.763	80.1	17.3	15.7	14.6	13.7	12.4
1.250	1.227	0.969	101.8	20.2	18.4	17.1	16.1	14.6
1.375	1.485	1.16	121.8	22.8	20.7	19.2	18.1	16.5
1.500	1.767	1.41	147.6	25.9	23.6	21.9	20.6	18.7
1.750	2.405	1.90	199.4	31.7	28.8	26.7	25.2	22.9
2.000	3.142	2.50	262.3	38.1	34.6	32.1	30.2	27.4

*$0.85N_b = N_{sa}$; $N_b = 24(f_c')^{0.5}h_{ef}^{1.5}$; $h_{ef} = (N_{sa}/(0.85 \times 24 \times f_c'^{0.5}))^{2/3}$.
†A_{se} taken from Table 2.

Table 3(c)—Required embedment for ductile behavior: Free field—single threaded cast-in headed stud anchor (Stud f_{uta} = 65 ksi, Reference D.3.6.1, 0.85 factor)

Nominal anchor diameter d_o, in.	Anchor areas		$N_{sa} = A_{se} \times f_{uta}$	Required embedment depth h_{ef} for ductile behavior, in.*				
				Concrete strength, psi				
	Gross area of bolt, A_D, in.^{2}	Tensile area, A_{se},† in.^{2}	f_{uta} = 65 ksi	3000	4000	5000	6000	8000
0.250	0.049	0.032	3.2	2.0	1.8	1.7	1.6	1.5
0.375	0.110	0.078	7.2	3.5	3.1	2.9	2.7	2.5
0.500	0.196	0.142	12.8	5.1	4.6	4.3	4.0	3.7
0.625	0.307	0.226	19.9	6.8	6.2	5.8	5.4	4.9
0.750	0.442	0.334	28.7	8.7	7.9	7.3	6.9	6.3
0.875	0.601	0.462	39.1	10.7	9.7	9.0	8.5	7.7
1.000	0.785	0.606	51.1	12.8	11.6	10.8	10.1	9.2
1.125	0.994	0.763	64.6	15.0	13.6	12.6	11.9	10.8
1.250	1.227	0.969	79.8	17.2	15.6	14.5	13.7	12.4
1.375	1.485	1.16	96.5	19.5	17.8	16.5	15.5	14.1
1.500	1.767	1.41	114.9	21.9	19.9	18.5	17.4	15.8
1.750	2.405	1.90	156.3	27.0	24.5	22.7	21.4	19.4
2.000	3.142	2.50	204.2	32.2	29.3	27.2	25.6	23.2

*$0.85N_b = N_{sa}$; $N_b = 24(f_c')^{0.5}h_{ef}^{1.5}$; $h_{ef} = (N_{sa}/(0.85 \times 24 \times f_c'^{0.5}))^{2/3}$.
†A_{se} taken from Table 2.

Table 4(a)—Anchor head and nut (square head) dimensions[*]

Nominal anchor diameter d_o, in.	Anchor areas: Gross, area of bolt, A_D, in.^{2}	Anchor areas: Tensile area, A_{se},[†] in.^{2}	Width F, in.	Width C, in.	Height H, in.	Gross area of head A_H,[‡] based on width F, in.^{2}	Net bearing area A_{brg},[§] in.^{2}
Head size							
0.250	0.049	0.032	0.375	0.500	0.188	0.14	0.09
0.375	0.110	0.078	0.563	0.813	0.250	0.32	0.21
0.500	0.196	0.142	0.750	1.063	0.313	0.56	0.37
0.625	0.307	0.226	0.938	1.313	0.438	0.88	0.57
0.750	0.442	0.334	1.125	1.563	0.500	1.27	0.82
0.875	0.601	0.462	1.313	1.875	0.625	1.72	1.12
1.000	0.785	0.606	1.500	2.125	0.688	2.25	1.46
1.125	0.994	0.763	1.688	2.375	0.750	2.85	1.85
1.250	1.227	0.969	1.875	2.625	0.875	3.52	2.29
1.375	1.485	1.16	2.063	2.938	0.938	4.25	2.77
1.500	1.767	1.41	2.250	3.188	1.000	5.06	3.30
1.750	2.405	1.90	—	—	—	—	—
2.000	3.142	2.50	—	—	—	—	—
Nut size[‖]							
0.250	0.049	0.032	0.438	0.625	0.250	0.19	0.14
0.375	0.110	0.078	0.625	0.875	0.313	0.39	0.28
0.500	0.196	0.142	0.813	1.125	0.438	0.66	0.46
0.625	0.307	0.226	1.00	1.438	0.563	1.00	0.69

[*]Dimensions taken from AISC *Steel Design Manual*.
[†]See Table 2 for definition of A_{se}.
[‡]$A_H = F^2$ or $A_H = 1.5F^2 \tan 30°$.
§$A_{brg} = A_H - A_D$.
[‖]For other diameters, the nut dimensions match the head.

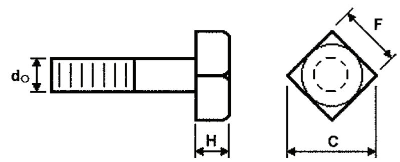


Table 4(b)—Anchor head and nut (hex head) dimensions[*]

Nominal anchor diameter d_o, in.	Anchor areas: Gross, area of bolt, A_D, in.^{2}	Anchor areas: Tensile area, A_{se},[†] in.^{2}	Width F, in.	Width C, in.	Height H, in.	Gross area of head A_H,[‡] based on width F, in.^{2}	Net bearing area A_{brg},[§] in.^{2}
0.250	0.049	0.032	0.438	0.500	0.188	0.17	0.12
0.375	0.110	0.078	0.563	0.625	0.250	0.27	0.16
0.500	0.196	0.142	0.750	0.875	0.375	0.49	0.29
0.625	0.307	0.226	0.938	1.063	0.438	0.76	0.45
0.750	0.442	0.334	1.125	1.313	0.500	1.10	0.65
0.875	0.601	0.462	1.313	1.500	0.563	1.49	0.89
1.000	0.785	0.606	1.500	1.750	0.688	1.95	1.16
1.125	0.994	0.763	1.688	1.938	0.750	2.47	1.47
1.250	1.227	0.969	1.875	2.188	0.875	3.04	1.82
1.375	1.485	1.16	2.063	2.375	0.938	3.68	2.20
1.500	1.767	1.41	2.250	2.625	1.000	4.38	2.62
1.750	2.405	1.90	2.625	3.000	1.188	5.97	3.56
2.000	3.142	2.50	3.000	3.438	1.375	7.79	4.65

[*]Dimensions taken from AISC *Steel Design Manual*.
[†]See Table 2 for definition of A_{se}.
[‡]$A_H = F^2$ or $A_H = 1.5F^2 \tan 30°$.
§$A_{brg} = A_H - A_D$.

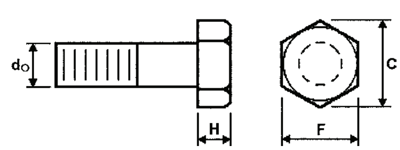


Table 4(c)—Anchor head and nut (heavy hex) dimensions*

Nominal anchor diameter d_o, in.	Anchor areas		Width F, in.	Width C, in.	Height H, in.	Gross area of head A_H,‡ based on width F, in.^{2}	Net bearing area A_{brg},§ in.^{2}
	Gross, area of bolt, A_D, in.^{2}	Tensile area, A_{se},† in.^{2}					
0.250	0.049	0.032	—	—	—	—	—
0.375	0.110	0.078	—	—	—	—	—
0.500	0.196	0.142	0.875	1.000	0.375	0.66	0.47
0.625	0.307	0.226	1.063	1.250	0.438	0.98	0.67
0.750	0.442	0.334	1.250	1.438	0.500	1.35	0.91
0.875	0.601	0.462	1.438	1.688	0.563	1.79	1.19
1.000	0.785	0.606	1.625	1.875	0.688	2.29	1.50
1.125	0.994	0.763	1.813	2.063	0.750	2.85	1.85
1.250	1.227	0.969	2.000	2.313	0.875	3.46	2.24
1.375	1.485	1.16	2.188	2.500	0.938	4.14	2.66
1.500	1.767	1.41	2.375	2.750	1.000	4.88	3.12
1.750	2.405	1.90	2.750	3.500	1.188	6.55	4.14
2.000	3.142	2.50	3.125	3.625	1.375	8.46	5.32

*Dimensions taken from AISC *Steel Design Manual*.
†See Table 2 for definition of A_{se}.
‡$A_H = F^2$ or $A_H = 1.5F^2 \tan 30°$.
§$A_{brg} = A_H - A_D$.

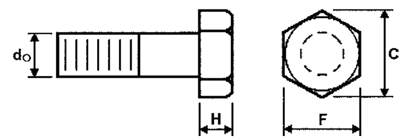


Table 5—Hardened washer dimensions

SAE dimensions*

Nominal anchor diameter d_o, in.	Anchor areas: Gross, area of bolt, A_D, in.^{2}	Anchor areas: Tensile area, A_{se},‡ in.^{2}	Washer dimensions: OD, in.	Washer dimensions: ID, in.	Washer dimensions: Thickness t, in.	Washer dimensions: Gross area of head A_H,§ in.^{2}	Washer dimensions: Net bearing area A_{brg}‖, in.^{2}
0.250	0.049	0.032	0.563	0.281	0.051/0.080	0.25	0.20
0.375	0.110	0.078	0.813	0.406	0.051/0.080	0.52	0.41
0.500	0.196	0.142	1.167	0.531	0.074/0.121	1.07	0.87
0.625	0.307	0.226	1.313	0.656	0.074/0.121	1.35	1.05
0.750	0.442	0.334	1.469	0.813	0.108/0.160	1.69	1.25
0.875	0.601	0.462	1.750	0.938	0.108/0.160	2.41	1.80
1.000	0.785	0.606	2.00	1.063	0.108/0.160	3.14	2.36
1.125	0.994	0.763	2.250	1.250	0.136/0.177 (0.305/0.375 extra thick)	3.98	2.98
1.250	1.227	0.969	2.500	1.375	0.136/0.192	4.91	3.68
1.375	1.485	1.16	2.750	1.500	0.136/0.213	5.94	4.45
1.500	1.767	1.41	3.000	1.625	0.153/0.213	7.07	5.30
1.750	2.405	1.90	3.375	1.875	0.153/0.213	8.95	6.54
2.000	3.142	2.50	3.750	2.125	0.153/0.213	11.04	7.90

U.S. Standard dimensions†

Nominal anchor diameter d_o, in.	Anchor areas: Gross, area of bolt, A_D, in.^{2}	Anchor areas: Tensile area, A_{se},‡ in.^{2}	Washer dimensions: OD, in.	Washer dimensions: ID, in.	Washer dimensions: Thickness t, in.	Washer dimensions: Gross area of head A_H,§ in.^{2}	Washer dimensions: Net bearing area A_{brg}‖, in.^{2}
0.250	0.049	0.032	0.750	0.313	0.064/0.080	0.44	0.39
0.375	0.110	0.078	1.000	0.438	0.079/0.093	0.79	0.67
0.500	0.196	0.142	1.375	0.375	0.122/0.146	1.48	1.29
0.625	0.307	0.226	1.750	0.656	0.136/0.160	2.41	2.10
0.750	0.442	0.334	2.000	0.813	0.136/0.160	3.14	2.70
0.875	0.601	0.462	2.250	0.938	0.136/0.160	3.98	3.37
1.000	0.785	0.606	2.500	1.063	0.136/0.192	4.91	4.12
1.125	0.994	0.763	2.750	1.250	0.126/0.192	5.94	4.95
1.250	1.227	0.969	3.000	1.375	0.126/0.192	7.07	5.84
1.375	1.485	1.160	3.250	1.500	0.126/0.192	8.30	6.81
1.500	1.767	1.405	3.500	1.625	0.153/0.213	9.62	7.85
1.750	2.405	1.899	4.250	1.875	0.153/0.213	14.19	11.78
2.000	3.142	2.50	4.500	2.125	0.153/0.213	15.90	12.76

*Hardened washers to SAE dimensions.
†Material ASTM F 436.
‡Refer to Table 2 for definition of A_{se}.
§$A_H = \pi(OD)^2/4$.
‖$A_{brg} = A_H - A_D$.

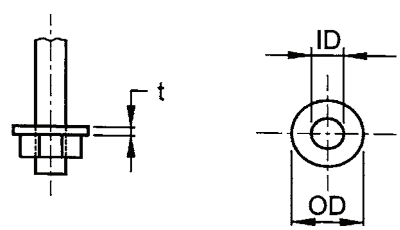
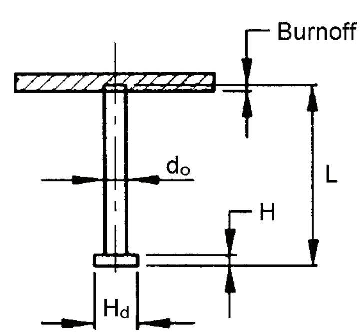


Table 6—Stud dimensions* (Steel: ASTM A 108; f_{ut} = 65 ksi; f_y = 51 ksi)

Diameter d_o, in.	Stud dimensions: Burnoff, in.	Stud dimensions: H, in.	Stud dimensions: A + burnoff, in.	Stud dimensions: H_d, in.	Areas: A_{stud},† in.^{2}	Areas: A_{brg},‡ in.^{2}
0.250	0.125	0.187	0.312	0.500	0.049	0.15
0.375	0.125	0.281	0.406	0.750	0.110	0.33
0.500	0.125	0.312	0.437	1.000	0.196	0.59
0.625	0.187	0.312	0.499	1.250	0.307	0.92
0.750	0.187	0.375	0.562	1.250	0.442	0.79
0.875	0.187	0.375	0.562	1.375	0.601	0.88
1.000	0.250	0.500	0.750	1.625	0.785	1.29



*Taken from Nelson Stud catalog.
†$A_{stud} = \pi(d_o)^2/4$.
‡$A_{brg} = \pi(H_d^2 - d_o^2)/4$.

APPENDIX B of 349.2R

ACI 349, Appendix D, Code and Commentary (Appendix Commentary follows the Code)

APPENDIX D—ANCHORING TO CONCRETE

D.1—Definitions

anchor—a steel element either cast into concrete or post-installed into a hardened concrete member and used to transmit applied loads, including headed bolts, headed studs, expansion anchors, undercut anchors, or specialty inserts.

anchor group—a number of anchors of approximately equal effective embedment depth with each anchor spaced at less than three times its embedment depth from one or more adjacent anchors.

anchor pullout strength—the strength corresponding to the anchoring device or a major component of the device sliding out from the concrete without breaking out a substantial portion of the surrounding concrete.

attachment—the structural assembly, external to the surface of the concrete, that transmits loads to or receives loads from the anchor.

brittle steel element—an element with a tensile test elongation of less than 14%, or reduction in area of less than 30%, or both.

cast-in anchor—a headed bolt or headed stud, installed before placing concrete.

concrete breakout strength—the strength corresponding to a volume of concrete surrounding the anchor or group of anchors separating from the member.

concrete pryout strength—the strength corresponding to formation of concrete spall behind short, stiff anchors displaced in the direction opposite to the applied shear force.

distance sleeve—a sleeve that encases the center part of an undercut anchor, a torque-controlled expansion anchor, or a displacement-controlled expansion anchor, but does not expand.

ductile embedment—an embedment designed for a ductile steel failure in accordance with D.3.6.1.

ductile steel element—an element with a tensile test elongation of at least 14% and reduction in area of at least 30%. A steel element meeting the requirements of ASTM A 307 shall be considered ductile.

edge distance—the distance from the edge of the concrete surface to the center of the nearest anchor.

effective embedment depth—the overall depth through which the anchor transfers force to or from the surrounding concrete. The effective embedment depth will normally be the depth of the concrete failure surface in tension applications. For cast-in headed anchor bolts and headed studs, the effective embedment depth is measured from the bearing contact surface of the head.

embedment—a steel component embedded in the concrete to transmit applied loads to or from the concrete structure. The embedment may be fabricated of plates, shapes, anchors, reinforcing bars, shear connectors, specialty inserts, or any combination thereof.

expansion anchor—a post-installed anchor inserted into hardened concrete that transfers loads to or from the concrete by direct bearing or friction or both. Expansion anchors may be torque-controlled, where the expansion is achieved by a torque acting on the screw or bolt; or displacement-controlled, where the expansion is achieved by impact forces acting on a sleeve or plug and the expansion is controlled by the length of travel of the sleeve or plug.

expansion sleeve—the outer part of an expansion anchor that is forced outward by the center part, either by applied torque or impact, to bear against the sides of the predrilled hole.

five percent fractile—a statistical term meaning 90% confidence that there is 95% probability of the actual strength exceeding the nominal strength.

headed stud—a steel anchor conforming to the requirements of AWS D1.1 and affixed to a plate or similar steel attachment by the stud arc welding process before casting.

post-installed anchor—an anchor installed in hardened concrete. Expansion anchors and undercut anchors are examples of post-installed anchors.

projected area—the area on the free surface of the concrete member that is used to represent the larger base of the assumed rectilinear failure surface.

side-face blowout strength—the strength of anchors with deeper embedment but thinner side cover corresponding to concrete spalling on the side face around the embedded head while no major breakout occurs at the top concrete surface.

specialty insert—predesigned and prefabricated cast-in anchors specifically designed for attachment of bolted or slotted connections. Specialty inserts are often used for handling, transportation, and erection, but are also used for anchoring structural members.

supplementary reinforcement—reinforcement proportioned to tie a potential concrete failure prism to the structural member.

undercut anchor—a post-installed anchor that develops its tensile strength from the mechanical interlock provided by undercutting of the concrete at the embedded end of the anchor. The undercutting is achieved with a special drill before installing the anchor or alternatively by the anchor itself during its installation.

D.2—Scope

D.2.1 This appendix provides design requirements for structural embedments in concrete used to transmit structural loads by means of tension, shear, or a combination of tension and shear between (a) connected structural members; or (b) safety-related attachments and structural members. Safety levels specified are intended for in-service conditions, rather than for short-term handling and construction conditions.

D.2.2 This appendix applies to both cast-in anchors and post-installed anchors. Through bolts, multiple anchors connected to a single steel plate at the embedded end of the anchors, and direct anchors such as powder or pneumatic actuated nails or bolts, are not included. Reinforcement used as part of the embedment shall be designed in accordance with other parts of this Code. Grouted embedments shall meet the requirements of D.12.

D.2.3 Headed studs and headed bolts having a geometry that has been demonstrated to result in a pullout strength in uncracked concrete equal or exceeding $\mathbf{1.4}\boldsymbol{N_p}$ (where $\boldsymbol{N_p}$ is given by Eq. (D-13)) are included. Post-installed anchors are included provided that D.3.3 is satisfied.

D.2.4 Load applications that are predominantly high-cycle fatigue are not covered by this appendix.

D.2.5 In addition to meeting the requirements of this appendix, consideration shall be given to the effect of the forces applied to the embedment on the behavior of the overall structure.

D.2.6 The jurisdiction of this Code covers steel material below the surface of the concrete and the anchors extending above the surface of the concrete. The requirements for the attachment to the embedment shall be in accordance with applicable Codes and are beyond the scope of this appendix.

D.3—General requirements

D.3.1 The embedment and surrounding concrete or grout shall be designed for critical effects of factored loads as determined by elastic analysis. Plastic analysis approaches are permitted where nominal strength is controlled by ductile steel elements, provided that deformational compatibility is taken into account. Assumptions used in distributing loads within the embedment shall be consistent with those used in the design of the attachment.

D.3.2 The design strength of anchors shall equal or exceed the largest required strength calculated from the applicable load combinations in 9.2 or C.2.

D.3.3 Post-installed structural anchors shall be tested before use, simulating the conditions of the intended field of application, to verify that they are capable of sustaining their design strength in cracked concrete under seismic loads. These verification tests shall be conducted by an independent testing agency and shall be certified by a licensed professional engineer with full description and details of the testing programs, procedures, results, and conclusions.

D.3.4 All provisions for anchor axial tension and shear strength apply to normalweight concrete only.

D.3.5 The values of $\boldsymbol{f_c'}$ used for calculation purposes in this appendix shall not exceed 10,000 psi for cast-in anchors, and 8000 psi for post-installed anchors. Testing is required for post-installed anchors when used in concrete with $\boldsymbol{f_c'}$ greater than 8000 psi.

D.3.6 *Embedment design*

D.3.6.1 Embedment design shall be controlled by the strength of embedment steel. The design strength shall be determined using the strength-reduction factor specified in D.4.4(a) or D.4.5(a). It shall be permitted to assume that design is controlled by the strength of embedment steel where the design concrete breakout tensile strength of the embedment, the design side blowout strength of the embedment, and the design pullout strength of the anchors exceed the nominal tensile strength of the embedment steel and when the design concrete breakout shear strength and design concrete pryout strength exceed the nominal shear strength of the embedment steel. The design concrete tensile strength, the design side blowout strength, the design pullout strength, the design concrete pryout strength, and the design concrete breakout shear strength shall be taken as 0.85 times the nominal strengths.

D.3.6.2 As an alternate to D.3.6.1, the attachment shall be designed to yield at a load level corresponding to anchor or group forces not greater than 75% of the anchor design strength specified in D.4.1.2. The anchor design strength shall be determined using the strength-reduction factors specified in D.4.4 or D.4.5.

D.3.6.3 It shall be permitted to design anchors as nonductile anchors for tension or shear loading, or both. The design strength of such anchors shall be taken as $\mathbf{0.60}\boldsymbol{\phi N_n}$ and $\mathbf{0.60}\boldsymbol{\phi V_n}$, where ϕ is given in D.4.4 or D.4.5, and $\boldsymbol{N_n}$ and $\boldsymbol{V_n}$ are determined in accordance with D.4.1.

D.3.7 Material and testing requirements for embedment steel shall be specified by the engineer so that the embedment design is compatible with the intended function of the attachment.

D.3.8 Embedment materials for ductile anchors other than reinforcing bars shall be ductile steel elements.

D.3.9 Ductile anchors that incorporate a reduced section in the tension or shear load path shall satisfy one of the following conditions:

(a) The nominal tensile strength of the reduced section shall be greater than the yield strength of the unreduced section;

(b) For bolts, the length of thread in the load path shall be at least two anchor diameters.

D.3.10 The design strength of embedment materials is permitted to be increased in accordance with Appendix F for embedments subject to impactive and impulsive loads.

D.3.11 Plastic deformation of the embedment is permitted for impactive and impulsive loading provided the strength of the embedment is controlled by the strength of the embedment steel as specified in D.3.6.

D.4—General requirements for strength of anchors

D.4.1 Strength design of anchors shall be based either on computation using design models that satisfy the requirements of D.4.2, or on test evaluation using the 5% fractile of test results for the following:

(a) steel strength of anchor in tension (D.5.1);

(b) steel strength of anchor in shear (D.6.1);

(c) concrete breakout strength of anchor in tension (D.5.2);

(d) concrete breakout strength of anchor in shear (D.6.2);

(e) pullout strength of anchor in tension (D.5.3);

(f) concrete side-face blowout strength of anchor in tension (D.5.4); and

(g) concrete pryout strength of anchor in shear (D.6.3).

In addition, anchors shall satisfy the required edge distances, spacings, and thicknesses to preclude splitting failure, as required in D.8.

D.4.1.1 For the design of anchors,

$$\phi N_n \geq N_{ua} \tag{D-1}$$

$$\phi V_n \geq V_{ua} \quad \text{(D-2)}$$

D.4.1.2 In Eq. (D-1) and (D-2), ϕN_n and ϕV_n are the lowest design strengths determined from all appropriate failure modes. ϕN_n is the lowest design strength in tension of an anchor or group of anchors as determined from consideration of ϕN_{sa}, $\phi n N_{pn}$, either ϕN_{sb} or ϕN_{sbg}, and either ϕN_{cb} or ϕN_{cbg}. ϕV_n is the lowest design strength in shear of an anchor or a group of anchors as determined from consideration of: ϕV_{sa}, either ϕV_{cb} or ϕV_{cbg}, and either ϕV_{cp} or ϕV_{cpg}.

D.4.1.3 When both N_{ua} and V_{ua} are present, interaction effects shall be considered in accordance with D.4.3.

D.4.2 The nominal strength for any anchor or group of anchors shall be based on design models that result in predictions of strength in substantial agreement with results of comprehensive tests. The materials used in the tests shall be compatible with the materials used in the structure. The nominal strength shall be based on the 5% fractile of the basic individual anchor strength. For nominal strengths related to concrete strength, modifications for size effects, the number of anchors, the effects of close spacing of anchors, proximity to edges, depth of the concrete member, eccentric loadings of anchor groups, and presence or absence of cracking shall be taken into account. Limits on edge distances and anchor spacing in the design models shall be consistent with the tests that verified the model.

D.4.2.1 The effect of supplementary reinforcement provided to confine or restrain the concrete breakout, or both, shall be permitted to be included in the design models used to satisfy D.4.2.

D.4.2.2 For anchors with diameters not exceeding 2 in., and tensile embedments not exceeding 25 in. in depth, the concrete breakout strength requirements shall be considered satisfied by the design procedure of D.5.2 and D.6.2.

D.4.3 Resistance to combined tensile and shear loads shall be considered in design using an interaction expression that results in computation of strength in substantial agreement with results of comprehensive tests. This requirement shall be considered satisfied by D.7.

D.4.4 Strength-reduction factor ϕ for anchors in concrete shall be as follows when the load combinations of 9.2 are used:

(a) Anchor governed by strength of a ductile steel element
- i) Tension loads 0.75
- ii) Shear loads 0.65

(b) Anchor governed by strength of a brittle steel element
- i) Tension loads 0.65
- ii) Shear loads 0.60

(c) Anchor governed by concrete breakout, side-face blowout, pullout, or pryout strength

	Condition A	Condition B
i) Shear loads	0.75	0.70
ii) Tension loads		
Cast-in headed studs or headed bolts	0.75	0.70
Post-installed	0.75	0.65

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member.

Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.

(d) Anchor controlled by concrete bearing 0.65

(e) Structural plates, shapes, and specialty inserts
- i) Tension, compression, and bending loads 0.90
- ii) Shear loads 0.55

(f) Embedded plates and shear lugs
- Shear toward free edge 0.80

D.4.5 Strength-reduction factor ϕ for anchors in concrete shall be as follows when the load combinations referenced in Appendix C are used:

(a) Anchor governed by strength of a ductile steel element
- i) Tension loads 0.80
- ii) Shear loads 0.75

(b) Anchor governed by strength of a brittle steel element
- i) Tension loads 0.70
- ii) Shear loads 0.65

(c) Anchor governed by concrete breakout, side-face blowout, pullout, or pryout strength

	Condition A	Condition B
i) Shear loads	0.85	0.75
ii) Tension loads		
Cast-in headed studs or headed bolts	0.85	0.75
Post-installed	0.85	0.75

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member.

Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.

(d) Anchor controlled by concrete bearing 0.70

(e) Structural plates, shapes, and specialty inserts
- i) Tension, compression, and bending loads 0.90
- ii) Shear loads 0.55

(f) Embedded plates and shear lugs
- Shear toward free edge 0.85

D.4.6 *Bearing strength*

D.4.6.1 A combination of bearing and shear friction mechanisms shall not be used to develop the nominal shear strength defined in accordance with 9.2 or C.2. If the requirements of 9.2.3 (or C.2.6) are satisfied, however, it shall be permitted to use the available confining force afforded by the tension anchors in combination with acting (or applied) loads used in determining the shear strength of embedments with shear lugs.

D.4.6.2 The design bearing strength used for concrete or grout placed against shear lugs shall not exceed $1.3\phi f'_c$ using a strength-reduction factor ϕ in accordance with D.4.4 if load combinations in 9.2 are used or in accordance with D.4.5 if load combinations in Appendix C are used. For grouted installations, the value of f'_c shall be the compressive strength of the grout or the concrete, whichever is less.

D.5—Design requirements for tensile loading

D.5.1 *Steel strength of anchor in tension*

D.5.1.1 The nominal strength of an anchor in tension as governed by the steel, N_{sa}, shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

D.5.1.2 The nominal strength of a single anchor or group of anchors in tension, N_{sa}, shall not exceed

$$N_{sa} = nA_{se} f_{uta} \tag{D-3}$$

where n is the number of anchors in the group, and f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ and 125,000 psi.

D.5.2 *Concrete breakout strength of anchor in tension*

D.5.2.1 The nominal concrete breakout strength, N_{cb} or N_{cbg}, of a single anchor or group of anchors in tension shall not exceed

(a) for a single anchor

$$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \tag{D-4}$$

(b) for a group of anchors

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \tag{D-5}$$

Factors $\psi_{ec,N}$, $\psi_{ed,N}$, $\psi_{c,N}$, and $\psi_{cp,N}$ are defined in D.5.2.4, D.5.2.5, D.5.2.6, and D.5.2.7, respectively. A_{Nc} is the projected concrete failure area of a single anchor or group of anchors that shall be approximated as the base of the rectilinear geometrical figure that results from projecting the failure surface outward $1.5h_{ef}$ from the centerlines of the anchor, or in the case of a group of anchors, from a line through a row of adjacent anchors. A_{Nc} shall not exceed nA_{Nco}, where n is the number of tensioned anchors in the group. A_{Nco} is the projected concrete failure area of a single anchor with an edge distance equal to or greater than $1.5h_{ef}$

$$A_{Nco} = 9h_{ef}^2 \tag{D-6}$$

D.5.2.2 The basic concrete breakout strength of a single anchor in tension in cracked concrete, N_b, shall not exceed

$$N_b = k_c \sqrt{f'_c}\, h_{ef}^{1.5} \tag{D-7}$$

where

k_c = 24 for cast-in anchors; and

k_c = 17 for post-installed anchors.

The value of k_c for post-installed anchors shall be permitted to be increased above 17 based on D.3.3 product-specific tests, but shall in no case exceed 24.

Alternatively, for cast-in headed studs and headed bolts with $11 \text{ in.} \le h_{ef} \le 25 \text{ in.}$, N_b shall not exceed

$$N_b = 16 \sqrt{f'_c}\, h_{ef}^{5/3} \tag{D-8}$$

D.5.2.3 Where anchors are located less than $1.5h_{ef}$ from three or more edges, the value of h_{ef} used in Eq. (D-4) through (D-11) shall be the greater of $c_{a,max}/1.5$ and 1/3 of the maximum spacing between anchors within the group.

D.5.2.4 The modification factor for anchor groups loaded eccentrically in tension is

$$\psi_{ec,N} = \frac{1}{\left(1 + \frac{2e'_N}{3h_{ef}}\right)} \le 1.0 \tag{D-9}$$

If the loading on an anchor group is such that only some anchors are in tension, only those anchors that are in tension shall be considered when determining the eccentricity e'_N for use in Eq. (D-9) and for the calculation of N_{cbg} in Eq. (D-5).

In the case where eccentric loading exists about two axes, the modification factor $\psi_{ec,N}$ shall be computed for each axis individually and the product of these factors used as $\psi_{ec,N}$ in Eq. (D-5).

D.5.2.5 The modification factor for edge effects for single anchors or anchor groups loaded in tension is

$$\psi_{ed,N} = 1 \text{ if } c_{a,min} \ge 1.5h_{ef} \tag{D-10}$$

$$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \text{ if } c_{a,min} < 1.5h_{ef} \tag{D-11}$$

D.5.2.6 For anchors located in a region of a concrete member where analysis indicates no cracking $(f_c < f_r)$ under the load combinations specified in 9.2 or C.2 with load factors taken as unity, the following modification factor shall be permitted:

$\psi_{c,N}$ = 1.25 for cast-in anchors; and

$\psi_{c,N}$ = 1.4 for post-installed anchors, where the value of k_c used in Eq. (D-7) is 17.

When analysis indicates cracking under the load combinations specified in 9.2 or C.2 with load factors taken as unity, $\psi_{c,N}$ shall be taken as 1.0 for both cast-in anchors and post-installed anchors. Post-installed anchors shall be qualified for use in cracked concrete in accordance with D.3.3. The cracking in the concrete shall be controlled by flexural reinforcement distributed in accordance with 10.6.4, or equivalent crack control shall be provided by confining reinforcement.

D.5.2.7 The modification factor for post-installed anchors designed for uncracked concrete in accordance with D.5.2.6 without supplementary reinforcement to control splitting is

$$\psi_{cp,N} = 1.0 \text{ if } c_{a,min} \ge c_{ac} \tag{D-12}$$

$$\psi_{cp,N} = \frac{c_{a,min}}{c_{ac}} \ge \frac{1.5h_{ef}}{c_{ac}} \text{ if } c_{a,min} < c_{ac} \tag{D-13}$$

where the critical distance c_{ac} is defined in D.8.6.

For all other cases, including cast-in anchors, $\psi_{cp,N}$ shall be taken as 1.0.

D.5.2.8 Where an additional plate or washer is added at the head of the anchor, it shall be 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 plate or washer. The effective perimeter shall not exceed the value at a section projected outward more than the thickness of the washer or plate from the outer edge of the head of the anchor.

D.5.2.9 For post-installed anchors, it shall be permitted to use a coefficient k_c in Eq. (D-7) or (D-8) based on the 5% fractile of results from product-specific tests. For such cases, the modification factor $\psi_{c,N}$ shall be based on a direct comparison between the average ultimate failure loads and the characteristic loads based on the 5% fractile of the product-specific testing in cracked concrete and otherwise identical product-specific testing in uncracked concrete.

D.5.3 *Pullout strength of anchor in tension*

D.5.3.1 The nominal pullout strength of a single anchor in tension, N_{pn}, shall not exceed

$$N_{pn} = \psi_{c,P}N_p \tag{D-14}$$

where $\psi_{c,P}$ is defined in D.5.3.5.

D.5.3.2 For post-installed expansion and undercut anchors, the values of N_p shall be based on the 5% fractile of results of tests performed and evaluated according to D.3.3. It is not permissible to calculate the pullout strength in tension for such anchors.

D.5.3.3 For single cast-in headed studs and headed bolts, it shall be permitted to evaluate the pullout strength in tension using D.5.3.4. Alternatively, it shall be permitted to use values of N_p based on the 5% fractile of tests performed and evaluated in accordance with D.3.3 but without the benefit of friction.

D.5.3.4 The pullout strength in tension of a single headed stud or headed bolt, N_p, for use in Eq. (D-14), shall not exceed

$$N_p = 8A_{brg}f_c' \tag{D-15}$$

D.5.3.5 For an anchor located in a region of a concrete member where analysis indicates no cracking $(f_t < f_r)$ under the load combinations specified in 9.2 or C.2 with load factors taken as unity, the following modification factor shall be permitted:

$$\psi_{c,P} = 1.4$$

Otherwise, $\psi_{c,P}$ shall be taken as 1.0.

D.5.4 *Concrete side-face blowout strength of a headed anchor in tension*

D.5.4.1 For a single-headed anchor with deep embedment close to an edge $(c_{a1} < 0.4h_{ef})$, the nominal side-face blowout strength, N_{sb}, shall not exceed

$$N_{sb} = 160c_{a1}\sqrt{A_{brg}}\sqrt{f_c'} \tag{D-16}$$

If c_{a2} for the single-headed anchor is less than $3c_{a1}$, the value of N_{sb} shall be multiplied by the factor $(1 + c_{a2}/c_{a1})/4$ where $1.0 \le c_{a2}/c_{a1} \le 3.0$.

D.5.4.2 For multiple-headed anchors with deep embedment close to an edge $(c_{a1} < 0.4h_{ef})$ and anchor spacing less than $6c_{a1}$, the nominal strength of anchors along the edge in a group for a side-face blowout failure N_{sbg} shall not exceed

$$N_{sbg} = \left(1 + \frac{s}{6c_{a1}}\right)N_{sb} \tag{D-17}$$

The nominal strength of the group of anchors shall be taken as the nominal strength of the outer anchors along the edge multiplied by the number of rows parallel to the edge.

D.6—Design requirements for shear loading

D.6.1 *Steel strength of anchor in shear*

D.6.1.1 The nominal strength of an anchor in shear as governed by steel, V_{sa}, shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

D.6.1.2 The nominal strength of a single anchor or group of anchors in shear, V_{sa}, shall not exceed (a) through (c):

(a) for cast-in headed stud anchors

$$V_{sa} = nA_{se}f_{uta} \tag{D-18}$$

where n is the number of anchors in the group and f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ and 125,000 psi.

(b) for cast-in headed bolt and for post-installed anchors where sleeves do not extend through the shear plane

$$V_{sa} = n0.6A_{se}f_{uta} \tag{D-19}$$

where n is the number of anchors in the group and f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ and 125,000 psi.

(c) for post-installed anchors where sleeves extend through the shear plane, V_{sa} shall be based on the results of tests performed and evaluated according to D.3.3. Alternatively, Eq. (D-19) shall be permitted to be used.

When the anchor is installed so that the critical failure plane does not pass through the sleeve, the area of the sleeve in Eq. (D-19) shall be taken as zero.

D.6.1.3 Where anchors are used with built-up grout pads, the nominal strengths of D.6.1.2 shall be multiplied by a 0.80 factor.

D.6.1.4 Friction between the baseplate and concrete shall be permitted to be considered to contribute to the nominal steel shear strength of the anchor in shear. The nominal shear strength resulting from friction between the baseplate and concrete (that is, without any contribution from anchors) may be taken as $0.40C_F$.

D.6.2 *Concrete breakout strength of anchor in shear*

D.6.2.1 The nominal concrete breakout strength, V_{cb} or V_{cbg}, in shear of a single anchor or group of anchors shall not exceed:

(a) for shear force perpendicular to the edge on a single anchor

$$V_{cb} = \frac{A_{Vc}}{A_{Vco}}\psi_{ed,V}\psi_{c,V}V_b \qquad \text{(D-20)}$$

(b) for shear force perpendicular to the edge on a group of anchors

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}}\psi_{ec,V}\psi_{ed,V}\psi_{c,V}V_b \qquad \text{(D-21)}$$

(c) for shear force parallel to an edge, V_{cb} or V_{cbg} shall be permitted to be twice the value of the shear force determined from Eq. (D-20) or (D-21), respectively, with the shear force assumed to act perpendicular to the edge and with $\psi_{ed,V}$ taken equal to 1.0.

(d) for anchors located at a corner, the limiting nominal concrete breakout strength shall be determined for each edge, and the minimum value shall be used.

Factors $\psi_{ec,V}$, $\psi_{ed,V}$, and $\psi_{c,V}$ are defined in D.6.2.5, D.6.2.6, and D.6.2.7, respectively. V_b is the basic concrete breakout strength value for a single anchor. A_{Vc} is the projected area of the failure surface on the side of the concrete member at its edge for a single anchor or a group of anchors. It shall be permitted to evaluate A_{Vc} as the base of a truncated half-pyramid projected on the side face of the member where the top of the half-pyramid is given by the axis of the anchor row selected as critical. The value of c_{a1} shall be taken as the distance from the edge to this axis. A_{Vc} shall not exceed nA_{Vco}, where n is the number of anchors in the group.

A_{Vco} is the projected area for a single anchor in a deep member with a distance from edges equal or greater than $1.5c_{a1}$ the direction perpendicular to the shear force. It shall be permitted to evaluate A_{Vco} as the base of a half-pyramid with a side length parallel to the edge of $3c_{a1}$ and a depth of $1.5c_{a1}$

$$A_{Vco} = 4.5(c_{a1})^2 \qquad \text{(D-22)}$$

Where anchors are located at varying distances from the edge and the anchors are welded to the attachment so as to distribute the force to all anchors, it shall be permitted to evaluate the strength based on the distance to the farthest row of anchors from the edge. In this case, it shall be permitted to base the value of c_{a1} on the distance from the edge to the axis of the farthest anchor row that is selected as critical, and all of the shear shall be assumed to be carried by this critical anchor row alone.

D.6.2.2 The basic concrete breakout strength in shear of a single anchor in cracked concrete, V_b, shall not exceed

$$V_b = 7\left(\frac{\ell_e}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f'_c}\,(c_{a1})^{1.5} \qquad \text{(D-23)}$$

where ℓ_e is the load-bearing length of the anchor for shear:

$\ell_e = h_{ef}$ for anchors with a constant stiffness over the full length of embedded section, such as headed studs are post-installed anchors with one tubular shell over full length of the embedment depth;

$\ell_e = h_{ef}$ for torque-controlled expansion anchors with a distance sleeve separated from expansion sleeve; and

in no case shall ℓ_e exceed $8d_o$.

D.6.2.3 For cast-in headed studs or headed bolts that are continuously welded to steel attachments having a minimum thickness equal to the greater of 3/8 in. or half of the anchor diameter, the basic concrete breakout strength in shear of a single anchor in cracked concrete, V_b, shall not exceed

$$V_b = 8\left(\frac{\ell_e}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f'_c}\,(c_{a1})^{1.5} \qquad \text{(D-24)}$$

where ℓ_e is defined in D.6.2.2, provided that:

(a) for groups of anchors, the strength is determined based on the strength of the row of anchors farthest from the edge;

(b) anchor spacing s is not less than 2.5 in.; and

(c) supplementary reinforcement is provided at the corners if $c_{a2} \le 1.5h_{ef}$.

D.6.2.4 Where anchors are influenced by three or more edges, the value of c_{a1} used in Eq. (D-20) through (D-27) shall not exceed the greatest of: $c_{a2}/1.5$ in either direction, $h_a/1.5$; and 1/3 of the maximum spacing between anchors within the group.

D.6.2.5 The modification factor for anchor groups loaded eccentrically in shear is

$$\psi_{ec,V} = \frac{1}{\left(1 + \frac{2e'_V}{3c_{a1}}\right)} \le 1 \qquad \text{(D-25)}$$

If the loading on an anchor group is such that only some anchors are loaded in shear in the same direction of the free edge, only those anchors that are loaded in shear in the direction of the free edge shall be considered when determining the eccentricity of e'_V for use in Eq. (D-25) and for the calculation of V_{cbg} in Eq. (D-21).

D.6.2.6 The modification factor for edge effect for a single anchor or group of anchors loaded in shear is

$$\psi_{ed,V} = 1.0 \text{ if } c_{a2} \ge 1.5c_{a1} \qquad \text{(D-26)}$$

$$\psi_{ed,V} = 0.7 + 0.3\frac{c_{a,2}}{1.5c_{a1}} \text{ if } c_{a,2} < 1.5c_{a1} \qquad \text{(D-27)}$$

D.6.2.7 For anchors located in a region of a concrete member where analysis indicates no cracking $(f_t < f_r)$ under the load combinations specified in 9.2 or C.2 with load factors taken as unity, the following modification factor shall be permitted

$$\psi_{c,V} = 1.4$$

For anchors located in a region of a concrete member where analysis indicates cracking under the load combinations specified in 9.2 or C.2 with load factors taken as unity, the following modification factors shall be permitted

$\psi_{c,V}$ = 1.0 for anchors in cracked concrete with no supplementary reinforcement or edge reinforcement smaller than a No. 4 bar;

$\psi_{c,V}$ = 1.2 for anchors in cracked concrete with supplementary reinforcement of a No. 4 bar or greater between the anchor and the edge; and

$\psi_{c,V}$ = 1.4 for anchors in cracked concrete with supplementary reinforcement of a No. 4 bar or greater between the anchor and the edge, and with the supplementary reinforcement enclosed within stirrups spaced at not more than 4 in.

To be considered as supplementary reinforcement, the reinforcement shall be designed to intersect the concrete breakout failure surface defined in D.6.2.1.

D.6.3 *Concrete pryout strength of anchor in shear*

D.6.3.1 The nominal pryout strength, V_{cp} or V_{cpg}, shall not exceed

(a) for a single anchor

$$V_{cp} = k_{cp}N_{cb} \tag{D-28}$$

(b) for a group of anchors

$$V_{cpg} = k_{cp}N_{cbg} \tag{D-29}$$

where

k_{cp} = **1.0** for h_{ef} < 2.5 in.; and

k_{cp} = **2.0** for $h_{ef} \geq$ 2.5 in.

N_{cb} and N_{cbg} shall be determined from Eq. (D-4) and (D-5), respectively.

D.7—Interaction of tensile and shear forces

Unless determined in accordance with D.4.3, anchors or groups of anchors that are subjected to both shear and axial loads shall be designed to satisfy the requirements of D.7.1 through D.7.3. The value of ϕN_n shall be as required in D.4.1.2. The value of ϕV_n shall be as defined in D.4.1.2.

D.7.1 If $V_{ua} \leq 0.2\phi V_n$, then full strength in tension shall be permitted: $\phi N_n \geq N_{ua}$.

D.7.2 If $N_{ua} \leq 0.2\phi N_n$, then full strength in shear shall be permitted: $\phi V_n \geq V_{ua}$.

D.7.3 If $V_{ua} > 0.2\phi V_n$ and $N_{ua} > 0.2\phi N_n$, then

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2 \tag{D-30}$$

D.8—Required edge distances, spacings, and thicknesses to preclude splitting failure

Minimum spacings and edge distances for anchors and minimum thicknesses of members shall conform to D.8.1 through D.8.6, unless supplementary reinforcement is provided to control splitting. Lesser values from product-specific tests performed in accordance with D.3.3 shall be permitted.

D.8.1 Minimum center-to-center spacing of anchors shall be $4d_o$ for untorqued cast-in anchors, and $6d_o$ for torqued cast-in anchors and post-installed anchors.

D.8.2 Minimum edge distances for cast-in headed anchors that will not be torqued shall be based on minimum cover requirements for reinforcement in 7.7. For cast-in headed anchors that will be torqued, the minimum edge distances shall be based on the greater of the minimum cover requirements for reinforcement in 7.7 or $6d_o$.

D.8.3 Minimum edge distances for post-installed anchors shall be based on the greater of the minimum cover requirements for reinforcement in 7.7, or the minimum edge distance requirements for the products as determined by tests in accordance with D.3.3, and shall not be less than 2.0 times the maximum aggregate size. In the absence of product-specific test information, the minimum edge distance shall be taken as not less than:

Undercut anchors .... $6d_o$

Torque-controlled anchors .... $8d_o$

Displacement-controlled anchors .... $10d_o$

D.8.4 Deleted section.

D.8.5 The value of h_{ef} for an expansion or undercut post-installed anchor shall not exceed the greater of 2/3 of the member thickness and the member thickness less 4 in.

D.8.6 Unless determined from tension tests in accordance with D.3.3, the critical edge distance c_{ac} shall not be taken less than:

Undercut anchors .... $2.5h_{ef}$

Torque-controlled anchors .... $4h_{ef}$

Displacement-controlled anchors .... $4h_{ef}$

D.8.7 Project drawings and project specifications shall specify use of anchors with a minimum edge distance as assumed in design.

D.9—Installation of anchors

D.9.1 Anchors shall be installed in accordance with the project drawings and project specifications and the requirements stipulated by the anchor manufacturer.

D.9.2 The engineer shall establish an inspection program to verify proper installation of the anchors.

D.9.3 The engineer shall establish a welding procedure to avoid excessive thermal deformation of an embedment that, if welded to the attachment, could cause spalling or cracking of the concrete or pullout of the anchor.

D.10—Structural plates, shapes, and specialty inserts

D.10.1 The design strength of embedded structural shapes, fabricated shapes, and shear lugs shall be determined based on fully yielded conditions, and using a ϕ in accordance with D.4.4 or D.4.5.

D.10.2 For structural shapes and fabricated steel sections, the web shall be designed for the shear, and the flanges shall be designed for the tension, compression, and bending.

D.10.3 The nominal strength of specialty inserts shall be based on the 5% fractile of results of tests performed and evaluated according to D.3. Embedment design shall be according to D.3 with strength-reduction factors according to D.4.4 or D.4.5.

D.11—Shear strength of embedded plates and shear lugs

D.11.1 *General*—The shear strength of grouted or cast-in embedments with shear lugs shall include consideration of the bearing strength of the concrete or grout placed against the shear lugs, the direct shear strength of the concrete or grout placed between shear lugs, and the confinement afforded by the tension anchors in combination with external loads acting across potential shear planes. Shear forces toward free edges and displacement compatibility between shear lugs shall also be considered. When multiple shear lugs are used to establish the design shear strength in a given direction, the magnitude of the allotted shear to each lug shall be in direct proportion to the total shear, the number of lugs, and the shear stiffness of each lug.

D.11.2 *Shear toward free edge*—For embedded plates and shear lugs bearing toward a free edge, unless reinforcement is provided to develop the required strength, the design shear strength for each lug or plate edge shall be determined based on a uniform tensile stress of $4\phi\sqrt{f_c'}$ acting on an effective stress area defined by projecting a 45-degree plane from bearing edges of the shear lug or base plate to the free surface. The bearing area of the shear lug or plate edge shall be excluded from the projected area. The ϕ-factor shall be taken in accordance with D.4.4 when using load combinations in 9.2 or in accordance with D.4.5 when using load combinations in C.2.

D.11.3 *Shear strength of embedments with embedded base plates*—For embedments having a base plate whose contact surface is below the surface of concrete, shear strength shall be permitted to be calculated using the shear-friction provisions of 11.7 (as modified by this section), using the following shear-friction coefficients:

Base plate without shear lugs.. 0.9
Base plate with shear lugs
that is designed to remain elastic 1.4

The tension anchor steel area required to resist external loads shall be added to the tension anchor steel area required due to shear friction.

D.12—Grouted embedments

D.12.1 Grouted embedments shall meet the applicable requirements of this appendix.

D.12.2 For general grouting purposes, the material requirements for cement grout shall be in accordance with Chapter 3. The use of special grouts, containing epoxy or other binding media, or those used to achieve properties such as high strength, low shrinkage or expansion, or early strength gain, shall be qualified for use by the engineer and specified in the contract documents.

D.12.3 Grouted embedments shall be tested to verify embedment strength. Grouted embedments installed in tension zones of concrete members shall be capable of sustaining design strength in cracked concrete. Tests shall be conducted by an independent testing agency and shall be certified by a licensed professional engineer with full description and details of the testing programs, procedures, results, and conclusions.

D.12.4 Grouted embedments shall be tested for the installed condition by testing randomly selected grouted embedments to a minimum of 100% of the required strength. The testing program shall be established by the engineer.

D.12.5 The tests required by D.12.3 and D.12.4 shall be permitted to be waived by the engineer if tests and installation data are available to demonstrate that the grouted embedment will function as designed or if the load transfer through the grout is by direct bearing or compression.

APPENDIX RD—ANCHORING TO CONCRETE COMMENTARY

References to hooked bolts (J- or L-bolt), as discussed in ACI 318-05, have been deleted because these anchors do not have a ductile failure mode, which is strongly recommended for anchors used in nuclear safety-related structures.

RD.1—Definitions

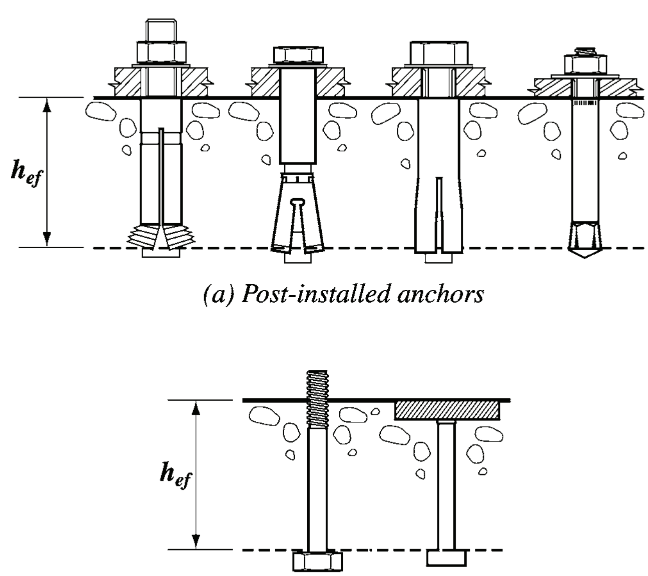
brittle steel element and **ductile steel element**—the 14% elongation should be measured over the gage length specified in the appropriate ASTM standard for the steel.

five percent fractile—the determination of the coefficient K_{05} associated with the 5% fractile, $\bar{x} - K_{05}S_s$, depends on the number of tests, n, used to compute $\bar{x}$ and S_s. Values of K_{05} range, for example, from 1.645 for $n = \infty$, to 2.010 for n = 40, and 2.568 for n = 10.

RD.2—Scope

RD.2.1 ACI 349 uses the term "embedments" to cover a broad scope that includes anchors, embedded plates, shear lugs, grouted embedments, and specialty inserts. It covers the same scope that is described in the 2001 Code.

RD.2.3 Typical cast-in headed studs and headed bolts with geometries consistent with ANSI/ASME B1.1,[D.1] B18.2.1,[D.2]



(a) Post-installed anchors

(b) Cast-in-place anchors

Fig. RD.1—Types of fasteners.

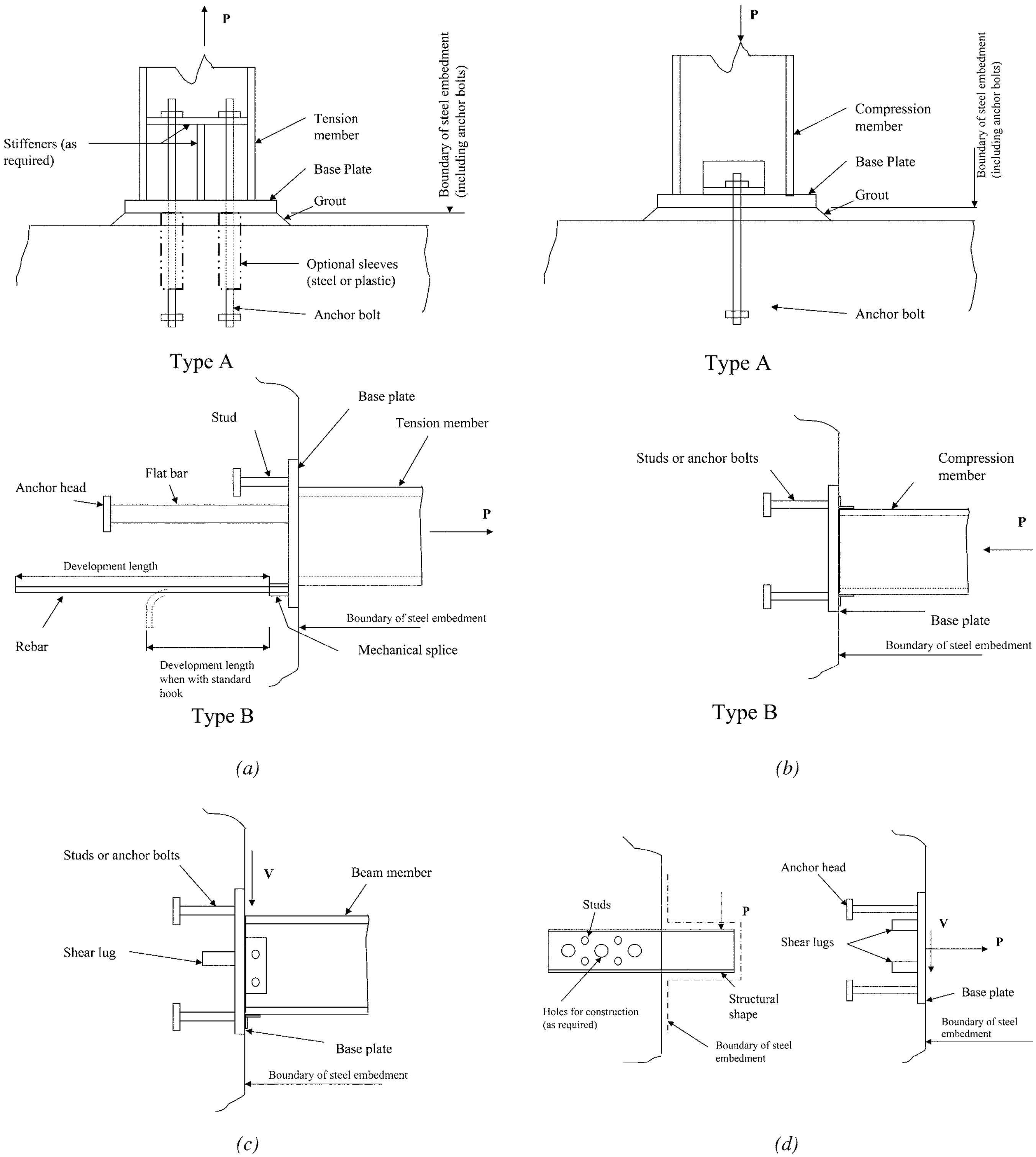


Fig. RD2.6—(a) Typical embedments for tension loads; (b) typical embedments for compressive loads; (c) typical embedments for shear loads; and (d) typical embedments for combined loads.

and B18.2.6[D.3] have been tested and proven to behave predictably, so calculated pullout values are acceptable. Post-installed anchors do not have predictable pullout capacities and, therefore, are required to be tested.

RD.2.6 Typical embedment configurations are shown in Fig. RD.2.6(a), (b), (c), and (d). These figures also indicate the extent of the embedment within the jurisdiction of this Code.

RD.3—General requirements

RD.3.1 When the strength of an anchor group is governed by breakage of the concrete, the behavior is brittle and there is limited redistribution of the forces between the highly-stressed and less-stressed anchors. In this case, the theory of elasticity is required to be used assuming the attachment that distributes loads to the anchors is sufficiently stiff. The forces in the anchors are considered to be proportional to the external load and its distance from the neutral axis of the anchor group.

If anchor strength is governed by ductile yielding of the anchor steel, significant redistribution of anchor forces can occur. In this case, an analysis based on the theory of elasticity will be conservative. References D.4 to D.6 discuss nonlinear analysis, using theory of plasticity, for the determination of the strengths of ductile anchor groups.

RD.3.3 Many anchors in a nuclear power plant must perform as designed with high confidence, even when exposed to significant seismic loads. To prevent unqualified anchors from being used in connections that must perform with high confidence under significant seismic load, all anchors are required to be qualified for seismic zone usage by satisfactory performance in passing simulated seismic tests. The qualification should be performed consistent with the provisions of this appendix and should be reviewed by a licensed professional engineer experienced in anchor technology. Typical simulated seismic-testing methods are described in Reference D.7. For a post-installed anchor to be used in conjunction with the requirements of this appendix, the results of tests have to indicate that pullout failures exhibit an acceptable load-displacement characteristic, or that pullout failures are precluded by another failure mode. ACI 349 requires that all post-installed anchors be qualified, by independent tests, for use in cracked concrete. Post-installed anchors qualified to the procedures of ACI 355.2 as Category I for use in cracked concrete are considered acceptable for use in nuclear power plants. Anchors qualified for use only in uncracked concrete are not recommended in nuclear power plant structures.

The design of the anchors for impactive or impulsive loads is not checked directly by simulated seismic tests. An anchor that has passed the simulated seismic tests, however, should function under impactive tensile loading in cracked concrete.

RD.3.4 The provisions of Appendix D are applicable to normalweight concrete. The design of anchors in heavyweight concrete should be based on testing for the specific heavyweight concrete.

RD.3.5 A limited number of tests of cast-in-place and post-installed anchors in high-strength concrete[D.8] indicate that the design procedures contained in this appendix become unconservative, particularly for cast-in anchors, at f_c' = 11,000 to 12,000 psi. Until further tests are available, an upper limit of f_c' = 10,000 psi has been imposed in the design of cast-in-place anchors. This is consistent with Chapters 11 and 12. The ACI 355.2 test method does not require testing of post-installed anchors in concrete with f_c' greater than 8000 psi because some post-installed anchors may have difficulty expanding in very high-strength concretes. Because of this, f_c' is limited to 8000 psi in the design of post-installed anchors unless testing is performed.

RD.3.6.1 The design provisions of ACI 349, Appendix D, for anchors in nuclear power plants, retain the philosophy of previous editions of ACI 349 by encouraging anchor designs to have a ductile-failure mode. This is consistent with the strength-design philosophy of reinforced concrete in flexure. The failure mechanism of the anchor is controlled by requiring yielding of the anchor prior to a brittle concrete failure. A ductile design provides greater margin than a nonductile design because it permits redistribution of load to adjacent anchors and can reduce the maximum dynamic load by energy absorption and reduction in stiffness. For such cases, the design strength is the nominal strength of the steel, multiplied by a strength-reduction factor of 0.75 if load combinations in 9.2 are used or 0.80 if load combinations in Appendix C are used.

The nominal tensile strength of the embedment should be determined based on those portions of the embedment that transmit tension or shear loads into the concrete. It is not necessary to develop an embedment for full axial tension and full shear if it can be demonstrated that the embedment will be subjected to one type of loading (such as tension, shear, or flexure). An embedment need not be developed for tension or shear if the load is less than 20% of the full tension or shear strength. This value of 20% is consistent with the value of 20% used in the equation in D.7.

An embedment may be considered subject to flexure only when the axial tension loads on the embedment are less than 20% of the nominal strength in tension.

RD.3.6.2 A ductile design can also be achieved by designing the attachment to yield before failure of the anchors. In such a case, the anchors can be nonductile as long as they are stronger than the yield strength of the attachment. This is established with a margin equivalent to that in D.3.6.1. D.3.6.2 is based on attachment yield strength f_y, whereas D.3.6.1 uses f_{ut} because attachments are typically of ASTM A 36 material, and the strength is better characterized by the yield strength. The 0.75 factor allows for the actual yield strength versus specified minimum yield strength.

RD.3.6.3 There are situations where a ductile-failure mode cannot be achieved. It is permissible to design anchors as ductile for one loading but nonductile for the other. Previous editions of ACI 349 included specific provisions for commercially available, nonductile expansion anchors that were penalized by specifying a lower strength-reduction factor. The current Appendix D includes more general provisions for anchors for which a ductile-failure mode cannot be achieved. Such situations can occur for anchors in shallow slabs, close to edges, or close to other anchors. The factor of 0.60 is specified to account for the lower margins inherent in a nonductile design relative to those in a ductile design.

RD.3.8 Ductile steel elements are defined in D.1 to have a minimum elongation of 14%. This requirement is meant to ensure sufficient ductility in the embedment steel. The limit of 14% is based on ASTM A 325[D.9] and A 490[D.10] anchor materials that have been shown to behave in a ductile manner when used for embedment steel.

RD.3.9 Anchors that incorporate a reduced section (such as threads, notch, or wedge) in the load path (the term "load path" includes the tension load path and the shear load path) may fail in the reduced section before sufficient inelastic deformation has occurred to allow redistribution of anchor tension and shear forces, thus exhibiting low ductility. This can be prevented by requirement (a), which ensures that yielding of the unreduced section will occur before failure of the reduced section. Shear failure can be affected significantly by reduced sections within five anchor diameters of the shear plane (many wedge-type anchors). In this case, tests for the evaluation of the shear strength are required. Tests reported in Reference D.4 for a limited number of attachment types, steel strength, and diameters have shown that threaded anchors will exhibit sufficient ductility to redistribute tension and shear forces.

RD.3.10 The design provisions for impulsive and impactive loads in Appendix F may be used for embedments. Energy can be absorbed by deformation of anchors designed for ductile steel failure.

RD.4—General requirements for strength of anchors

RD.4.1 This section provides requirements for establishing the strength of anchors to concrete. The various types of steel and concrete failure modes for anchors are shown in Fig. RD.4.1(a) and (b). Comprehensive discussions of anchor failure modes are included in References D.11 to D.13. Any model that complies with the requirements of D.4.2 and D.4.3 can be used to establish the concrete-related strengths. For anchors such as headed bolts, headed studs, and post-installed anchors, the concrete breakout design methods of D.5.2 and D.6.2 are acceptable. The anchor strength is also dependent on the pullout strength of D.5.3, the side-face blowout strength of D.5.4, and the minimum spacings and edge distances of D.8. The design of anchors for tension recognizes that the strength of anchors is sensitive to appropriate installation; installation requirements are included in D.9. Some post-installed anchors are less sensitive to installation errors and tolerances. This is reflected in varied ϕ-factors based on the assessment criteria of ACI 355.2.

Test procedures can also be used to determine the single-anchor breakout strength in tension and in shear. The test results, however, are required to be evaluated on a basis statistically equivalent to that used to select the values for the concrete breakout method "considered to satisfy" provisions of D.4.2. The basic strength cannot be taken greater than the 5% fractile. The number of tests has to be sufficient for statistical validity and should be considered in the determination of the 5% fractile.

RD.4.2 and **RD.4.3**—D.4.2 and D.4.3 establish the performance factors for which anchor design models are required to be verified. Many possible design approaches exist, and the designer is always permitted to "design by test" using D.4.2 as long as sufficient data are available to verify the model.

RD.4.2.1 The addition of supplementary reinforcement in the direction of the load, confining reinforcement, or both, can greatly enhance the strength and ductility of the anchor connection. Such enhancement is practical with cast-in anchors such as those used in precast sections.

The shear strength of headed anchors located near the edge of a member can be significantly increased with appropriate supplementary reinforcement. References D.11, D.14, and D.15 provide substantial information on design of such reinforcement. The effect of such supplementary reinforce-

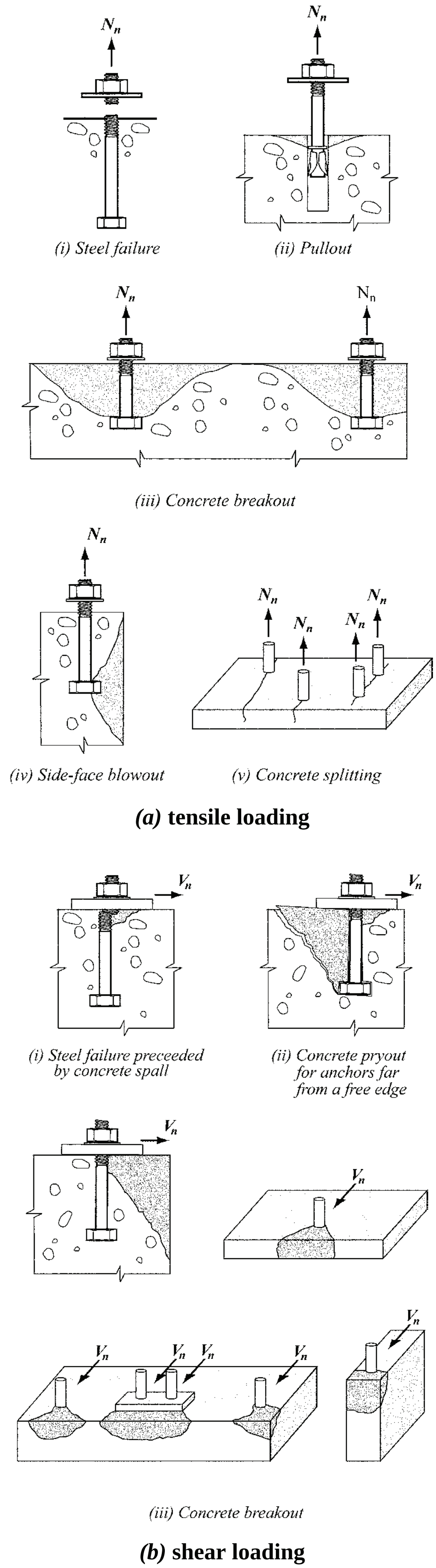


Fig. RD.4.1—Failure modes for anchors.

ment is not included in the ACI 355.2 anchor acceptance tests or in the concrete breakout calculation method of D.5.2 and D.6.2. The designer has to rely on other test data and design theories to include the effects of supplementary reinforcement.

For anchors exceeding the limitations of D.4.2.2, or for situations where geometric restrictions limit concrete breakout strength, or both, reinforcement oriented in the direction of load and proportioned to resist the total load within the breakout prism, and fully anchored on both sides of the breakout planes, may be provided instead of calculating concrete breakout strength.

The concrete breakout strength of an unreinforced connection can be taken as an indication of the load at which significant cracking will occur. Such cracking can represent a serviceability problem if not controlled. (See RD.6.2.1.)

RD.4.2.2 The method for determining concrete breakout strength included as "considered to satisfy" D.4.2 was developed from the concrete capacity design (CCD) method,[D.12,D.13] which was an adaptation of the κ method[D.16,D.17] and is considered to be accurate, relatively easy to apply, and capable of extension to irregular layouts. The CCD method predicts the strength of an anchor or group of anchors by using a basic equation for tension, or for shear for a single anchor in cracked concrete, and multiplied by factors that account for the number of anchors, edge distance, spacing, eccentricity and absence of cracking. The limitations on anchor size and embedment length are based on the current range of test data.

The concrete breakout strength calculations are based on a model suggested in the κ method. It is consistent with a breakout prism angle of approximately 35 degrees (Fig. RD.4.2.2(a) and (b)).

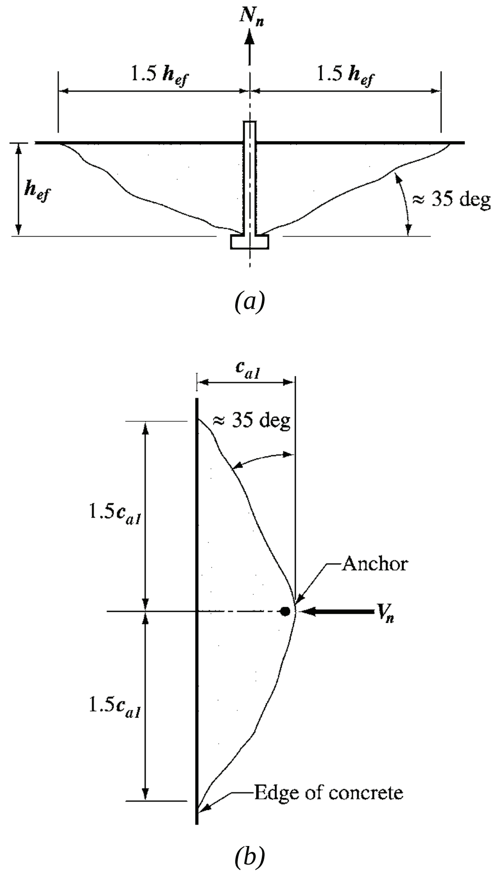


Fig. RD.4.2.2—Breakout cone for: (a) tension; and (b) shear.

RD.4.4 The ϕ-factors for steel strength are based on using f_{ut} to determine the nominal strength of the anchor (see D.5.1 and D.6.1) rather than f_y as used in the design of reinforced concrete members. Although the ϕ-factors for use with f_{ut} appear low, they result in a level of safety consistent with the use of higher ϕ-factors applied to f_y. The smaller ϕ-factors for shear than for tension do not reflect basic material differences but rather account for the possibility of a non-uniform distribution of shear in connections with multiple anchors. It is acceptable to have a ductile failure of a steel element in the attachment if the attachment is designed so that it will undergo ductile yielding at a load level no greater than 75% of the minimum design strength of an anchor (see D.3.6.2). For anchors governed by the more brittle concrete breakout or blowout failure, two conditions are recognized. If supplementary reinforcement is provided to tie the failure prism into the structural member (Condition A), more ductility is present than in the case where such supplementary reinforcement is not present (Condition B). Design of supplementary reinforcement is discussed in RD.4.2.1 and References D.11, D.14, and D.15. Further discussion of strength-reduction factors is presented in RD.4.5.

The strengths of anchors under shear forces are not as sensitive to installation errors and tolerances. Therefore, for shear calculations of all anchors, ϕ = **0.70**.

RD.4.5 As noted in R9.1, the 2006 Code incorporated the load factors of SEI/ASCE 7-02 and the corresponding strength-reduction factors provided in the ACI 318-99 Appendix C into Section 9.2 and 9.3, except that the factor for flexure has been increased. Investigative studies for the ϕ-factors to be used for Appendix D were based on the ACI 349-01 and 9.2 and 9.3 load and strength-reduction factors. The resulting ϕ-factors are presented in D.4.5 for use with the load factors of the 2006 Appendix C. The ϕ-factors for use with the load factors of the ACI 318-99, Appendix C were determined in a manner consistent with the other ϕ-factors of the ACI 318-99 Appendix C. These ϕ-factors are presented in D.4.4 for use with the load factors of 2006 Section 9.2. Since investigative studies for ϕ-factors to be used with Appendix D, for brittle concrete failure modes, were performed for the load and strength-reduction factors now given in Appendix C, the discussion of the selection of these ϕ-factors appears in this section.

Even though the ϕ-factor for plain concrete in Appendix C uses a value of 0.65, the basic factor for brittle concrete failures (ϕ = 0.75) was chosen based on results of probabilistic studies[D.18] that indicated the use of ϕ = 0.65 with mean values of concrete-controlled failures produced adequate safety levels. Because the nominal resistance expressions used in this appendix and in the test requirements are based on the 5% fractiles, the ϕ = 0.65 value would be overly conservative. Comparison with other design procedures and probabilistic studies[D.18] indicated that the choice of ϕ = 0.75

was justified. For applications with supplementary reinforcement and more ductile failures (Condition A), the ϕ-factors are increased. The value of $\phi = 0.85$ is compatible with the level of safety for shear failures in reinforced concrete beams, and has been recommended in the *PCI Design Handbook*[D.19] and by earlier versions of ACI 349.[D.20]

RD.4.6 *Bearing strength*

RD.4.6.1 D.4.5.1 prohibits the designer from combining shear strength of bearing (for example, a shear lug) and shear friction (such as shear studs) mechanisms. This exclusion is justified in that it is difficult to predict the distribution of shear resistance as a result of differential stiffness of the two mechanisms. This exclusion is required because of the displacement incompatibility of these two independent and nonconcurrent mechanisms. Tests show that the relatively smaller displacements associated with the bearing mode preclude development of the shear-friction mode until after bearing mode failure.[D.21] As described in RD.11.1, however, the confining forces afforded by the tension anchors in combination with other concurrent external loads acting across potential shear planes can result in a significant and reliable increase in bearing mode shear strength and can therefore be used.

RD.4.6.2 For shear lugs, the nominal bearing strength value of $\mathbf{1.3}f_c'$ is recommended based on the tests described in Reference D.21 rather than the general provisions of 10.15. The factor of 0.65 corresponds to that used for bearing on concrete in Chapter 9. The factor of 0.70 corresponds to that used for bearing on concrete in Appendix C.

RD.5—Design requirements for tensile loading

RD.5.1 *Steel strength of anchor in tension*

RD.5.1.2 The nominal tensile strength of anchors is best represented by $A_{se} f_{uta}$ rather than $A_{se} f_{ya}$ because the large majority of anchor materials do not exhibit a well-defined yield point. The American Institute of Steel Construction (AISC) has based tensile strength of anchors on $A_{se} f_{uta}$ since the 1986 edition of their specifications. The use of Eq. (D-3) with Section 9.2 load factors and the ϕ-factors of D.4.4 give design strengths consistent with the "AISC Load and Resistance Factor Design Specifications for Structural Steel Buildings."[D.22]

The limitation of $\mathbf{1.9}f_{ya}$ on f_{uta} is to ensure that, under service load conditions, the anchor does not exceed f_{ya}. The limit on f_{uta} of $\mathbf{1.9}f_{ya}$ was determined by converting the LRFD provisions to corresponding service load conditions. For Section 9.2, the average load factor of 1.4 (from $\mathbf{1.2}D + \mathbf{1.6}L$) divided by the highest ϕ-factor (0.75 for tension) results in a limit of f_{ut}/f_y of 1.4/0.75 = 1.87. For Appendix C, the average load factor of 1.55 (from $\mathbf{1.4}D + \mathbf{1.7}L$), divided by the highest ϕ-factor (0.80 for tension), results in a limit f_{uta}/f_{ya} of 1.55/0.8 = 1.94. For consistent results, the serviceability limitation of f_{uta} was taken as $\mathbf{1.9}f_{ya}$. If the ratio of f_{uta} to f_{ya} exceeds this value, the anchoring may be subjected to service loads above f_{ya}. Although not a concern for standard structural steel anchors (maximum value of f_{uta}/f_{ya} is 1.6 for ASTM A 307[D.23]), the limitation is applicable to some stainless steels.

RD.5.2 *Concrete breakout strength of anchors in tension*

RD.5.2.1 The effects of multiple anchors, spacing of anchors, and edge distance on the nominal concrete breakout strength in tension are included by applying the modification factors A_{Nc}/A_{Nco} and $\psi_{ed,N}$ in Eq. (D-4) and (D-5).

Figure RD.5.2.1(a) shows A_{Nco} and the development of Eq. (D-6). A_{Nco} is the maximum projected area for a single anchor. Figure RD.5.2.1(b) shows examples of the projected areas for various single-anchor and multiple-anchor arrangements. Because A_{Nc} is the total projected area for a group of anchors, and A_{Nco} is the area for a single anchor, there is no need to include n, the number of anchors, in Eq. (D-4) or (D-5). If anchor groups are positioned in such a way that their projected areas overlap, the value of A_{Nc} is required to be reduced accordingly.

RD.5.2.2 The basic equation for anchor strength was derived[D.12-D.14,D.17] assuming a concrete failure prism with an angle of approximately 35 degrees, considering fracture mechanics concepts.

The values of k_c in Eq. (D-7) were determined from a large database of test results in uncracked concrete[D.12] at the 5% fractile. The values were adjusted to corresponding k_c values for cracked concrete.[D.13,D.24] Higher k_c values for post-installed anchors may be permitted, provided they have been determined from product approval testing in accordance with ACI 355.2. For anchors with a deep embedment (h_{ef} > 11 in.), test evidence indicates the use of $h_{ef}^{1.5}$ can be overly conservative for some cases. Often such tests have been with selected aggregates for special applications. An alternative expression (Eq. (D-8)) is provided using $h_{ef}^{5/3}$ for evaluation of cast-in anchors with 11 in. $\leq h_{ef} \leq$ 25 in. The limit of 25 in. corresponds to the upper range of the test data. This expression can also be appropriate for some undercut post-installed anchors. D.4.2, however, should be used with test results to justify such applications.

RD.5.2.3 For anchors located less than $\mathbf{1.5}h_{ef}$ from three or more edges, the tensile breakout strength computed by the CCD method, which is the basis for Eq. (D-4) to (D-11), gives misleading results.[D.25] This occurs because the ordinary definitions of A_{Nc}/A_{Nco} do not correctly reflect the edge effects. This problem is corrected by limiting the value of h_{ef} used in Eq. (D-4) to (D-11) to $c_{a,max}/\mathbf{1.5}$, where $c_{a,max}$ is the largest of the influencing edge distances that are less than or equal to the actual $\mathbf{1.5}h_{ef}$, but not less than 1/3 of the maximum spacing between anchors for anchor groups. The limit on h_{ef} of not less than 1/3 of the maximum spacing between anchors for anchor groups prevents the designer from using a calculated strength based on individual breakout prisms for a group anchor configuration.

This approach is illustrated in Figure RD.5.2.3. In this example, the proposed limit on the value of h_{ef} to be used in the computations where $h_{ef} = c_{a,max}/\mathbf{1.5}$, results in $h_{ef} = h'_{ef}$ = 4 in. For this example, this would be the proper value to be used for h_{ef} in computing the resistance even if the actual embedment depth is larger.

The requirement of D.5.2.3 may be visualized by moving the actual concrete breakout surface originating at the actual h_{ef} toward the surface of the concrete perpendicular to the

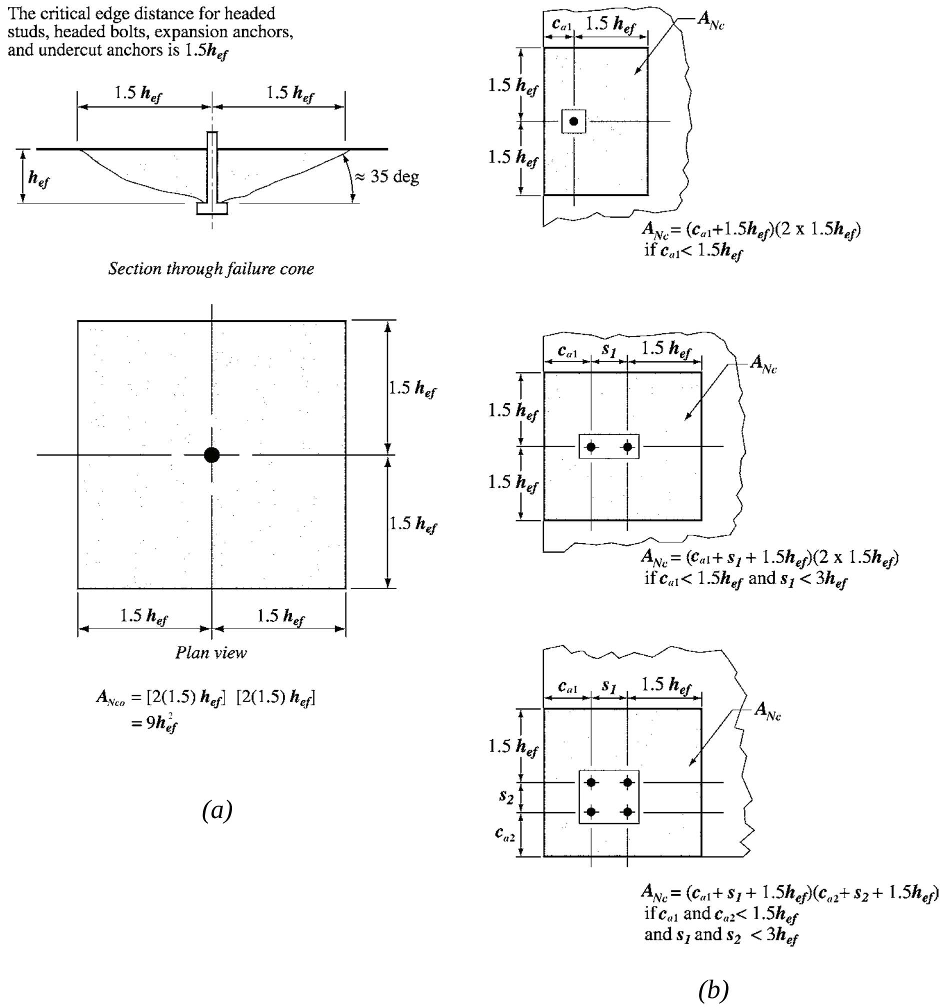


Fig. RD.5.2.1—(a) Calculation of $\mathbf{A_{Nco}}$*; and (b) projected areas for single anchors and groups of anchors and calculation of* $\mathbf{A_{Nc}}$.

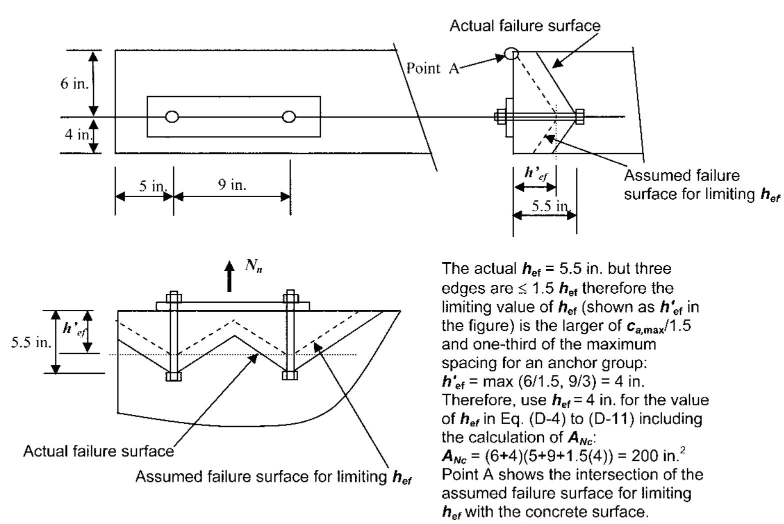


Fig. RD.5.2.3—Tension in narrow members.

applied tension load. The limit on h_{ef} for use in Eq. (D-4) to (D-11) occurs when either the outer boundaries of the failure surface first intersect a free edge or when the intersection of the breakout surface originating between anchors within the group first intersects the surface of the concrete. For the example shown in Fig. RD.5.2.3, Point A shows the controlling intersection.

RD.5.2.4 Figure RD.5.2.4(a) shows a group of anchors that are all in tension but that have a resultant force eccentric with respect to the centroid of the anchor group. Groups of anchors can be loaded in such a way that only some of the anchors are in tension [Fig. RD.5.2.4(b)]. In this case, only the anchors in tension are to be considered in the determination of e'_N. The anchor loading has to be determined as the resultant anchor tension at an eccentricity with respect to the center of gravity of the anchors in tension.

RD.5.2.5 If anchors are located close to an edge so that there is not enough space for a complete breakout prism to develop, the load-bearing strength of the anchor is further reduced beyond that reflected in A_{Nc}/A_{Nco}. If the smallest side cover distance is greater than $1.5h_{ef}$, a complete prism can form and there is no reduction ($\psi_{ed,N} = 1$). If the side cover is less than $1.5h_{ef}$, the factor $\psi_{ed,N}$ is required to adjust for the edge effect.[D.12]

RD.5.2.6 The analysis for the determination of crack formation should include the effects of restrained shrinkage (see 7.12.1.2), and should consider all specified load combinations using unfactored loads. The anchor qualification tests of ACI 355.2 require that anchors in cracked concrete zones perform well in a crack that is 0.012 in. wide. If wider cracks are expected, confining reinforcement to control the crack width to approximately 0.012 in. should be provided.

The concrete breakout strengths given by Eq. (D-7) and (D-8) assume cracked concrete (that is, $\psi_{c,N}$ = 1.0) with $\psi_{ed,N}\, k_c$ = 24 for cast-in-place, and 17 for post-installed (cast-in 40% higher). When the uncracked concrete $\psi_{c,N}$ factors are applied (1.25 for cast-in, and 1.4 for post-installed), the results are $\psi_{ed,N}\, k_c$ factors of 30 for cast-in and 24 for post-installed (25% higher for cast-in). This agrees with field observations and tests that show cast-in anchor strength exceeds that of post-installed for both cracked and uncracked concrete.

When k_c used in Eq. (D-7) is taken from the ACI 355.2 product evaluation report for post-installed anchors approved for use in both cracked and uncracked concrete, the value of both k_c and $\psi_{c,N}$ is based on the ACI 355.2 product evaluation report.

For post-installed anchors approved for use only in uncracked concrete in accordance with ACI 355.2, the value of k_c in the ACI 355.2 product evaluation report is used in Eq. (D-7), and $\psi_{c,N}$ shall be 1.0.

RD.5.2.7 The design provisions in D.5 are based on the assumption that the basic concrete breakout strength can be achieved if the minimum edge distance $c_{a,min}$ equals $1.5h_{ef}$. However, test results[D.26] indicate that many torque-controlled and displacement-controlled expansion anchors and some undercut anchors require minimum edge distances exceeding $1.5h_{ef}$ to achieve the basic concrete breakout

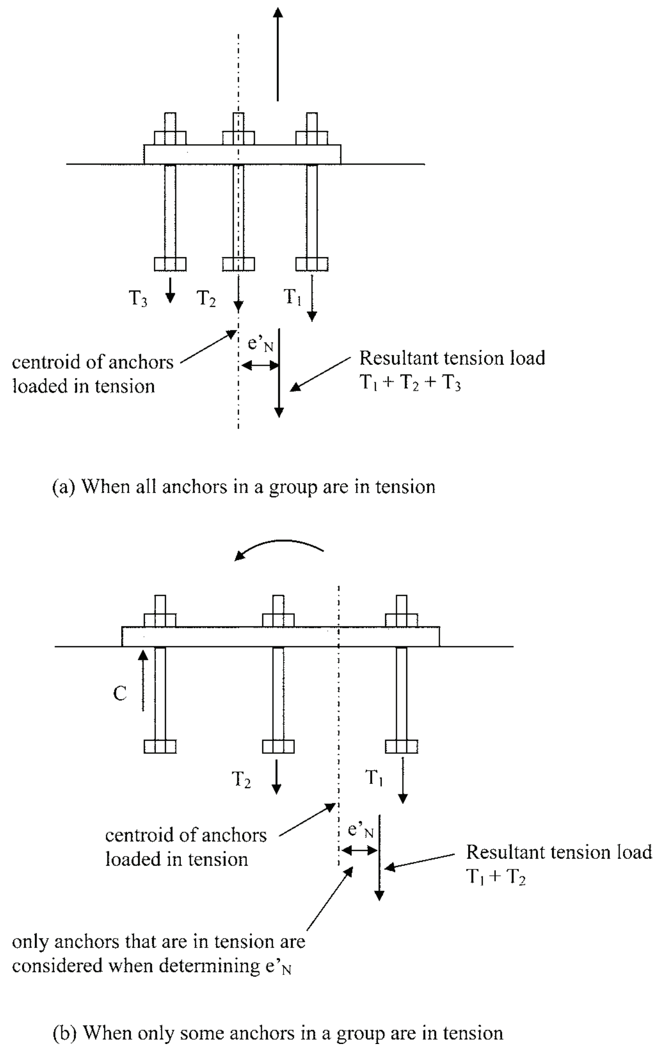


Fig. RD.5.2.4—Definition of $\mathbf{e'_N}$ for group anchors.

strength when tested in uncracked concrete without supplementary reinforcement to control splitting. When a tension load is applied, the resulting tensile stresses at the embedded end of the anchor are added to the tensile stresses induced due to anchor installation, and a splitting failure may occur before reaching the concrete breakout strength defined in D.5.2.1. To account for this potential splitting mode of failure, the basic concrete breakout strength is reduced by a factor $\psi_{cp,N}$ if $c_{a,min}$ is less than the critical edge distance c_{ac}. If supplementary reinforcement to control splitting is present or if the anchors are located in a region where analysis indicates cracking of the concrete at service loads, then the reduction factor $\psi_{cp,N}$ is taken as 1.0. The presence of supplementary reinforcement to control splitting does not affect the selection of Condition A or B in D.4.4 or D.4.5.

RD.5.2.8 In the future, there are expected to be more expansion and undercut anchors that are to be calculated with the k-value for headed studs. Tests with one special undercut anchor have shown that this is possible.

RD.5.3 *Pullout strength of anchor in tension*

RD.5.3.2 The pullout strength equations given in D.5.3.4 and D.5.3.5 are only applicable to cast-in headed anchors.[D.11]

They are not applicable to expansion and undercut anchors that use various mechanisms for end anchorage unless the validity of the pullout strength equations are verified by tests.

RD.5.3.3 The pullout strength in tension of headed studs or headed bolts can be increased by providing confining reinforcement, such as closely spaced spirals, throughout the head region. This increase can be demonstrated by tests.

RD.5.3.4 Equation (D-15) corresponds to the load at which the concrete under the anchor head begins to crush.[D.11] It is not the load required to pull the anchor completely out of the concrete, so the equation contains no term relating to embedment depth. The designer should be aware that local crushing under the head will greatly reduce the stiffness of the connection, and generally will be the beginning of a pullout failure.

RD.5.4 *Concrete side-face blowout strength of a headed anchor in tension*—The design requirements for side-face blowout are based on the recommendations of Reference D.27. Side-face blowout may control when the anchor is close to an edge **($c < 0.4h_{ef}$)**. These requirements are applicable to headed anchors that usually are cast-in anchors. Splitting during installation rather than side-face blowout generally governs post-installed anchors, and is evaluated by the ACI 355.2 requirements. When a group of anchors is close to an edge, side-face blowout will be controlled by the row of anchors closest to the edge. The anchors away from the edge will have greater strength than those closest to the edge. The side-face blowout of the group is conservatively calculated using the strength of the anchors closest to the edge.

RD.6—Design requirements for shear loading

RD.6.1 *Steel strength of anchor in shear*

RD.6.1.2 The nominal shear strength of anchors is best represented by $A_{se} f_{uta}$ for headed stud anchors and $0.6A_{se} f_{uta}$ for other anchors rather than a function of $A_{se} f_{ya}$ because typical anchor materials do not exhibit a well-defined yield point. The use of Eq. (D-18) and (D-19) with Section 9.2 load factors and the ϕ-factors of D.4.4 give design strengths consistent with the "AISC Load and Resistance Factor Design Specifications for Structural Steel Buildings."[D.22]

The limitation of $1.9f_y$ on f_{uta} is to ensure that, under service load conditions, the anchor stress does not exceed f_{ya}. The limit on f_{uta} of $1.9f_{ya}$ was determined by converting the LRFD provisions to corresponding service load conditions as discussed in RD.5.1.2.

RD.6.1.3 The shear strength of a grouted base plate is based on limited testing. It is recommended that the height of the grout pad not exceed 2 in.

RD.6.1.4 The friction force that develops between the base plate and concrete due to the compressive resultant from moment or axial load or both contributes to the shear strength of the connection. For as-rolled base plates installed against hardened concrete, the coefficient of friction is approximately 0.40.[D.4]

If the frictional strength is larger than the applied shear force, the base plate will not slip. When the frictional strength is less than the applied shear, the shear resistance will be a combination of both frictional strength and shear strength provided by the anchors. It must be assured that the compressive resultant used in determining the frictional resistance acts concurrent with the shear force. The presence or absence of loads should satisfy Section 9.2.3. Compressive resultants due to secondary loads should not be considered.

RD.6.2 *Concrete breakout strength of anchor in shear*

RD.6.2.1 The shear strength equations were developed from the CCD method. They assume a breakout cone angle of approximately 35 degrees (refer to Fig. RD.4.2.2(b)), and consider fracture mechanics theory. The effects of multiple anchors, spacing of anchors, edge distance, and thickness of the concrete member on nominal concrete breakout strength in shear are included by applying the reduction factor A_{Vc}/A_{Vco} in Eq. (D-20) and (D-21), and $\psi_{ec,V}$ in Eq. (D-21). For anchors far from the edge, D.6.2 usually will not govern. For these cases, D.6.1 and D.6.3 often govern.

Figure RD.6.2.1(a) shows A_{Vco} and the development of Eq. (D-22). A_{Vco} is the maximum projected area for a single anchor that approximates the surface area of the full breakout prism or cone for an anchor unaffected by edge distance, spacing or depth of member. Figure RD.6.2.1(b) shows examples of the projected areas for various single anchor and multiple anchor arrangements. A_{Vc} approximates the full surface area of the breakout cone for the particular arrangement of anchors. Because A_{Vc} is the total projected area for a group of anchors, and A_{Vco} is the area for a single anchor, there is no need to include the number of anchors in the equation.

When using Eq. (D-21) for anchor groups loaded in shear, both assumptions for load distribution illustrated in examples on the right side of Fig. RD.6.2.1(b) should be considered because the anchors nearest the edge could fail first or the whole group could fail as a unit with the failure surface originating from the anchors farthest from the edge. If the anchors are welded to a common plate, when the anchor nearest the front edge begins to form a failure cone, shear load would be transferred to the stiffer and stronger rear anchor. For this reason, anchors welded to a common plate do not need to consider the failure mode shown in the upper right figure of Fig. RD.6.2.1(b). The *PCI Design Handbook* approach[D.19] suggests in Section 6.5.2.2 that the strength of the anchors away from the edge be considered. Because this is a reasonable approach, assuming that the anchors are spaced far enough apart so that the shear failure surfaces do not intersect,[D.14] D.6.2 allows such a procedure. If the failure surfaces do not intersect, as would generally occur if the anchor spacing s is equal to or greater than $1.5c_{a1}$, then after formation of the near-edge failure surface, the higher strength of the farther anchor would resist most of the load. As shown in the bottom right example in Fig. RD.6.2.1(b), it would be appropriate to consider the full shear strength to be provided by this anchor with its much larger resisting failure surface. No contribution of the anchor near the edge is then considered. Checking the near-edge anchor condition is advisable to preclude undesirable cracking at service load conditions. Further discussion of design for multiple anchors is given in Reference D.11.

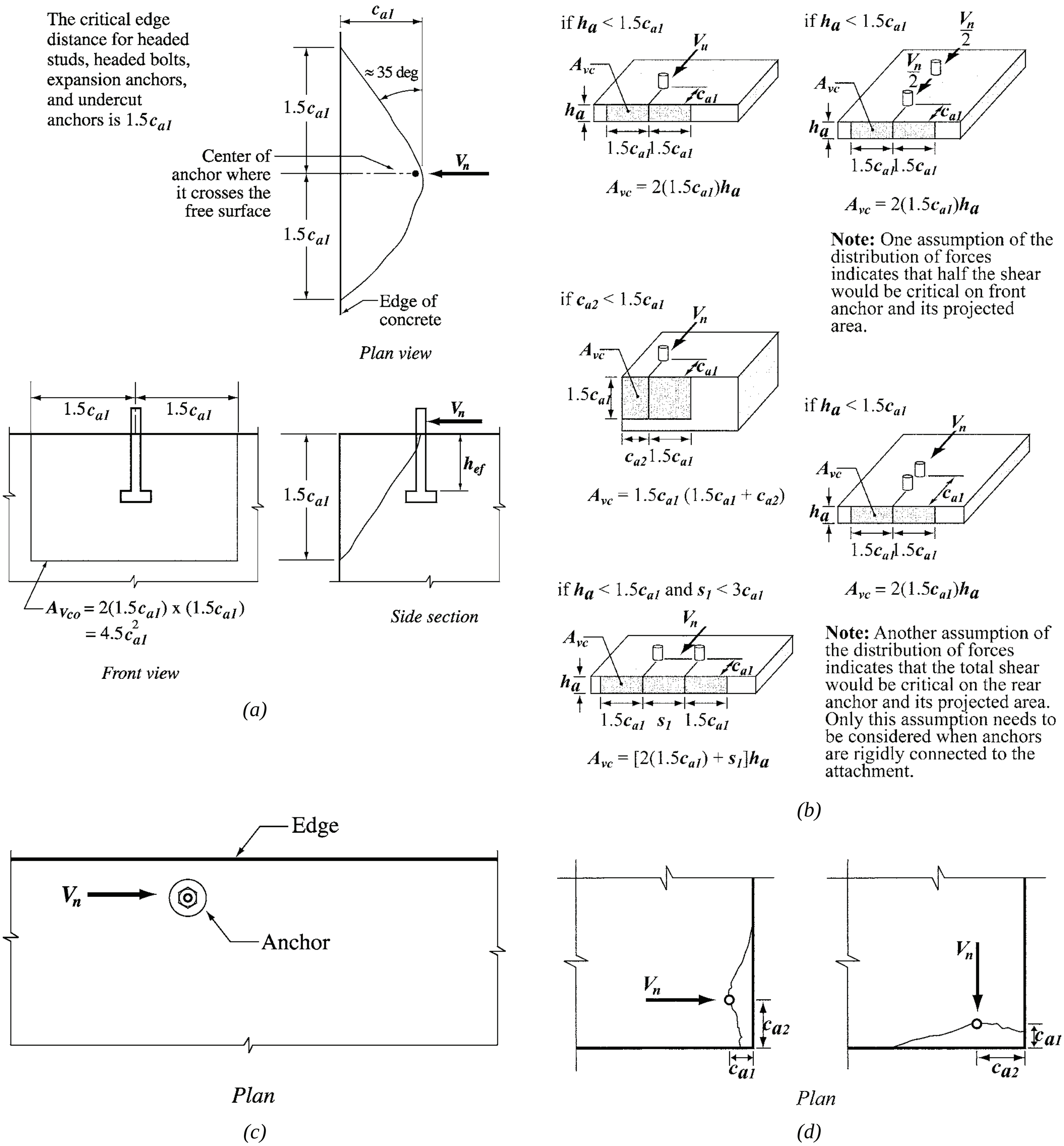


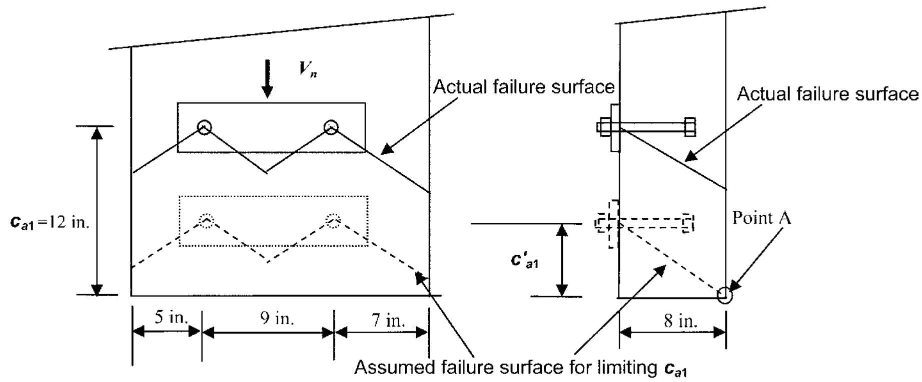
Fig. RD.6.2.1—(a) Calculation of $\mathbf{A_{Vco}}$*; (b) Projected area for single anchors and groups of anchors and calculation of* $\mathbf{A_{Vc}}$*; (c) shear force parallel to an edge; and (d) shear force near a corner.*

For the case of anchors near a corner subjected to a shear load with components normal to each edge, a satisfactory solution is to check the connection independently for each component of the shear load. Other specialized cases, such as the shear resistance of anchor groups where all anchors do not have the same edge distance, are treated in Reference D.14.

The detailed provisions of D.6.2.1(a) apply to the case of shear load directed toward an edge. When the shear load is directed away from the edge, the strength will usually be governed by D.6.1 or D.6.3.

The case of shear load parallel to an edge is shown in Fig. RD.6.2.1(c). A special case can arise with shear load parallel to the edge near a corner (refer to Fig. RD.6.2.1(d)). The provisions for shear in the direction of the load should be checked in addition to the parallel-to-edge provisions.

RD.6.2.2 Similar to the concrete breakout tensile strength, the concrete breakout shear capacity does not



The actual c_{a1} = 12 in. but two orthogonal edges c_{a2} and h_a are ≤ 1.5 c_{a1} therefore the limiting value of c_{a1} (shown as c'_{a1} in the figure) is the larger of $c_{a2,max}$**/1.5**, h_a**/1.5** and one-third of the maximum spacing for an anchor group:
c'_{a1} = max (7/1.5, 8/1.5, 9/3) = 5.33 in.
Therefore, use c'_{a1} = 5.33 in. in Eq. (D-21) to (D-28) including the calculation of A_{Vc}:
A_{Vc} = (5 + 9 + 7)(1.5(5.33)) = 168 in.^{2}
Point A shows the intersection of the assumed failure surface for limiting c_{a1} with the concrete surface.

Fig. RD.6.2.4—Shear when anchors are influenced by three or more degrees.

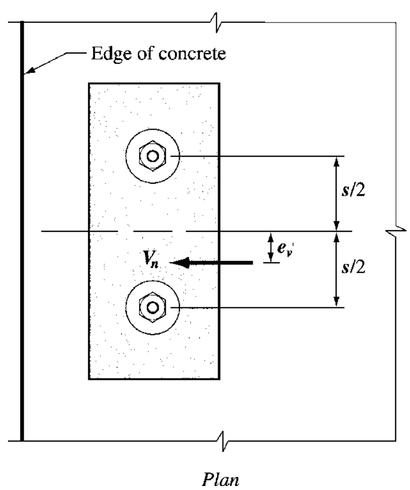


Fig. RD.6.2.5—Definition of dimensions e'_V .

increase with the failure surface, which is proportional to $(c_{a1})^2$. Instead, the capacity increases proportionally to $(c_{a1})^{1.5}$ due to size effect. The strength is also influenced by the anchor stiffness and the anchor diameter.[D.12-D.14,D.17]

The constant, 7, in the shear strength equation was determined from test data reported in Reference D.12 at the 5% fractile adjusted for cracking.

RD.6.2.3 For the special case of cast-in headed bolts continuously welded to an attachment, test data[D.28,D.29] show that somewhat higher shear strength exists, possibly due to the stiff welding connection clamping the bolt more effectively than an attachment with an anchor gap. Because of this, the basic shear value for such anchors is increased. Limits are imposed to ensure sufficient rigidity. The design of supplementary reinforcement is discussed in References D.11, D.14, and D.15.

RD.6.2.4 For anchors influenced by three or more edges where any edge distance is less than $1.5c_{a1}$, the shear breakout strength computed by the basic CCD method, which is the basis for Eq. (D-21) through (D-28), gives safe but misleading results. These special cases were studied for the κ method[D.17] and the problem was pointed out by Lutz.[D.25] Similar to the approach used for tensile breakouts in D.5.2.3, a correct evaluation of the strength is determined if the value of c_{a1} to be used in Eq. (D-21) to (D-28) is limited to the maximum of $c_{a2}/1.5$ in each direction, $h_a/1.5$, and 1/3 of the maximum spacing between anchors for anchor groups. The limit on c_{a1} of at least 1/3 of the maximum spacing between anchors for anchor groups prevents the designer using a calculated strength based on individual breakout prisms for a group configuration. This approach is illustrated in Fig. RD.6.2.4. In this example, the proposed limit on the value of c_{a1} to be used in the computations where c_{a1} = the largest of $c_{a2}/1.5$ in each direction, $h_a/1.5$, results in c_{a1} = 5.33 in. For this example, this would be the proper value to be used for c_{a1} in computing the resistance even if the actual edge distance that the shear is directed toward is larger. The requirement of D.6.2.4 may be visualized by moving the actual concrete breakout surface originating at the actual c_{a1} toward the surface of the concrete perpendicular to the applied shear force. The limit on c_{a1} for use in Eq. (D-21) to (D-28) occurs when either the outer boundaries on the failure surface first intersect a free edge or when the intersection of the breakout surface originating between anchors within the group first intersects the surface of the concrete. For the example shown in Fig. RD.6.2.4, Point A shows the controlling intersection.

RD.6.2.5 This section provides a modification factor for an eccentric shear force toward an edge on a group of anchors. If the shear force originates above the plane of the concrete surface, the shear should first be resolved as a shear in the plane of the concrete surface, with a moment that may or may not also cause tension in the anchors, depending on the normal force. Figure RD.6.2.5 defines the term e'_V for calculating the $\psi_{ec,V}$ modification factor that accounts for the fact that more shear is applied on one anchor than the other, tending to split the concrete near an edge.

RD.6.2.7 Torque-controlled and displacement-controlled expansion anchors are permitted in cracked concrete under pure shear.

RD.6.3 *Concrete pryout strength of anchor in shear*—Reference D.12 indicates that the pryout shear resistance can be approximated as one to two times the anchor tensile resistance with the lower value appropriate for h_{ef} less than 2.5 in.

RD.7—Interaction of tensile and shear forces

The shear-tension interaction expression has traditionally been expressed as

$$\left(\frac{N_{ua}}{N_n}\right)^{\varsigma} + \left(\frac{V_{ua}}{V_n}\right)^{\varsigma} \leq 1.0$$

where ς varies from 1 to 2. The current trilinear recommendation is a simplification of the expression where $\varsigma = 5/3$ (Fig. RD.7). The limits were chosen to eliminate the requirement for computation of interaction effects where very small values of the second force are present. Any other interaction expression that is verified by test data, however, can be used to satisfy D.4.3.

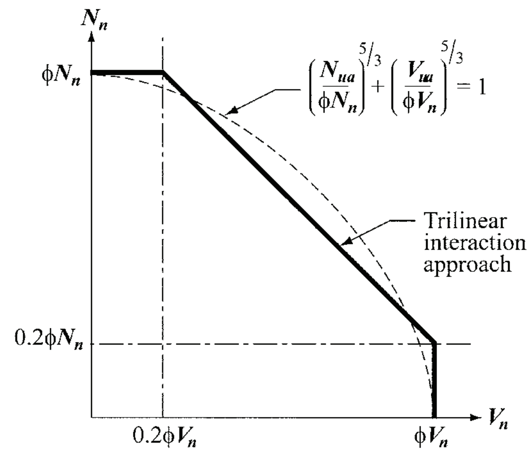


Fig. RD.7—Shear and tensile load interaction equation.

RD.8—Required edge distances, spacings, and thicknesses to preclude splitting failure

The minimum spacings, edge distances, and thicknesses are dependent on the anchor characteristics. Installation forces and torques in post-installed anchors can cause splitting of the surrounding concrete. Such splitting also can be produced in subsequent torquing during connection of attachments to anchors including cast-in anchors. The primary source for minimum spacings, edge distances, and thicknesses of post-installed anchors should be the product-specific tests. In some cases, however, specific products are not known in the design stage. Approximate values are provided for use in design.

RD.8.2 Because the edge cover over a deep embedment close to the edge can have a significant effect on the side-face blowout strength of D.5.4, in addition to the normal concrete cover requirements, the designer may wish to use larger cover to increase the side-face blowout strength.

RD.8.3 Drilling holes for post-installed anchors can cause microcracking. The requirement for a minimum edge distance twice the maximum aggregate size is to minimize the effects of such microcracking.

RD.8.4 Intentionally left blank.

RD.8.5 This minimum thickness requirement is not applicable to through-bolts because they are outside the scope of Appendix D. In addition, splitting failures are caused by the load transfer between the bolt and the concrete. Because through-bolts transfer their load differently than cast-in or expansion and undercut anchors, they would not be subject to the same member thickness requirements. Post-installed anchors should not be embedded deeper than 2/3 of the member thickness.

RD.8.6 The critical edge distance c_{ac} is determined by the corner test in ACI 355.2. Research has indicated that the corner-test requirements are not met with $c_{a,min} = 1.5h_{ef}$ for many expansion anchors and some undercut anchors because installation of these types of anchors introduces splitting tensile stresses in the concrete that are increased during load application, potentially resulting in a premature splitting failure. To permit the design of these types of anchors when product-specific information is not available, conservative default values for c_{ac} are provided.

RD.9—Installation of anchors

Many anchor performance characteristics depend on proper installation of the anchor. Anchor strength and deformations can be assessed by acceptance testing under ACI 355.2. These tests are performed out assuming that the manufacturer's installation directions will be followed. Certain types of anchors can be sensitive to variations in hole diameter, cleaning conditions, orientation of the axis, magnitude of the installation torque, crack width, and other variables. Some of this sensitivity is indirectly reflected in the assigned ϕ-values for the different anchor categories, which depend in part on the results of the installation safety tests. Gross deviations from the ACI 355.2 acceptance testing results could occur if anchor components are incorrectly exchanged, or if anchor installation criteria and procedures vary from those recommended. Project specifications should require that anchors be installed according to the manufacturer's recommendations.

RD.10—Structural plates, shapes, and specialty inserts

Design strengths for structural plates, shapes, and specialty inserts are based on the ϕ-values in the AISC-LRFD *Steel Manual*. The ϕ-value of 0.90 for tension, compression, and bending was established based on SEI/ASCE 7 load combinations. The value of 0.55 for shear is a product of ϕ = 0.90 and $F_v = 0.6F_y$. For these elements, the same ϕ-factors are used for the load combinations in Section 9.2 and Appendix C of the Code.

RD.11—Shear strength of embedded plates and shear lugs

RD.11.1 *Shear lugs*—The Code requirements for the design of shear lugs are based on testing reported in

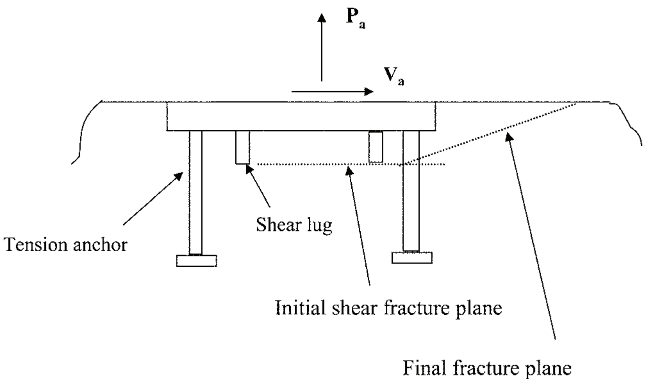


Fig. RD.11.1—Fracture planes for embedments with shear lugs.

Reference D.21. This testing confirmed that shear lugs are effective with axial compression and tension loads on the embedment, and that the strength is increased due to the confinement afforded by the tension anchors in combination with external loads. The shear strength of the embedment is the sum of the bearing strength and the strength due to confinement.

The tests also revealed two distinct response modes:

(1) A bearing mode characterized by shear resistance from direct bearing of shear lugs and inset faceplate edges on concrete or grout augmented by shear resistance from confinement effects associated with tension anchors and external concurrent axial loads; and

(2) A shear-friction mode such as defined in 11.7 of the Code.

The embedments first respond in the bearing mode and then progress into the shear-friction mode subsequent to formation of final fracture planes in the concrete in front of the shear lugs or base plate edge.

The bearing strength of single shear lugs bearing on concrete is defined in D.4.6. For multiple lugs, the shear strength should not exceed the shear strength between shear lugs as defined by a shear plane between the shear lugs, as shown in Fig. RD.11.1 and a shear stress limited to $\mathbf{10}\phi\sqrt{f_c'}$, with ϕ equal to 0.85.

The anchorage shear strength due to confinement can be taken as $\phi \boldsymbol{K_c}(\boldsymbol{N_y} - \boldsymbol{P_a})$, with ϕ equal to 0.85, where $\boldsymbol{N_y}$ is the yield strength of the tension anchors equal to $\boldsymbol{nA_{se} f_y}$, and $\boldsymbol{P_a}$ is the factored external axial load on the anchorage. ($\boldsymbol{P_a}$ is positive for tension and negative for compression). This approach considers the effect of the tension anchors and external loads acting across the initial shear fracture planes (see Fig. RD.11.1). When $\boldsymbol{P_a}$ is negative, the provisions of Section 9.2.3 regarding use of load factors of 0.9 or zero must also be considered. The confinement coefficient $\boldsymbol{K_c}$, given in Reference D.21, is as follows:

$\boldsymbol{K_c}$ **= 1.6** for inset base plates without shear lugs, or for anchorage with multiple shear lugs of height $\boldsymbol{h}$ and spacing $\boldsymbol{s}$ (clear distance face-to-face between shear lugs) less than or equal to $\mathbf{0.13}\boldsymbol{h}\sqrt{f_c'}$; and

$\boldsymbol{K_c}$ **= 1.8** for anchorage with a single shear lug located a distance $\boldsymbol{h}$ or greater from the front edge of the base plate, or with multiple shear lugs and a shear lug spacing $\boldsymbol{s}$ greater than $\mathbf{0.13}\boldsymbol{h}\sqrt{f_c'}$.

These values of confinement factor $\boldsymbol{K_c}$ are based on the analysis of test data. The different $\boldsymbol{K_c}$ values for plates with and without shear lugs primarily reflect the difference in initial shear-fracture location with respect to the tension anchors. The tests also show that the shear strength due to confinement is directly additive to the shear strength determined by bearing or by shear stress. The tension anchor steel area required to resist applied moments can also be used for determining $\boldsymbol{N_{sa}}$, providing that the compressive reaction from the applied moment acts across the potential shear plane in front of the shear lug.

For inset base plates, the area of the base plate edge in contact with the concrete can be used as an additional shear-lug-bearing area providing displacement compatibility with shear lugs can be demonstrated. This requirement can be satisfied by designing the shear lug to remain elastic under factored loads with a displacement (shear plus flexure) less than 0.01 in.

For cases such as in grouted installations where the bottom of the base plate is above the surface of the concrete, the shear-lug-bearing area should be limited to the contact area below the plane defined by the concrete surface. This accounts for the potential extension of the initial shear fracture plane (formed by the shear lugs) beyond the perimeter of the base plate, that could diminish the effective bearing area.

Multiple shear lugs should be proportioned by considering relative shear stiffness. When multiple shear lugs are used near an edge, the effective stress area for the concrete design shear strength should be evaluated for the embedment shear at each shear lug.

RD.11.3 *Shear strength of embedments with embedded base plates*—The coefficient of 1.4 for embedments with shear lugs reflects concrete-to-concrete friction afforded by confinement of concrete between the shear lug(s) and the base plate (post-bearing mode behavior). This value corresponds to the friction coefficient of 1.4 recommended in 11.7 of the Code for concrete-to-concrete friction, and is confirmed by tests discussed in Reference D.21.

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