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

Reported by ACI Committee 349
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ACI 349.2R-07 supersedes ACI 349.2R-97 and was adopted and published November 2007.
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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 concrete breakout strength. 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

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_{brg}$</td>
<td>bearing area of the head of stud or anchor bolt, in.$^2$</td>
</tr>
<tr>
<td>$A_{brg,pl}$</td>
<td>the effective bearing area of a steel base plate, in.$^2$</td>
</tr>
<tr>
<td>$A_D$</td>
<td>gross cross-sectional area of anchor, in.$^2$</td>
</tr>
<tr>
<td>$A_H$</td>
<td>gross cross-sectional area of anchor head, in.$^2$</td>
</tr>
<tr>
<td>$A_{Nc}$</td>
<td>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</td>
</tr>
<tr>
<td>$A_{Nco}$</td>
<td>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</td>
</tr>
<tr>
<td>$A_{se}$</td>
<td>effective cross-sectional area of anchor, in.$^2$</td>
</tr>
<tr>
<td>$A_{se,t}$</td>
<td>effective cross-sectional area of anchor (required to resist tension loads), in.$^2$</td>
</tr>
<tr>
<td>$A_{se,v}$</td>
<td>effective cross-sectional area of anchor (required to resist shear loads), in.$^2$</td>
</tr>
<tr>
<td>$A_{Vc}$</td>
<td>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</td>
</tr>
<tr>
<td>$A_{Vco}$</td>
<td>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</td>
</tr>
<tr>
<td>$a$</td>
<td>moment arm from row of anchors to mid-thickness of adjacent steel tube wall, in.</td>
</tr>
<tr>
<td>$b$</td>
<td>width of steel base plate, in.</td>
</tr>
<tr>
<td>$b_{eff}$</td>
<td>effective width of steel base plate, in.</td>
</tr>
<tr>
<td>$b_f$</td>
<td>flange width of supported steel member, in.</td>
</tr>
<tr>
<td>$C$</td>
<td>the compressive resultant force between the embedment and the concrete resulting from factored moment and factored axial load applied to the embedment, lb</td>
</tr>
<tr>
<td>$C_m$</td>
<td>the resultant compressive force in concrete due to factored moment acting on a steel base plate, kips</td>
</tr>
<tr>
<td>$c_{a1}$</td>
<td>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.</td>
</tr>
<tr>
<td>$c_{a2}$</td>
<td>distance from center of an anchor shaft to the edge of concrete in the direction orthogonal to $c_{a1}$, in.</td>
</tr>
<tr>
<td>$c_{ac}$</td>
<td>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</td>
</tr>
<tr>
<td>$c_{a,max}$</td>
<td>maximum distance from center of an anchor shaft to the edge of concrete, in.</td>
</tr>
<tr>
<td>$c_{a,min}$</td>
<td>minimum distance from center of an anchor shaft to the edge of concrete, in.</td>
</tr>
<tr>
<td>$d$</td>
<td>moment arm from resultant compression force on base plate to center of tension force in anchors, in.</td>
</tr>
<tr>
<td>$d_c$</td>
<td>distance from resultant compression force to adjacent edge of supported steel member, in.</td>
</tr>
<tr>
<td>$d_e$</td>
<td>distance from edge of steel base plate to the resultant compression force, in.</td>
</tr>
<tr>
<td>$d_h$</td>
<td>nominal diameter of anchor head, in.</td>
</tr>
<tr>
<td>$d_o$</td>
<td>outside diameter of anchor or shaft diameter of headed stud or headed bolt, in.</td>
</tr>
<tr>
<td>$d_s$</td>
<td>depth of supported steel member, in.</td>
</tr>
</tbody>
</table>

*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.

\( t_e \) = load bearing length of anchor for shear, not to exceed \( 8d_t \), in., see D.6.2.2

\( M_{fl} \) = nominal flexural strength at section, kip-in.

\( M_p \) = plastic moment of steel plate, kip-in.

\( M_{fp} \) = 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

\( N_t \) = 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 = factored diameter of steel washer, in.

\( P_a \) = 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

\( V_{ua} \) = factored shear force at section, kips

\( V_{uaa} \) = factored shear force applied to a single anchor or group of anchors, lb

\( w \) = moment arm from corner anchor to mid-thickness of adjacent steel tube wall, in.

\( Z \) = plastic section modulus of steel base plate, in.\(^3\)

\( \phi \) = 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
ψ_{ed,N} = factor used to modify tensile strength of anchors based on proximity to edges of concrete member, see D.5.2.5

ψ_{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 ψ-factors) is made. The Engineer should reconcile the differences between these two codes when designing the base plate.

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

Strength reduction factor φ for frictional resistance is not explicitly defined in the Code. As frictional resistance is not related to a steel mode of failure, the examples have used the φ-factor from D.4.4c or D.4.5c (depending on whether 9.2 or C.2 of the Code is used, respectively).
PART A—Examples: Ductile single embedded element in semi-infinite concrete

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

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

Given:
Concrete edges
\[ c_{a1} = c_{a2} = 12 \text{ in.} \]
\[ h_a = 18 \text{ in.} \]
Concrete
\[ f'_c = 4000 \text{ psi} \]
Stud material (A29/A108)\(^*\)
\[ f_{ya} = 51 \text{ ksi} \]
\[ f_{uta} = 65 \text{ ksi} \]
Plate
3 x 3 x 3/8 in. thick
\[ F_y = 36 \text{ ksi} \]
Loads
\[ N_{ua} = 8 \text{ kips} \]

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

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

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

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.4.1.1</td>
<td>Equate the external factored load to the internal design strength and solve for the required steel area of the stud.</td>
<td>( \Phi N_a \geq N_{ua} ) (D-1)</td>
</tr>
<tr>
<td>D.5.1.2</td>
<td>Assume embedment will be designed as ductile in accordance with D.3.6.1 (in Step 2). Therefore, ( \Phi = 0.80 ) for tension.</td>
<td>( N_{sa} = \Phi n A_{se} f_{uta} )</td>
</tr>
<tr>
<td>D.5.1.2</td>
<td>Calculate nominal steel strength of selected stud. (Appendix A, Table 6)</td>
<td>( N_{sa} = 0.80 \times 1.0 \times A_{se} \times 65 \text{ kips} )</td>
</tr>
<tr>
<td>D.5.1.2</td>
<td>Material properties are given. See footnote on previous page.</td>
<td>( A_{se, req} = 0.154 \text{ in.}^2 ) required</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Anchor diameter, in.</th>
<th>Effective area ( A_{se} ), in.</th>
<th>controls</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8</td>
<td>0.110</td>
<td></td>
</tr>
<tr>
<td>1/2</td>
<td>0.196</td>
<td></td>
</tr>
</tbody>
</table>

For use one 1/2 in. diameter stud
\( A_{se} = 0.196 \text{ in.}^2 > 0.154 \text{ in.}^2 \)

\( N_{sa} = n A_{se} f_{uta} \) (D-3)

\( f_{uta} = 65 \leq 1.9 f_{ya} = 1.9 \times 51 = 96.9 \text{ ksi} \leq 125,000 \text{ psi} \) OK

### STEP 2: Determine required embedment length for the stud to prevent concrete breakout failure in tension.

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.5.2</td>
<td>Calculate the required embedment depth for the stud to prevent concrete breakout failure.</td>
<td>From Step 1:</td>
</tr>
<tr>
<td>D.3.6.1</td>
<td>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 ( \Phi )-factor for steel used previously.</td>
<td>( 0.85 N_{cb, req} = N_{sa} )</td>
</tr>
<tr>
<td>D.5.2.1</td>
<td>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_{cb}, N_p, \text{ etc.} ). Note that 0.85 is not a ( \Phi )-factor.</td>
<td>( N_{cb, req} = N_{sa}/0.85 )</td>
</tr>
<tr>
<td>D.5.2.1</td>
<td>Concrete breakout strength for a single stud:</td>
<td>( = 12.74/0.85 = 14.99 \text{ kips} )</td>
</tr>
</tbody>
</table>

\( N_{cb} = (A_{Nc}/A_{Nco}) \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \) lb (D-4)

\( A_{Nc}/A_{Nco} = 1.0 \)

\( A_{Nc} = 9 h_{ef}^2 \)

\( h_{ef} = 4.69 \text{ in.} > 4.60 \text{ in.} \) OK

From Table 6, Appendix A for additional stud properties.

<table>
<thead>
<tr>
<th>( k_c )</th>
<th>24 for cast-in headed stud</th>
<th>24 ( h_{ef}^{1.5} ) lb</th>
</tr>
</thead>
<tbody>
<tr>
<td>( k_c )</td>
<td>24 ( h_{ef}^{1.5} ) kips</td>
<td>1518 ( h_{ef}^{1.5} ) kips</td>
</tr>
</tbody>
</table>

Determine required embedment length \( h_{ef, req} \).

The embedment length is calculated as the total stud length, minus head thickness, plus plate thickness, minus burnoff. Standard length and head dimensions are given by the manufacturer. Typical values are given in Table 6, Appendix A.

Calculate \( N_{cb} \) using \( h_{ef, provided} \).

\( N_{cb} = 1.52 \times 4.69^{1.5} \)

\( = 15.44 \text{ kips} \geq N_{cb, req} = 14.99 \text{ kips} \) OK
<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>STEP 3: Check pullout strength of stud.</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D.5.3</td>
<td>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.</td>
<td></td>
</tr>
<tr>
<td>D.5.3.1</td>
<td>Concrete is cracked per problem statement.</td>
<td>[ N_{pn} = \psi_{c,p} N_P ] (D-14)</td>
</tr>
<tr>
<td>D.5.3.4</td>
<td>Calculate pullout strength of anchor.</td>
<td>[ N_p = 8A_{brg} f'<em>{c,c} ] [ = 8A</em>{brg} \times 4 ] [ = 32A_{brg} \text{ kips} ] (D-15)</td>
</tr>
<tr>
<td>D.5.3.5</td>
<td>[ \psi_{c,p} = 1.0 ] for cracked concrete.</td>
<td>[ \psi_{c,p} = 1.0 ]</td>
</tr>
<tr>
<td>D.3.6.1</td>
<td>Design embedment as ductile, in accordance with D.3.6.1: [ 0.85N_{pn} \geq N_{sa} ]</td>
<td>[ 0.85N_{pn} = 0.85 \times 18.88 ] [ = 16.05 \text{ kips} &gt; N_{sa} = 12.74 \text{ kips} ]</td>
</tr>
<tr>
<td></td>
<td><strong>Therefore ductile</strong></td>
<td><strong>OK</strong></td>
</tr>
</tbody>
</table>

| **STEP 4: Check concrete side-face blowout.** | | |
| D.5.4 | Because this stud is far away from the an edge, side-face blowout \[ N_{sb} \] will not be a factor, and will not be checked in this example. | N/A |

| **STEP 5: Summary.** | | |
| **Given** | Applied load | \[ N_{ua} = 8 \text{ kips} \] |
| **Step 1** D.4.5.a | Design steel tensile strength | \[ \phi N_{sa} = 0.8 \times 12.74 = 10.19 \text{ kips} \] |
| **Step 2** D.4.5.c | Design concrete breakout strength | \[ \phi N_{cb} = 0.75 \times 15.44 = 11.58 \text{ kips} \] |
| **Step 3** D.4.5.c | Design concrete pullout strength | \[ \phi N_{pn} = 0.75 \times 18.88 = 14.16 \text{ kips} \] |
| **Step 4** D.4.5.c | Design concrete side-face blowout strength | \[ \phi N_{sb} = N/A \] |
| **D.4.1.2** | Design strength of stud in tension | \[ \phi N_n = \min(\phi N_{sa}, \phi N_{cb}, \phi N_{pn}) \] \[ = \min(10.19, 11.58, 14.16) \] \[ = 10.19 \text{ kips} > N_{sa} = 8 \text{ kips} \] **OK** |
| **D.3.6.1** | Ductility | \[ \min(0.85N_{cb}, 0.85N_{pn}) > N_{sa} \] \[ \min(0.85 \times 15.44, 0.85 \times 18.88) \] \[ = 13.12 > 12.74 \text{ kips} \] **OK** |

| **STEP 6: Check plate thickness.** | | |
| AISC | Because the load is applied directly over the stud, the only requirement on plate thickness is that it satisfies the minimum thickness required for stud welding. | Stud welding of 1/2 in. diameter studs is acceptable on 3/8 in. thick plate per D.6.2.3. **OK** |

*The length of stud selected is not a standard length. User should consult with manufacturer(s) for readily available lengths. In the above example, the effective embedment length \( h_e \) is taken to the face of the concrete. If the plate was larger than the projected surface area, then the embedment length would exclude the thickness of the embedded plate.
Example A2—Single stud, shear only

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

Given:

- Edges
  - \( c_{a1} = 10 \text{ in.} \)
  - \( c_{a2} = 18 \text{ in.} \)
  - \( h_a = 18 \text{ in.} \)

- Concrete
  - \( f'_c = 4000 \text{ psi} \)

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

- Plate
  - Assume 3 x 3 x 3/8 in. thick
  - \( F_y = 36 \text{ ksi} \)

- Loads
  - \( V_{ua} = 6 \text{ kips} \)

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

Assumptions:

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

---

*Stud material is A29/A108, material properties per AWS D1.1, 2006, Table 7.1, Type B stud. Yield strength = 51 ksi; tensile strength = 65 ksi. It has elongation of 20% and reduction in area of 50%; meets the definition of a ductile steel element given in D.1, and meets the tensile strength requirements of D.5.1.2 and D.6.1.2: \( f_{uta} \leq 1.9f_{ya} \) (65 \( \leq 1.9 \times 51 = 96.9 < 125 \text{ ksi} \)).
### CODE SECTION | DESIGN PROCEDURE | CALCULATION
---|---|---
D.4.1 | Equate the external factored load to the internal design strength and solve for the required steel area of the stud. | \[ \phi V_n \geq V_{ut} \]
D.6.1.2 | Assume embedment will be designed as ductile in accordance with D.3.6.1 (in Step 2). Therefore, \( \phi = 0.75 \) for shear loads. | \( A_{se,req} = \frac{V_{ut}}{(\phi f_{uta})} \)
D.6.1 | Design required area of stud using the steel strength provision (Appendix A, Table 6) | \( A_{se,req} = \frac{6}{(0.75 \times 1 \times 65)} \)
 | | = 0.123 in.\(^2\) required
D.6.1.2 | Calculate the nominal shear capacity of the selected stud. | Use one 1/2 in. diameter stud
 | | | \( A_{se} = 0.196 \text{ in.}^2 > 0.123 \text{ in.}^2 \) 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: | \[ 0.85 V_{cb} \geq A_{se} f_{uta} \]
D.6.2.1 | Calculate concrete breakout strength \( V_{cb} \) in shear for a single stud. | Min \( V_{cb} = A_{se} f_{uta} / 0.85 \)
 | | | = 0.196 \times 65 / 0.85 \)
 | | | = 14.99 kips
D.6.2.2 | Calculate projected area for a single stud. See figures above for illustration of \( A_{Vc} \) and \( A_{Vco} \). Because edges are far enough away, \( A_{Vc} \) and \( A_{Vco} \) are equal. | \( V_b = 8(L/d_o)^{0.2} \sqrt{d_o} f_{c1}^{0.5} c_{al}^{1.5} \)
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. | \( V_e \leq 8d_o \)
 | | | Assume \( \ell_e = 2.5 \) in.
 | | | \( (L/d_o) = 2.5/0.5 = 5.0 \)
 | | | \( V_b = 8 \times 5^{0.2} \times 0.5^{0.5} \times 4000^{0.5} \times c_{al}^{1.5} \)
 | | | = 494 \times c_{al}^{1.5} \)
 | | | \( = 0.494 c_{al}^{1.5} \) kips
D.6.2.6 | See definition in D.6.2.2 for limits on \( \ell_e \). | \( \psi_{ed,V} = 1.0 \)
 | | | \( \psi_{c,V} = 1.0 \)
D.6.2.7 | Modification factors for shear for: Edge effects \( \psi_{ed,V} \) | \( V_{cb,req} = 14.99 \text{ kips} \)
 | | | Concrete cracking \( \psi_{c,V} \) | \( = 1.0 \times 1.0 \times 0.494 c_{al}^{1.5} \)
 | | | | \( = 0.494 c_{al}^{1.5} \) kips

**Concrete is cracked per problem statement. No additional supplementary steel is provided.**

**Calculate \( V_{cb} \) using \( c_{al} = 10 \) in. provided.**
### 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.

\[ V_{cp} = 12.74 \text{ kips} \]  
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:

\[ 0.85V_{cp} \geq V_{sa} \]

\[ V_{cp,req} = 12.74/0.85 = 14.99 \text{ kips} \]  
\( V_{cp} = k_{cp}N_{cb} \)  
\( k_{cp} = 2.0 \)

\( N_{cb,req} = 14.99/2.0 = 7.50 \text{ kips (required)} \)

Calculation of required embedment depth of the anchor to prevent pryout.

**D.6.3** Design required embedment depth, from the concrete pryout strength requirement. Assume \( h_{ef} > 2.5 \text{ in.} \) Therefore, \( k_{cp} = 2.0 \).

**D.5.2** \( N_{cb} \) 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.

**D.5.2.2** Basic concrete breakout strength for a single anchor in tension: \( k_{c} = 24 \) for cast-in headed studs. Assume \( h_{ef} < 11 \text{ in.} \)

\[ N_{p} = 8A_{brg}f'_{c} \]

\[ = 8 \times 4 \times A_{brg} \]

\[ = 32A_{brg} \text{ kips} \]

\[ \psi_{c,p} = 1.0 \]

\[ A_{brg} = 0.589 \text{ in.}^{2} \]

\[ N_{pn} = 1 \times 32 \times 0.589 = 18.8 \text{ kips} \]

\( N_{pn} \) for this problem is calculated in the pryout section shown in Step 3.

**STEP 4: Check pullout strength of stud to check head of the stud.**

**D.5.3** Checking of stud head is required to develop the concrete breakout strength \( N_{cb} \) used to check concrete pryout.

**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.

**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.)

\[ \psi_{c,p} = 1.0 \]

\[ A_{brg} = 0.589 \text{ in.}^{2} \]

\[ N_{pn} = 1 \times 32 \times 0.589 = 18.8 \text{ kips} \]

\( N_{pn} = \psi_{c,p}N_{p} \)  
\( N_{p} = 8A_{brg}f'_{c} \)

\( = 8 \times 4 \times A_{brg} \)

\( = 32A_{brg} \) kips

Use 1/2 in. diameter x 3-1/2 in. long stud.

**D.3.6.1** Design embedment as ductile, in accordance with D.3.6.1:

\[ 0.85N_{pn} > N_{sa} \]

\[ 0.85 \times 18.8 = 16 \text{ kips} > N_{sa} = 12.74 \text{ kips} \]

Therefore ductile

\[ N_{pn} = 0.85 \times 18.8 \]

\[ = 16 \text{ kips} > N_{sa} = 12.74 \text{ kips} \]

Use 1/2 in. diameter x 3-1/2 in. long stud.

Specified 1/2 in. diameter anchor head dimension
### Code Section | Design Procedure | Calculation
---|---|---
**STEP 5: Summary of design strength.**

<table>
<thead>
<tr>
<th>Given</th>
<th>Applied load</th>
<th>$V_{sa} = 6 \text{ kips}$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Step 1</strong>&lt;br&gt;D.4.5.a</td>
<td>Design steel shear strength</td>
<td>$\phi V_{sa} = 0.75 \times 12.74 = 9.56 \text{ kips}$ OK</td>
</tr>
<tr>
<td><strong>Step 2</strong>&lt;br&gt;D.4.5.c</td>
<td>Design concrete breakout strength</td>
<td>$\phi V_{cb} = 0.75 \times 15.62 = 11.72 \text{ kips}$ OK</td>
</tr>
<tr>
<td><strong>Step 3</strong>&lt;br&gt;D.4.5.c</td>
<td>Design concrete pryout strength</td>
<td>$\phi V_{cp} = 0.75 \times 16.27 = 12.20 \text{ kips}$</td>
</tr>
</tbody>
</table>
| D.4.1.2 | Design strength of stud in shear | $\phi V_n = \min(\phi V_{sa}, \phi V_{cb}, \phi V_{cp})$
| | | $= \min(9.56, 11.72, 12.20)$
| | | $= 9.56 > V_{sa} = 6 \text{ kips}$ OK |
| D.3.6.1 | Ductility | $\min(0.85 V_{cb}, 0.85 V_{cp}) \geq V_{sa}$
| | | $\min(0.85 \times 15.62, 0.85 \times 16.27)$
| | | $= 13.28 > 12.74 \text{ kips}$ 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 $d_p/t < 2.7$. | $t = \max(3/8 \text{ in., } 0.5/2 = 0.25 \text{ in.})$
| | | $= 3/8 \text{ in.}$
| | | $\min t > 0.5/2.7 = 0.19 \text{ in.} < 3/8 \text{ in.}$
| | | $3/8 \text{ in. thick plate is OK}$ |

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.
- $\phi$-factors are based on Condition B in D.4.5 of the Code (no supplementary reinforcement).

---

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

#### STEP 1: Determine required steel area of the stud.

D.4.1.1 Equate the external factored load to the internal design strength and solve for the required steel area of the stud.

\[ \phi N_n \geq N_{ua} \]

\[ N_u = N_{sa} = nA_{se,t}f_{uta} \]

D.5.1 Use the tension provisions of D.5.1 to determine the required steel area for tension load.

\[ A_{se,t} = \text{required steel area for tension load}. \]

\[ A_{se,t} = \frac{N_{sa}}{(\phi f_{uta})} = \frac{8}{(0.80 \times 1.0 \times 65)} = 0.154 \text{ in.}^2 \]

D.3.6.1 Assume embedment will be designed as ductile in accordance with D.3.6.1 (in Step 2). Therefore, \( \phi = 0.80 \) for tension and 0.75 for shear. Steel material is ductile. (See asterisked note on previous page.)

D.6.1 Use the shear provisions of D.6.1 to determine the required steel area for shear load.

\[ A_{se,v} = \text{required steel area for shear}. \]

\[ A_{se,v} = \frac{V_{sa}}{(\phi f_{uta})} = \frac{6}{(0.75 \times 1.0 \times 65)} = 0.123 \text{ in.}^2 \]

D.7.3 Total required area

\[ A_{se,req} = (A_{se,t} + A_{se,v})/1.2 \]

\[ A_{se,req} = (0.154 + 0.123)/1.2 = 0.231 \text{ in.}^2 \]

Anchor diameter, in. | Effective area \( A_{se,in} \) in.\(^2\) | Controls
--- | --- | ---
1/2 | 0.196 | 5/8 | 0.307

(Refer to Appendix A, Table 6)

Calculated the nominal steel strength \( N_{sa} \).

\[ N_{sa} = nA_{se,f_{uta}} = 1.0 \times 0.307 \times 65 = 19.96 \text{ kips} \]

Calculated the nominal steel strength \( V_{sa} \).

\[ V_{sa} = nA_{se,f_{uta}} = 1.0 \times 0.307 \times 65 = 19.96 \text{ kips} \]

#### STEP 2: Determine required embedment length for the stud 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 \( \phi \)-factor used previously. The steel capacity is based on the selected stud diameter.

D.3.6.1 The requirements for ductile design are given in D.3.6.1. For tension load, this requires that

\[ 0.85N_{cb,req} \geq N_{sa} \]

\[ N_{cb,req} = N_{ua}/0.85 = 19.96/0.85 = 23.48 \text{ kips} \]

D.5.2.1 Calculate concrete breakout strength for a single anchor.

\[ N_{cb} = (A_{Nc}/A_{Nco})\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_{cb} \text{ lb} \]

\[ A_{Nc}/A_{Nco} = 1.0 \]

Modification factors for:

- Edge effects \( \psi_{ed,N} = 1.0 \)
- Concrete cracking \( \psi_{c,N} = 1.0 \)
- \( \psi_{cp,N} = N/A \) for studs

Use one 5/8 in. diameter stud.

\[ A_{se} = 0.307 \text{ in.}^2 \geq 0.231 \text{ in.}^2 \]

OK

From Step 1:

\[ N_{sa} = 19.96 \text{ kips} \]

\[ N_{cb} = (A_{Nc}/A_{Nco})\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_{cb} \]
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<td><strong>STEP 2 (cont.)</strong></td>
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</table>
| D.5.2.2 | $k_e = 24$ for cast-in-place stud
Assume $h_{ef} < 11$ in. | $N_b = k_c \sqrt{f'_c} h_{ef}^{1.5}$ lb
$= 24 \sqrt{4000} h_{ef}^{1.5}$
$= 1.52h_{ef}^{1.5}$ kips
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. |
| | 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.** | |
| **D.5.3** | Stud head is required to develop the concrete breakout strength $N_{cb}$ |
| | Procedure is similar to that used in Example A1. Calculate the nominal pullout strength $N_{pn}$ of the stud in tension in accordance with D.5.3. |
| | Concrete is cracked per problem statement. Therefore, $\psi_{c,p} = 1.0$. Bearing area is based on manufacturer data. (Appendix A, Table 6). |
| | 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. |
| | Therefore ductile |
| | 5/8 in diameter x 6-3/4 in. long stud. OK |
| **D.5.3.4** | |
| **D.5.3.5** | |
| **D.3.6.1** | |
| **STEP 4: Check concrete side-face blowout.** | |
| **D.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 $c_{a,min} > 0.4h_{ef}$. |
| **RD.5.4** | In this example:
$h_{ef} = 6.63$ in.
$0.4h_{ef} = 0.4 \times 6.63 = 2.6$ 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. |
| | $N_{sb} = 160c_a A_{brg} \sqrt{f'_c}$ |
| | $A_{brg} = 0.92$ 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 |
| | 5/8 in. diameter x 6-3/4 in. long stud. OK |
CONCRETE CAPACITY DESIGN (CCD) METHOD—EMBEDMENT DESIGN EXAMPLES

**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:

\[
0.85V_{cb} > A_{se}f_{uta}
\]

\[
V_{cb,req} = \frac{A_{se}f_{uta}}{0.85} = \frac{0.307 \times 65}{0.85} = 23.48 \text{ kips}
\]

**D.6.2.1** Calculate concrete breakout strength \(V_{cb}\) in shear for a single stud.

\[
V_{cb} = (A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}V_b
\]

Calculate 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.

**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.

\[
V_b = 8(\ell_e/d_o)^{0.2}\sqrt{d_o/t_c} c_{al}^{1.5}
\]

Concrete is cracked per problem statement. No additional supplementary steel is provided.

**D.6.2.6** Modification factors for shear for:

- Edge effects \(\Psi_{ed,V}\)
- Concrete cracking \(\Psi_{c,V}\)

\[
\Psi_{ed,V} = 1.0 \quad \Psi_{c,V} = 1.0
\]

Step 1

Steel strength in shear \(V_{sa}\).

Calculate \(V_{cb}\) using \(c_{al} = 12\) in. provided.

**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:

\[
0.85V_{cp} > V_{sa}
\]

\[
V_{cp,req} = V_{sa}/0.85 = 19.96/0.85 = 23.5 \text{ kips}
\]

\[
V_{cp} = k_{cp}N_{cb}
\]

- \(k_{cp} = 2.0\)
- \(N_{cb} = 25.95 \text{ kips}\)

\[
V_{cp} = 2 \times 25.95 = 51.9 \text{ kips} >> 23.5 \text{ kips}
\]

5/8 in. diameter x 6-3/4 in. long stud OK
**CODE SECTION**

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<tr>
<td>D.4.1.2 TENSION</td>
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<tr>
<td>Applied load</td>
<td>$N_{sa} = 8 \text{ kips}$</td>
</tr>
<tr>
<td>Step 1</td>
<td></td>
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<tr>
<td>D.4.5.a Steel tensile strength</td>
<td>$\phi N_{sa} = 0.8 \times 19.96 = 15.97 \text{ kips}$</td>
</tr>
<tr>
<td>Step 2</td>
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</tr>
<tr>
<td>D.4.5.c Concrete breakout strength</td>
<td>$\phi N_{cb} = 0.75 \times 25.95 = 19.46 \text{ kips}$</td>
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<tr>
<td>Step 3</td>
<td></td>
</tr>
<tr>
<td>D.4.5.c Concrete pullout strength</td>
<td>$\phi N_{pn} = 0.75 \times 29.44 = 22.08 \text{ kips}$</td>
</tr>
<tr>
<td>Step 4</td>
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<tr>
<td>D.4.5.c Concrete side-face blowout strength</td>
<td>$\phi N_{sb} = 0.75 \times 116.5 = 87.40 \text{ kips}$</td>
</tr>
<tr>
<td>D.4.1.2 Design strength of stud in tension</td>
<td>$\phi N_n = \min(\phi N_{sa}, \phi N_{cb}, \phi N_{pn})$</td>
</tr>
<tr>
<td></td>
<td>$= \min(15.97, 19.46, 22.08, 87.40)$</td>
</tr>
<tr>
<td></td>
<td>$= 15.97 \text{ kips} &gt; N_{sa} = 8 \text{ kips}$ OK</td>
</tr>
<tr>
<td>SHEAR</td>
<td></td>
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<tr>
<td>Applied load</td>
<td>$V_{sa} = 6 \text{ kips}$</td>
</tr>
<tr>
<td>Step 1</td>
<td></td>
</tr>
<tr>
<td>D.4.5.a Steel strength</td>
<td>$\phi V_{sa} = 0.75 \times 19.96 = 14.97 \text{ kips}$</td>
</tr>
<tr>
<td>Step 2</td>
<td></td>
</tr>
<tr>
<td>D.4.5.c Concrete breakout strength</td>
<td>$\phi V_{cb} = 0.75 \times 25.19 = 18.89 \text{ kips}$</td>
</tr>
<tr>
<td>Step 3</td>
<td></td>
</tr>
<tr>
<td>D.4.5.c Concrete pryout strength</td>
<td>$\phi V_{cp} = 0.75 \times 51.9 = 38.9 \text{ kips}$</td>
</tr>
<tr>
<td>D.4.1.2 Design strength of stud in shear</td>
<td>$\phi V_n = \min(\phi V_{sa}, \phi V_{cb}, \phi V_{cp})$</td>
</tr>
<tr>
<td></td>
<td>$= \min(14.97, 18.89, 38.9)$</td>
</tr>
<tr>
<td></td>
<td>$= 14.97 \text{ kips} &gt; V_{sa} = 6 \text{ kips}$ OK</td>
</tr>
<tr>
<td>D.3.6.1 Ductility tension:</td>
<td>$\min(0.85N_{cb}, 0.85N_{pn}, 0.85N_{sb}) \geq N_{sa}$</td>
</tr>
<tr>
<td></td>
<td>$\min(0.85 \times 25.95, 0.85 \times 29.44, 0.85 \times 116.5)$</td>
</tr>
<tr>
<td></td>
<td>$= 22.06 &gt; N_{sa} = 19.96 \text{ kips}$ OK</td>
</tr>
<tr>
<td>Ductility shear:</td>
<td>$\min(0.85V_{cb}, 0.85V_{cp}) &gt; V_{sa}$</td>
</tr>
<tr>
<td></td>
<td>$\min(0.85 \times 25.19, 0.85 \times 51.9)$</td>
</tr>
<tr>
<td></td>
<td>$\min(21.41, 44.12)$</td>
</tr>
<tr>
<td></td>
<td>$= 21.41 &gt; V_{sa} = 19.96 \text{ kips}$ OK</td>
</tr>
</tbody>
</table>

**STEP 8: Check interaction of tension and shear forces**

<table>
<thead>
<tr>
<th>D.7</th>
<th>$V_{sa}/\phi V_n &gt; 0.2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.7.1</td>
<td>Full strength in tension shall not be permitted.</td>
</tr>
<tr>
<td></td>
<td>$V_{sa}/\phi V_n = 6.0/14.97$</td>
</tr>
<tr>
<td></td>
<td>$= 0.40 &gt; 0.2$</td>
</tr>
<tr>
<td>D.7.2</td>
<td>$N_{sa}/\phi N_n &gt; 0.2$</td>
</tr>
<tr>
<td></td>
<td>Full strength in shear shall not be permitted.</td>
</tr>
<tr>
<td></td>
<td>$N_{sa}/\phi N_n = 8.0/15.97$</td>
</tr>
<tr>
<td></td>
<td>$= 0.50 &gt; 0.2$</td>
</tr>
<tr>
<td>D.7.3</td>
<td>$\frac{N_{sa} + \phi V_n}{\phi N_n} \leq 1.2$</td>
</tr>
<tr>
<td></td>
<td>$\frac{N_{sa} + \phi V_n}{\phi N_n} = 0.50 + 0.40 = 0.90$ (D-3)</td>
</tr>
<tr>
<td></td>
<td>$0.90 &lt; 1.2$ OK</td>
</tr>
</tbody>
</table>

**STEP 9: Calculate minimum plate thickness**

<table>
<thead>
<tr>
<th>D.6.2.3</th>
<th>Select plate thickness equal to or greater than 3/8 in. or half the anchor diameter.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tests(^*)</td>
<td>have also shown that plate rupture is prevented when $d_{i}/t &lt; 2.7$.</td>
</tr>
<tr>
<td>$t_{req}$</td>
<td>$&gt; 0.5/2.7 = 0.19 \text{ in.} &lt; 3/8 \text{ in.}$</td>
</tr>
<tr>
<td>$t$</td>
<td>$= \max(3/8 \text{ in.}, 0.5/2 = 0.25 \text{ in.})$</td>
</tr>
<tr>
<td></td>
<td>$= 3/8 \text{ in.}$</td>
</tr>
</tbody>
</table>


**TENSION**

<table>
<thead>
<tr>
<th>TENSION</th>
<th>Step 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied load</td>
<td>$N_{sa} = 8 \text{ kips}$</td>
</tr>
<tr>
<td>Steel tensile strength</td>
<td>$\phi N_{sa} = 0.8 \times 19.96 = 15.97 \text{ kips}$</td>
</tr>
</tbody>
</table>

**SHEAR**

<table>
<thead>
<tr>
<th>SHEAR</th>
<th>Step 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied load</td>
<td>$V_{sa} = 6 \text{ kips}$</td>
</tr>
<tr>
<td>Steel strength</td>
<td>$\phi V_{sa} = 0.75 \times 19.96 = 14.97 \text{ kips}$</td>
</tr>
</tbody>
</table>

**STEP 9: Calculate minimum plate thickness**

<table>
<thead>
<tr>
<th>D.6.2.3</th>
<th>Select plate thickness equal to or greater than 3/8 in. or half the anchor diameter.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tests(^*)</td>
<td>have also shown that plate rupture is prevented when $d_{i}/t &lt; 2.7$.</td>
</tr>
<tr>
<td>$t_{req}$</td>
<td>$&gt; 0.5/2.7 = 0.19 \text{ in.} &lt; 3/8 \text{ in.}$</td>
</tr>
<tr>
<td>$t$</td>
<td>$= \max(3/8 \text{ in.}, 0.5/2 = 0.25 \text{ in.})$</td>
</tr>
<tr>
<td></td>
<td>$= 3/8 \text{ in.}$</td>
</tr>
</tbody>
</table>

**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.
- \(\phi\)-factors are based on Condition B in D.4.5 of the Code (no supplementary reinforcement).

---

*ASTM F 1554-00 specification, Grade 105, Class 1A, bolt material will be used. Bolt identification is AB105 with a tensile strength in the range of 125 to 150 ksi, and minimum yield strength of 105 ksi for 1/4 and 3 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.*
### STEP 1: Determine required steel area of the bolt.

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.4.1.1</td>
<td>Equate the external factored load to the internal design strength and solve for the required steel area of the bolt.</td>
<td>$\phi N_n \geq N_{u,t}$  \hspace{1cm} (D-1) \hspace{1cm} $N_n = N_{u,t} = nA_{se,t}f_{u,t}$ \hspace{1cm} (D-3)</td>
</tr>
<tr>
<td>D.5.1</td>
<td>Use the tension provisions of D.5.1 to determine the required steel area for tension load.</td>
<td>$A_{se,t} = \frac{N_{u,t}}{\phi n f_{u,t}} = \frac{(40/0.80 \times 1.0 \times 125)}{0.40 \text{ in.}^2}$</td>
</tr>
<tr>
<td>D.5.1.2</td>
<td>Assume embedment will be designed as ductile in accordance with D.3.6.1 (in Step 2). Therefore, $\phi = 0.80$ for tension and 0.75 for shear. Assume bolt is far away from edge.</td>
<td></td>
</tr>
<tr>
<td>D.6.1</td>
<td>Use the shear provisions of D.6.1 to determine the required steel area for shear load.</td>
<td>$A_{se,v} = \frac{V_{u,d}}{\phi n 0.6 A_{se,v} f_{u,t}}$ (D-18)</td>
</tr>
<tr>
<td>D.6.1.2</td>
<td>$A_{se,v} = \frac{V_{u,d}}{\phi n 0.6 A_{se,v} f_{u,t}}$ (D-18)</td>
<td>$A_{se,v} = \frac{V_{u,d}}{\phi n 0.6 A_{se,v} f_{u,t}}$ (D-18)</td>
</tr>
<tr>
<td>D.6.2</td>
<td>Add the area of steel required for tension to the area of steel required for shear.</td>
<td>$A_{se,req} = (0.40 + 0.36)/1.2 = 0.63 \text{ in.}^2$</td>
</tr>
<tr>
<td>D.7.3</td>
<td>This assumes interaction between tension and shear, which will be checked in Step 8.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Anchor diameter, in.</th>
<th>Effective area $A_{se,\text{in.}^2}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0.606</td>
</tr>
<tr>
<td>1.125</td>
<td>0.763</td>
</tr>
</tbody>
</table>

**controls**

Step 1 is to use one 1-1/8 in. diameter headed bolt. $A_{se} = 0.76 \text{ in.}^2 > 0.64 \text{ in.}^2$ OK

Calculate the nominal steel strength $N_{sa}$ in tension.

$N_{sa} = n A_{se} f_{u,t} = 1.0 \times 0.76 \times 125 = 95.0 \text{ kips}$

Calculate the nominal steel strength $V_{sa}$ in shear.

$V_{sa} = n 0.6 A_{se} f_{u,t} = 1.0 \times 0.6 \times 0.76 \times 125 = 57.0 \text{ kips}$

### STEP 2: Determine required embedment length for the bolt to prevent concrete breakout failure in tension.

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.5.2</td>
<td>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 $\phi$-factor used previously. The steel capacity is based on the selected anchor diameter. Use $N_{sa}$ to determine required $h_{ef}$.</td>
<td>From Step 1: $N_{sa} = 95.0 \text{ kips}$</td>
</tr>
<tr>
<td>D.3.6.1</td>
<td>The requirements for ductile design are given in D.3.6.1. For tension load, this requires that $0.85 N_{cb} \geq N_{sa}$</td>
<td>$0.85 N_{cb,\text{req}} \geq N_{sa}$ \hspace{1cm} (D-4) $N_{cb,\text{req}} = N_{sa}/0.85$ $= 95/0.85$ $= 111.8 \text{ kips}$</td>
</tr>
<tr>
<td>D.5.2.1</td>
<td>Calculate concrete breakout strength for a single bolt. For a single bolt away from edge:</td>
<td>$N_{cb} = (A_{se}/A_{Nco}) \psi_{cd} b \psi_{c} N_{b}$  \hspace{1cm} (D-4) $A_{se}/A_{Nco} = 1.0$</td>
</tr>
</tbody>
</table>

---

\[\text{Equation no.}\]

\[\phi N_n \geq N_{u,t}\]

\[N_n = N_{u,t} = nA_{se,t}f_{u,t}\]

\[A_{se,t} = \frac{N_{u,t}}{\phi n f_{u,t}} = \frac{(40/0.80 \times 1.0 \times 125)}{0.40 \text{ in.}^2}\]

\[A_{se,v} = \frac{V_{u,d}}{\phi n 0.6 A_{se,v} f_{u,t}}\]

\[A_{se,req} = (0.40 + 0.36)/1.2 = 0.63 \text{ in.}^2\]

\[N_{sa} = n A_{se} f_{u,t} = 1.0 \times 0.76 \times 125 = 95.0 \text{ kips}\]

\[V_{sa} = n 0.6 A_{se} f_{u,t} = 1.0 \times 0.6 \times 0.76 \times 125 = 57.0 \text{ kips}\]

\[0.85 N_{cb} \geq N_{sa}\]

\[0.85 N_{cb,\text{req}} \geq N_{sa}\]

\[N_{cb,\text{req}} = N_{sa}/0.85\]

\[= 95/0.85\]

\[= 111.8 \text{ kips}\]

\[N_{cb} = (A_{se}/A_{Nco}) \psi_{cd} b \psi_{c} N_{b}\]

\[A_{se}/A_{Nco} = 1.0\]
**CODE SECTION** | **DESIGN PROCEDURE** | **CALCULATION**
---|---|---
**STEP 2 (cont.)** | Modification factors for: | 
D.5.2.5 Edge effects $\psi_{ed,N}$  | $\psi_{ed,N} = 1.0$  
D.5.2.6 Concrete cracking $\psi_{c,N}$  | $\psi_{c,N} = 1.0$  
D.5.2.7 $\psi_{cp,N}$ applies to post-installed anchors only  | $\psi_{cp,N} = N/A$ for bolts

Basic concrete breakout strength:  
$k_c = 24$ for cast-in-place anchors.  

Determine the required effective embedment depth $h_{ef,req}$.  

Determine bolt length $L$.  

Calculate $N_{cb}$ using $h_{ef,provided}$.  

Note: for 11 in. $\leq h_{ef} \leq 25$ 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.

$$N_b = k_c \sqrt[k_c]{h_{ef}}^{1.5}$$  
$$= 24 \sqrt[24]{4000} h_{ef}^{1.5}$$  
$$= 1.52 h_{ef}^{1.5} \text{ kips}$$

$$N_{cb} = (A_{Nc}/A_{Nco}) \psi_{ed,N} \psi_{c,N} N_b$$  

$$111.8 = 1.0 \times 1.0 \times 1.0 \times 1.52 h_{ef}^{1.5}$$  

$$h_{ef,req} = 17.57 \text{ in.}$$

**STEP 3: Check pullout strength of bolt.**

D.5.3 Procedure is the same as that used in Example A1. Calculate the pullout strength of the bolt in tension in accordance with D.5.3.  

Calculate pullout strength.  

Concrete is cracked per problem statement.  

$\psi_{c,P} = 1.0$ for cracked concrete.  

ASTM F 1554 recommends a heavy hex head and washer.  

Appendix A, Table 4(c), $A_{brg} = 1.85 \text{ in.}^2$  

For this combination of steel strength and concrete strength, the anchor head alone is not sufficient to develop the required bearing strength.

Find required net bearing area.  

Design embedment as ductile, in accordance with D.3.6.1:  

$0.85 N_{pn} \geq N_{sa}$

See Appendix A, Table 4, for anchor head areas.

Design a washer to meet the required bearing area. From Table 5, one can see that the SAE washers will not work, and U.S. standard washers seem too thin. Try a square plate.

$0.85 N_{pn} = N_{sa} = 95 \text{ kips} (N_{sa} \text{ from Step 1})$  

$N_{pn,req} = 95/0.85 = 111.8 \text{ kips}$  

$111.8 = 32 A_{brg}$  

$A_{brg,req} = 111.8/32 = 3.49 \text{ in.}^2$

For 1-1/8 in. bolt, hex head area is:  

$A_H = 2.85 \text{ in.}^2$  

$A_{req} = 4.48 - 2.85 = 1.63 \text{ in.}^2$  

Use a square plate 2-1/4 in. each side.  

$A_{washer} = 2.25^2 = 5.06 \text{ in.}^2 > 4.48 \text{ in.}^2$  

OK
### STEP 3 (cont.)

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
</table>
| D.3.6.1      | Calculate \( N_{pn} \) using \( A_{brg, provided} \). | \[ A_{brg, provided} = 5.06 - 0.99 \]  
\[ = 4.07 \text{ in.}^2 > 3.49 \text{ in.}^2 \]  
OK |
| D.3.6.1      | Check ductility: 0.85\( N_{pn} \) > \( N_{sa} \). | \[ N_{pn} = 1.0 \times 32 \times 4.07 = 130.24 \text{ kips} \]  
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 1.5\( h_{ef} \) from the effective perimeter of the washer. Therefore, the concrete breakout strength can be revised. |
| D.3.6.1      | \( A_{brg, provided} = 5.06 - 0.99 \)  
\( = 4.07 \text{ in.}^2 > 3.49 \text{ in.}^2 \)  
OK |
| D.3.6.1      | \( N_{pn} = 1.0 \times 32 \times 4.07 = 130.24 \text{ kips} \)  
\( 0.85 N_{pn} = 0.85 \times 130.24 = 110.7 \text{ kips} > N_{sa} = 95 \text{ kips} \)  
Use 1-1/8 in. diameter AB 105 bolt, embedded 18.5 in., with a square washer 2-1/4 in. sides and 3/8 in. thick. |
| D.6.1.2      | Compute nominal steel strength in shear. | \[ V_{sa} = n0.6A_{se}f_{uta} \]  
\[ = 1.0 \times 0.6 \times 0.76 \times 215 \]  
\[ = 57 \text{ kips} \] |
| D.6.1.2      | Ensure that the embedment design is controlled by the strength of the embedment steel. The requirement for ductile design is given in D.3.6.1. For shear load, this requires that: | \[ 0.85 V_{cb} > V_{sa} \]  
\[ V_{cb, req} = \frac{V_{sa}}{0.85} = \frac{570.85}{0.85} = 67.06 \text{ kips} \] |
| D.6.2.1      | Calculate concrete breakout strength \( V_{cb} \) in shear for a single anchor. | \[ V_{cb} = (A_{Vc}/A_{Vco}) \psi_{ed,V} \psi_{c,V} V_{b} \]  
(D-20) |
| D.6.2.2      | The basic concrete breakout strength \( V_{b} \) is determined using D.6.2.2. | \[ \ell_{c}/d_o = 17.75/1.125 = 15.78 > 8.0 \]  
Use \( \ell_{c}/d_o = 8.0 \)  
\[ V_{b} = 7(\ell_{c}/d_o)^{0.2} \sqrt{d_o a_{f} f_{c}'} c_{a1}^{1.5} \]  
(D-23) |
| D.6.2.6      | Modification factors for: | \( \psi_{ed,V} = 1.0 \)  
\( \psi_{c,V} = 1.0 \) |
| D.6.2.7      | Edge effects \( \psi_{ed,V} \)  
Concrete cracking \( \psi_{c,V} \) | \[ c_{a1, req} = 20.72 \text{ in.} < 24.0 \text{ in.} \]  
OK |
| 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. | \[ V_{cp} = k_{cp} N_{cb} \]  
\[ k_{cp} = 2.0 \]  
\[ N_{cb} = 113.7 \text{ kips} \]  
\[ V_{cp} = 2 \times 113.7 \]  
\[ = 227.4 \text{ kips} >> 95 \text{ kips} \] |

### STEP 4: Check concrete side-face blowout.

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.5.4</td>
<td>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.</td>
</tr>
</tbody>
</table>

### STEP 5: Determine required edge distance to prevent concrete breakout failure in shear.

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
</tr>
</thead>
</table>
| D.6.2.7      | Edge effects \( \psi_{ed,V} \)  
Concrete cracking \( \psi_{c,V} \) | \( \psi_{ed,V} = 1.0 \)  
\( \psi_{c,V} = 1.0 \) |
| D.6.2.6      | Modification factors for: | \( c_{a1, req} = 20.72 \text{ in.} < 24.0 \text{ in.} \)  
OK |
| D.6.2.2      | The basic concrete breakout strength \( V_{b} \) is determined using D.6.2.2. | \[ \ell_{c}/d_o = 17.75/1.125 = 15.78 > 8.0 \]  
Use \( \ell_{c}/d_o = 8.0 \)  
\[ V_{b} = 7(\ell_{c}/d_o)^{0.2} \sqrt{d_o a_{f} f_{c}'} c_{a1}^{1.5} \]  
(D-23) |
| 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) |
| D.6.6        | 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. | \[ V_{cp} = k_{cp} N_{cb} \]  
\[ k_{cp} = 2.0 \]  
\[ N_{cb} = 113.7 \text{ kips} \]  
\[ V_{cp} = 2 \times 113.7 \]  
\[ = 227.4 \text{ kips} >> 95 \text{ kips} \] |

### STEP 6: Check concrete pryout failure.

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
</tr>
</thead>
</table>
| 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. | \[ V_{cp} = k_{cp} N_{cb} \]  
\[ k_{cp} = 2.0 \]  
\[ N_{cb} = 113.7 \text{ kips} \]  
\[ V_{cp} = 2 \times 113.7 \]  
\[ = 227.4 \text{ kips} >> 95 \text{ kips} \] |
**CONCRETE CAPACITY DESIGN (CCD) METHOD—EMBEDMENT DESIGN EXAMPLES**

**STEP 7: Summary.**

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.4.1.2</td>
<td><strong>TENSION</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Applied load</td>
<td>(N_{ua} = 40) kips</td>
</tr>
<tr>
<td>Step 1</td>
<td>Steel strength</td>
<td>(\phi N_{sa} = 0.8 \times 95.0 = 76.0) kips</td>
</tr>
<tr>
<td>D.4.5.a</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Step 2</td>
<td>Concrete breakout strength</td>
<td>(\phi N_{cb} = 0.75 \times 113.7 = 85.3) kips</td>
</tr>
<tr>
<td>D.4.5.c</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Step 3</td>
<td>Concrete pullout strength</td>
<td>(\phi N_{pm} = 0.75 \times 130.24 = 97.68) kips</td>
</tr>
<tr>
<td>D.4.5.c</td>
<td>Concrete side-face blowout strength</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>Design strength of stud in tension</td>
<td>(\phi N_n = \min(\phi N_{sa}, \phi N_{cb}, \phi N_{pm}))</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D.4.1.2</td>
<td><strong>SHEAR</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Applied load</td>
<td>(V_{ua} = 20) kips</td>
</tr>
<tr>
<td>Step 1</td>
<td>Steel strength</td>
<td>(\phi V_{sa} = 0.75 \times 57.0 = 42.75) kips</td>
</tr>
<tr>
<td>D.4.5.a</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Step 2</td>
<td>Concrete breakout strength</td>
<td>(\phi V_{cb} = 0.75 \times 83.70 = 62.78) kips</td>
</tr>
<tr>
<td>D.4.5.c</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Step 3</td>
<td>Concrete pryout strength</td>
<td>(\phi V_{cp} = 0.75 \times 227.4 = 170.55) kips</td>
</tr>
<tr>
<td></td>
<td>Design strength of stud in shear</td>
<td>(\phi V_n = \min(\phi V_{sa}, \phi V_{cb}, \phi V_{cp}))</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**STEP 8: Check interaction of tension and shear forces.**

\[
\frac{V_{ua}}{\phi V_n} > 0.2
\]
\[
\frac{N_{ua}}{\phi N_n} > 0.2
\]

**STEP 9: Calculate minimum plate thickness.**

| AISC | Select plate thickness using the appropriate steel code. This step is not included in this example. |

---

\(N_{ua}/\phi N_n = 40/76.0 = 0.53 > 0.2\)

\(\frac{V_{ua}}{\phi V_n} = 20/42.75 = 0.47 > 0.2\)

\(\frac{V_{ua}}{\phi V_n} \leq 1.2\)

\(\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2\)

\(0.53 + 0.47 = 1.00 < 1.2\)
PART B—Examples: Ductile multiple embedded elements in semi-infinite concrete

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

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

Given:
- Concrete edges
  \[ c_{a1} = c_{a2} > c_{a,min} = 15 \text{ in.} \]
  \[ h_a = 18 \text{ in.} \]

- Concrete
  \[ f'_{c} = 4000 \text{ psi} \]

- Stud material (A29/A108)*
  \[ f_{ya} = 51 \text{ ksi} \]
  \[ f_{uta} = 65 \text{ ksi} \]

- Plate
  \[ F_y = 36 \text{ ksi} \]

- Load
  \[ N_{ua} = 28 \text{ kips} \]

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

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

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

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.4.1.1</td>
<td>Equate internal design strength $\phi N_n$ to the external factored load $N_{sat}$.</td>
<td>$\phi N_n &gt; N_{sat}$ (D-1)</td>
</tr>
<tr>
<td></td>
<td>In ductile design, the internal strength $N_n$ is controlled by the steel strength of the stud, $N_{sa}$.</td>
<td></td>
</tr>
<tr>
<td>D.4.5</td>
<td>The required steel strength of the stud, $N_{sa,req}$, is multiplied by $\phi = 0.80$ for tension because the embedment and the steel stud is ductile and the load factors are based on Appendix C of the Code.</td>
<td>$0.80N_{sa,req} \geq 28$</td>
</tr>
<tr>
<td>D.5.1</td>
<td>Solve for the required steel area $A_{se,req}$ for a single stud.</td>
<td>$N_{sa,req} = 28/0.8 = 35.0$ kips</td>
</tr>
</tbody>
</table>

**Anchor diameter, in.**

| 3/8     | 0.110 | controls (Refer to Appendix A, Table 6) |
| 1/2     | 0.196 |

**STEP 2: Determine the minimum embedment length and spacing for the studs to prevent concrete breakout failure in tension.**

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.3.6.1</td>
<td>Ensure steel strength controls:</td>
<td>From Step 1: $N_{sa} = 51.0$ kips</td>
</tr>
<tr>
<td></td>
<td>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, $N_{sa}$.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>The design concrete breakout strength shall be taken as 0.85 times the nominal strength.</td>
<td>$0.85N_{cbg,req} &gt; N_{sa}$</td>
</tr>
<tr>
<td>D.5.2</td>
<td>$N_{cbg}$ is the nominal concrete breakout strength in tension of a group of anchors.</td>
<td>$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$ (D-5)</td>
</tr>
<tr>
<td>D.5.2.1</td>
<td>$A_{Nc}$ is not calculated because it is assumed that spacing is not limited. Therefore, only the ratio $A_{Nc}/A_{Nco}$ is needed.</td>
<td>$A_{Nc} = 4 \times A_{Nco}$</td>
</tr>
<tr>
<td>D.5.2.2</td>
<td>Modification factors are 1.0 for:</td>
<td>$A_{Nc}/A_{Nco} = 4$</td>
</tr>
<tr>
<td>D.5.2.4</td>
<td>Eccentricity effects $\psi_{ec,N}$</td>
<td>$\psi_{ec,N} = 1.0$</td>
</tr>
<tr>
<td>D.5.2.5</td>
<td>Edge effects $\psi_{ed,N}$</td>
<td>$\psi_{ed,N} = 1.0$</td>
</tr>
<tr>
<td></td>
<td>$c_{a,min} = 15$ in. per problem statement.</td>
<td>$c_{a,min} = 15$ in. per problem statement.</td>
</tr>
<tr>
<td></td>
<td>Edge effect factor $\psi_{ed,N}$ will be 1.0 as long as $c_{a,min} \geq 1.5hef$ or $hef \leq c_{a,min}/1.5$.</td>
<td>$hef \leq c_{a,min}/1.5$.</td>
</tr>
<tr>
<td></td>
<td>Therefore, the embedment $hef$ needs to be less than 15/1.5 = 10 in. to ensure no reduction due to edge distance.</td>
<td>$\psi_{ed,N} = 1.0$</td>
</tr>
<tr>
<td>D.5.2.6</td>
<td>Concrete cracking $\psi_{c,N}$. Concrete is cracked per problem statement.</td>
<td>$\psi_{c,N} = 1.0$</td>
</tr>
<tr>
<td>D.5.2.7</td>
<td>$\psi_{cp,N}$ is not used for cast-in-place anchors.</td>
<td>$\psi_{cp,N} = N/A$ for studs</td>
</tr>
<tr>
<td>D.5.2.2</td>
<td>$N_b$ is the basic concrete breakout strength in tension of single anchor in cracked concrete.</td>
<td>$N_b = k_c\sqrt[1.5]{hef}^{1.5}$ (D-7)</td>
</tr>
<tr>
<td></td>
<td>Assume the embedment will be less than 11 in., and use Eq. (D-7). For cast-in-place anchors, use $k_c = 24$.</td>
<td>$N_b = 24(4000)^{0.5}hef^{1.5}$</td>
</tr>
<tr>
<td>D.5.2.4</td>
<td>$N_{cbg}$ is the nominal concrete breakout strength in tension of a group of anchors.</td>
<td>$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$ (D-5)</td>
</tr>
<tr>
<td></td>
<td>Calculate the minimum required effective embedment depth $hef$ by setting $N_{cbg}$ equal to $N_{cbg,req}$.</td>
<td>$6.08 \times (hef)^{1.5} = 60.0$ kips</td>
</tr>
</tbody>
</table>

**Effective area $A_{se,req}$ controls**

| 3/8     | 0.110 |
| 1/2     | 0.196 |

**Use 1/2 in. diameter studs.**

<table>
<thead>
<tr>
<th>Anchor diameter, in.</th>
<th>Effective area $A_{se,req}$, in. $^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8</td>
<td>0.110</td>
</tr>
<tr>
<td>1/2</td>
<td>0.196</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>3/8</th>
<th>0.110</th>
<th>controls (Refer to Appendix A, Table 6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2</td>
<td>0.196</td>
<td></td>
</tr>
</tbody>
</table>

**EQUATION no.**

- $\phi N_n > N_{sat}$ (D-1)
- $N_{sa,req} = 28/0.8 = 35.0$ kips
- $N_{sa} = nA_{se,futa}$
- $A_{se} = 0.196$ in. $^2 > 0.13$ in. $^2$ OK
- $A_{se} = 0.196$ in. $^2 > 0.13$ in. $^2$ OK
- $N_{sa} = nA_{se,futa}$
- $N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$ (D-5)
**STEP 2 (cont.)**

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
</table>
| **D.5.2**    | Determine the total length $L$ of the stud: | $L_{req} = h_{ef,req} + \text{head thickness + burnoff}$  
$= 4.6 + 0.312 + 0.125$  
$= 5.04$ 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 $s$: | $L_{req} = h_{ef,req} + \text{head thickness + burnoff}$  
$= 4.6 + 0.312 + 0.125$  
$= 5.04$ 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**    | Determine the actual concrete breakout failure $N_{cbg}$ using the actual embedment and spacing. | $N_{cbg} = (\frac{A_{Nc}}{A_{Nco}})\psi_{c,N}\psi_{ed,N}\psi_{c,NNb}$  
(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 $A_{Nco}$ |  
$A_{Nco} = 9 \times h_{ef}^2$  
$= 9 \times (4.81)^2$  
$= 208.2$ in.$^2$  
**D.5.2.1**  | The ratio $A_{Nc}/A_{Nco}$ of is limited to 4. |  
$A_{Nc}/A_{Nco} = 866.1/208.2$  
$= 4.16 > 4.0$  
Use 4.0  
**D.5.2.1**  | 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. |  
$\psi_{c,N}$ = 1.0  
$$N_{cbg} = 4 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 16.03$$  
$= 64.1$ kips  
Check ductility.  
$0.85N_{cbg} > N_{sa}$  
$= 54.5$ kips > 51.0 kips ($N_{sa}$)  
OK  
**D.5.2**  | **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

---

**Note:** Typical values for head thickness and burnoff are provided in Table 6 of Appendix A.
### Code Section: Design Procedure and Calculation

#### STEP 4: Check concrete side-face blowout.

<table>
<thead>
<tr>
<th>Code Section</th>
<th>Design Procedure</th>
<th>Calculation</th>
</tr>
</thead>
</table>
| D.5.4.1      | Check concrete side-face blowout. | $c_{a1} > 0.4h_{ef}$  
|              |                   | $= 0.4 \times 4.81$  
|              |                   | $= 1.92$  
|              |                   | $c_{a1} = 15$  
|              |                   | > $1.92$  |

If $c_{a1} > 0.4h_{ef}$, side-face blowout will not be a factor. (Same is also true for $c_{a2}$).

**OK**

#### STEP 5: Summary.

<table>
<thead>
<tr>
<th>Given</th>
<th>Applied load</th>
<th>$N_{ua} = 28$ kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>Step 1</td>
<td>Design steel tensile strength</td>
<td>$\phi N_{sa} = 0.8 \times 51.0 = 40.8$ kips</td>
</tr>
<tr>
<td>Step 2</td>
<td>Design concrete breakout strength</td>
<td>$\phi N_{c:bg} = 0.75 \times 64.1 = 48.1$ kips</td>
</tr>
<tr>
<td>Step 3</td>
<td>Design concrete pullout strength</td>
<td>$\phi N_{pn} = 0.75 \times 75.6 = 56.7$ kips</td>
</tr>
<tr>
<td>Step 4</td>
<td>Design concrete side-face blowout strength</td>
<td>$\phi N_{sb} = N/A$</td>
</tr>
</tbody>
</table>

**OK**

#### Design strength of stud in tension

- $\phi N_n = \min(\phi N_{sa}, \phi N_{c:bg}, \phi N_{pn})$  
- $= \min(40.8, 48.1, 56.7)$  
- $= 40.8$ kips > $N_{ua} = 28$ kips  

**OK**

#### Ductility:

- $\min(0.85N_{c:bg}, 0.85N_{pn}) > N_{sa}$  
- $\min(0.85 \times 64.1, 0.85 \times 75.6)$  
- $= 54.5$ 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
  $c_{a,\text{min}} = 15$ in.
  $h_\text{a} = 18$ in.
- Base plate
  8 x 8 in.
- Spacing
  $s = 6$ in.
- Concrete
  $f'_c = 4000$ psi
- Stud material (A29/A108)*
  $f_{y,a} = 51$ ksi
  $f_{u,a} = 65$ 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.

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

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

### CODE

<table>
<thead>
<tr>
<th>SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.4.1.1</td>
<td>Equate internal design strength $\phi N_n$ to the external factored load $N_{nat}$</td>
<td>$\phi N_n \geq N_{nat}$</td>
</tr>
<tr>
<td>D.4.5</td>
<td>The required steel strength of the stud, $N_{sa,req}$, is multiplied by $\phi$ = 0.80 for tension because the embedment and the steel stud is ductile and the load factors are based on Appendix C of the Code.</td>
<td>$0.80 N_{sa,req} \geq 28$</td>
</tr>
<tr>
<td>D.5.1</td>
<td>Solve for the required steel area $A_{se,req}$ for a single stud.</td>
<td>$N_{sa,req} = \frac{nA_{se,futa}}{0.80}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$= 35.0$ kips</td>
</tr>
<tr>
<td>D.5.1</td>
<td>Determine the nominal tensile strength $N_{sa}$ of four 1/2 in. diameter studs.</td>
<td>$N_{sa} = nA_{se,futa}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$= 4 \times 0.196 \times 65$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$= 51.0$ kips</td>
</tr>
</tbody>
</table>

### STEP 2: Determine the minimum embedment length and spacing for the studs to prevent concrete breakout failure in tension.

<table>
<thead>
<tr>
<th>D.3.6.1</th>
<th>Ensure steel strength controls:</th>
</tr>
</thead>
<tbody>
<tr>
<td>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, $N_{sa}$.</td>
<td></td>
</tr>
<tr>
<td>The design concrete breakout strength shall be taken as 0.85 times the nominal strength.</td>
<td>$0.85 N_{cbg,req} &gt; N_{sa}$</td>
</tr>
<tr>
<td>$N_{cbg,req} &gt; 51/0.85$</td>
<td></td>
</tr>
<tr>
<td>$N_{cbg,req} &gt; 60.0$ kips</td>
<td></td>
</tr>
<tr>
<td>D.5.2</td>
<td>$N_{cbg}$ is the nominal concrete breakout strength in tension of a group of anchors.</td>
</tr>
<tr>
<td>D.5.2.4</td>
<td>Eccentricity effects $\psi_{ec,N}$</td>
</tr>
<tr>
<td>D.5.2.5</td>
<td>Edge effects $\psi_{ed,N}$</td>
</tr>
<tr>
<td>$c_{a,min} = 15$ in. per problem statement.</td>
<td>$\psi_{ed,N} = 1.0$</td>
</tr>
<tr>
<td>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.</td>
<td></td>
</tr>
<tr>
<td>D.5.2.6</td>
<td>Concrete cracking $\psi_{c,N}$: Concrete is cracked per problem statement.</td>
</tr>
<tr>
<td>D.5.2.7</td>
<td>$\psi_{cp,N}$ is not used for cast-in-place anchors.</td>
</tr>
<tr>
<td>D.5.2.2</td>
<td>$N_b$ is the basic concrete breakout strength in tension of single anchor in cracked concrete.</td>
</tr>
<tr>
<td>$= 1.52 h_{ef}^{1.5}$ kips</td>
<td></td>
</tr>
<tr>
<td>D.5.2.1</td>
<td>$N_{cbg}$ is the nominal concrete breakout strength in tension of a group of anchors.</td>
</tr>
<tr>
<td>Use trial and error; increase $h_{ef}$ until $N_{cbg}$ is equal or greater than the required $N_{cbg,req}$.</td>
<td></td>
</tr>
<tr>
<td>First iteration: Try $h_{ef} = 8$ in.</td>
<td></td>
</tr>
</tbody>
</table>
ANc 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 ANc. Spacing is 6 in. per problem statement.

ANco is the projected area of the failure surface of a single anchor remote from edges.

Ratio of areas.

Basic concrete breakout strength.

Nominal group concrete breakout strength Ncbg.

Because Ncbg of 53.6 kips is less than required (60 kips), one needs to increase the effective embedment depth hef and try again.

Second iteration: Try hef = 9 in.

Determine ANc.

Determine ANco.

Determine the ratio of ANc/ANco.

Basic concrete breakout strength.

Nominal group concrete breakout strength Ncbg.

Because the concrete breakout strength Ncbg of 61.1 kips is greater than the required value of 60 kips, the embedment depth hef of 9 in. will produce a ductile design.

Determine the total length L of the stud:

The total required length L of the stud is equal to the required effective embedment depth hef 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.

Determine actual hef.

Spacing is 6 in. as per problem statement.

Determine ANc.
### Design Procedure

#### Step 2 (cont.)

<table>
<thead>
<tr>
<th>Code Section</th>
<th>Procedure</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.5.2.1</td>
<td>Determine $A_{Nco}$</td>
<td>$A_{Nco} = 9 \times h_{ef}^2 = 9 \times (9.06)^2 = 739 \text{ in.}^2$ (D-6)</td>
</tr>
<tr>
<td>D.5.2.1</td>
<td>Determine the ratio of $A_{Nc}/A_{Nco}$</td>
<td>$A_{Nc}/A_{Nco} = 1100/739 = 1.49$</td>
</tr>
<tr>
<td>D.5.2</td>
<td>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.</td>
<td>$N_b = 1.25 (h_{ef})^{1.5} = 1.25 \times (9.06)^{1.5} = 41.5 \text{ kips}$</td>
</tr>
<tr>
<td></td>
<td>Nominal group concrete breakout strength $N_{cbg}$</td>
<td>$N_{cbg} = 1.49 \times 41.5 = 61.7 \text{ kips}$</td>
</tr>
<tr>
<td></td>
<td>Check ductility. $0.85N_{cbg} &gt; N_{sa}$</td>
<td>$0.85N_{cbg} = 0.85 \times 61.7 = 52.4 \text{ kips} &gt; 51.0 \text{ kips} (N_{sa})$ OK</td>
</tr>
</tbody>
</table>

#### Step 3: Check pullout strength of stud.

<table>
<thead>
<tr>
<th>Code Section</th>
<th>Procedure</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.5.3.1</td>
<td>Determine $N_{pn}$: $N_{pn}$ is the nominal pullout strength in tension of a single anchor.</td>
<td>$N_{pn} = \psi_{c,P}N_p$ (D-14)</td>
</tr>
<tr>
<td>D.5.3.1</td>
<td>$N_p$ is the pullout strength in tension of a single anchor in cracked concrete.</td>
<td>$N_p = 8A_{brg}f_c'$ (D-15)</td>
</tr>
<tr>
<td>D.5.3.5</td>
<td>Concrete is cracked per problem statement.</td>
<td>$\psi_{c,P} = 1.0$</td>
</tr>
<tr>
<td>D.5.3.4</td>
<td>Calculate the bearing area. The anchor head diameter is 1.0 in. for a 1/2 in. diameter stud (Table 6, Appendix A).</td>
<td>$A_{brg} = \pi \times (1.00^2 - 0.50^2)/4 = 0.59 \text{ in.}^2$</td>
</tr>
<tr>
<td></td>
<td>$N_p = 8 \times 0.59 \times 4 = 18.9 \text{ kips}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$N_{pn} = \psi_{c,P}N_p$ (D-14)</td>
<td>$N_{pn} = 1.0 \times 18.9 = 18.9 \text{ kips/stud}$</td>
</tr>
<tr>
<td></td>
<td>$= 18.9 \text{ kips/stud}$</td>
<td>$= 75.6 \text{ kips} (4 \text{ studs})$</td>
</tr>
<tr>
<td>D.3.6.1</td>
<td>Check ductility. $0.85N_{pn} &gt; N_{sa}$</td>
<td>$0.85N_{pn} = 0.85 \times 75.6 = 64.3 \text{ kips} &gt; 51.0 \text{ kips} (N_{sa})$ OK</td>
</tr>
</tbody>
</table>

#### Step 4: Check concrete side-face blowout.

<table>
<thead>
<tr>
<th>Code Section</th>
<th>Procedure</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.5.4.1</td>
<td>Check concrete side-face blowout. If $c_{a1} &gt; 0.4h_{ef}$, side-face blowout will not be a factor. (Same is also true for $c_{a2}$).</td>
<td>$c_{a1} \geq 0.4h_{ef}$</td>
</tr>
<tr>
<td></td>
<td>$\geq 0.4 \times 9.06$</td>
<td>$\geq 3.6 \text{ in.}$</td>
</tr>
<tr>
<td></td>
<td>$c_{a1} = 15 \text{ in.} &gt; 3.6 \text{ in.}$ OK</td>
<td></td>
</tr>
</tbody>
</table>

#### Step 5: Summary.

<table>
<thead>
<tr>
<th>Given</th>
<th>Procedure</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied load</td>
<td>$N_{ua} = 28 \text{ kips}$</td>
<td></td>
</tr>
<tr>
<td>Design steel tensile strength</td>
<td>$\phi N_{sa} = 0.8 \times 51.0 = 40.8 \text{ kips}$</td>
<td></td>
</tr>
<tr>
<td>Design concrete breakout strength</td>
<td>$\phi N_{cbg} = 0.75 \times 61.7 = 46.3 \text{ kips}$</td>
<td></td>
</tr>
<tr>
<td>Design concrete pullout strength</td>
<td>$\phi N_{pn} = 0.75 \times 75.6 = 56.7 \text{ kips}$</td>
<td></td>
</tr>
<tr>
<td>Design concrete side-face blowout strength</td>
<td>$\phi N_{sb} = N/A$</td>
<td></td>
</tr>
<tr>
<td>Design strength of stud in tension</td>
<td>$\phi N_n = \min(\phi N_{sa}, \phi N_{cbg}, \phi N_{pn})$</td>
<td></td>
</tr>
<tr>
<td>$= \min(40.8, 46.3, 56.7)$</td>
<td>$= 40.8 \text{ kips} &gt; N_{ua} = 28 \text{ kips}$ OK</td>
<td></td>
</tr>
</tbody>
</table>
### CODE SECTION  
#### DESIGN PROCEDURE  
#### CALCULATION

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
</table>
| D.3.6.1 | Ductility: | \[
\min(0.85N_{cbg}, 0.85N_{pbn}) > N_{sa} \\
\min(0.85 \times 61.7, 0.85 \times 75.6) > 51.0 \\
\min(52.4, 64.3) = 52.4 \text{ kips} > N_{sa} = 51.0 \text{ kips} \quad \text{OK}
\] |

### STEP 6: Check the required plate thickness.

**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, \( \phi_b = 0.90 \).

\[
M_n = M_p = F_y Z \leq 1.5 M_y
\]

Evaluate plate at Section a-a:

Moment in plate at Section a-a:

\[
\text{Use the midthickness of the tube as a point of fixity.}
\]

Nominal capacity of the plate. The effective width is assumed to be equal to the width of the attachment plus 2\( t \) on either side. Use the plastic section modulus \( Z \).

Assume plate thickness is 1/2 in.

Plastic moment:

\[
\text{Nominal moment capacity } M_n
\]

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\(^ \dagger \):

\[
T_{2\text{studs}} = 28/2 = 14.0 \text{ kips}
\]

\[
a = 1.5 + 3/32 = 1.6
\]

\[
M_{n,a-a} = T_{2\text{stud}} a \\
= 14 \times 1.6 = 22.4 \text{ in.-kips}
\]

\[
M_{n,a-a} = Z \times F_y \\
= 1/4 \times b_{\text{eff},a-a} \times t^2 \\
b_{\text{eff},a-a} = 3 + 2 \times 0.5 + 2 \times 0.5 = 5 \text{ in.}
\]

\[
F_y = 36 \text{ ksi}
\]

\[
M_{n,a-a} = \phi_b Z F_y \\
= 0.9 \times 45t^2 \\
= 40.5t^2
\]

\[
t_{\text{min,a-a}} = \sqrt{22.4 / 40.5} = 0.74 \text{ in.}
\]

\[
T = 28/4 = 7 \text{ kips}
\]

\[
M_{n,a-b} = T w \\
w = 1.5 \times 20.5 + t_{\text{tube}}/2 = 2.22 \text{ in.}
\]

\[
M_{n,a,b} = 7 \times 2.22 = 15 \text{ in.-kips}
\]
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.

\[
M_{p,b-b} = Z \times F_y \\
Z = \frac{1}{4} \times b_{eff,b-b} \times t^2 \\
b_{eff,b-b} = 4 \times (3/4)\text{ in.} \\
F_y = 36\text{ ksi}
\]

\[
M_p = Z \times F_y \\
= \frac{1}{4} \times 3.0 \times t^2 \times 36 \\
= 27t^2
\]

\[
\phi_bM_p = 0.9 \times 27t^2 \\
= 24.3t^2
\]

\[
t_{min,a-a} = \sqrt{\frac{15.5}{24.3}} = 0.80\text{ in.}
\]

Use \(t = 7/8\) in.

Use 8 in. x 8 in. x 7/8 in. embedded plate.

In the above example, the effective embedment \(b_e\) 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
  \[ c_{a1} = c_{a2} = 12 \text{ in.} \]
- Base plate
  8 x 8 in.
- Spacing
  \( s = 6 \text{ in.} \)
- Concrete
  \( f'_c = 4000 \text{ psi} \)
- Anchor material (F 1554 Gr. 36)*
  \( f_{ya} = 36 \text{ ksi} \)
  \( f_{uata} = 58 \text{ ksi} \)
- Anchor type
  Threaded, undercut
  \( k_c = 24 \) from product-specific tests
- Plate
  \( F_y = 36 \text{ ksi} \)
- Load
  \( N_{ua} = 28 \text{ kips} \)

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

Assumptions:
- Concrete is cracked.
- \( \phi \)-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.

*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 \( f_{uata} = 58 \text{ ksi} < 1.9 f_{ya} = 1.9 \times 36 = 64 \text{ ksi} \).
### STEP 1: Determine the required anchor diameter.

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.4.1.1</td>
<td>Equate internal design strength $\phi N_n$ to the external factored load $N_{sa}$</td>
<td>$\phi N_n \geq N_{sa}$ (D-1)</td>
</tr>
<tr>
<td></td>
<td>In ductile design, the internal strength $N_n$ is controlled by the steel strength of the bolt, $N_{sa}$</td>
<td>$N_n = N_{sa} = nA_{se} f_{uta}$</td>
</tr>
<tr>
<td>D.4.5</td>
<td>The required steel strength of the bolt, $N_{sa,req}$, is multiplied by $\phi = 0.80$ for tension because the embedment and the steel bolt is ductile and the load factors are based on Appendix C of the Code.</td>
<td>$0.80 \times N_{sa,req} \geq 28$</td>
</tr>
<tr>
<td>D.5.1</td>
<td>Solve for the required steel area, $A_{se,req}$, for a single bolt. (Refer to Appendix A, Table 3A)</td>
<td>$N_{sa,req} = nA_{se} f_{uta}$ (D-3)</td>
</tr>
<tr>
<td>D.5.1.2</td>
<td>Determine the nominal tensile strength $N_{sa}$ of four 5/8 in. diameter bolts.</td>
<td>$A_{se} = 0.226 \text{ in.}^2 &gt; 0.15 \text{ in.}^2$ (required)</td>
</tr>
<tr>
<td></td>
<td>Note: It is assumed that prying will not occur. See † note in Example B1(b).</td>
<td>OK</td>
</tr>
</tbody>
</table>

### STEP 2: Determine the minimum embedment of the anchors to prevent concrete breakout failure in tension.

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.3.6.1</td>
<td>Ensure steel strength controls:</td>
<td>$N_{cbg} = (A_{Nc}/A_{Nco}) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$ (D-5)</td>
</tr>
<tr>
<td></td>
<td>To prevent concrete breakout failure in tension, the required design concrete breakout strength has to be greater than the nominal tensile strength of the embedment steel, $N_{sa}$.</td>
<td></td>
</tr>
<tr>
<td>D.5.2</td>
<td>The design concrete breakout strength shall be taken as 0.85 times the nominal strength.</td>
<td>$0.85 N_{cbg} &gt; N_{sa}$</td>
</tr>
<tr>
<td>D.5.2.1</td>
<td>$N_{cbg}$ is the nominal concrete breakout strength in tension of a group of anchors.</td>
<td>$N_{cbg} = N_{sa,req} / 0.85$</td>
</tr>
<tr>
<td></td>
<td>$A_{Nc}$ 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 $A_{Nc}$.</td>
<td>$A_{Nc} = [(1.5 \times h_{ef}) + s + c_{a1}]^2$</td>
</tr>
<tr>
<td></td>
<td>Spacing $s$ is 6 in. per problem statement.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Edge distances $c_{a1}$ and $c_{a2}$ are 12 in. per problem statement.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Because it is not known if the edge distance has an effect on $A_{Nc}$ (that is, it needs to be checked that $c_{amin} \leq 1.5 h_{ef}$), it will be assumed it does; $\psi_{ec,N}$ and $A_{Nc}$ will be calculated for each iteration.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_{Nco}$ is the projected area of the failure surface of a single anchor remote from edges.</td>
<td>$A_{Nco} = 9 \times h_{ef}^2$ (D-6)</td>
</tr>
<tr>
<td>D.5.2.4</td>
<td>Modification factors:</td>
<td>$\psi_{ec,N} = 1.0$</td>
</tr>
<tr>
<td>D.5.2.5</td>
<td>Eccentricity effects $\psi_{ec,N}$. Assume no load eccentricity.</td>
<td>$\psi_{ed,N} = [0.7 + 0.3 \times (c_{amin} / (1.5h_{ef})]$ (D-11)</td>
</tr>
<tr>
<td>D.5.2.6</td>
<td>Edge effects $\psi_{ed,N}$.</td>
<td>$\psi_{c,N} = 1.0$</td>
</tr>
<tr>
<td>D.5.2.7</td>
<td>Concrete cracking $\psi_{cp,N}$. Concrete is cracked per problem statement.</td>
<td>$\psi_{cp,N} = 1.0$</td>
</tr>
<tr>
<td>D.8.6</td>
<td>$\psi_{cp,N}$ depends on tests. $\psi_{cp,N} = 1.5 h_{ef} / c_{arc}$. If no tests are done, then $c_{arc} = 2.5 h_{ef}$.</td>
<td></td>
</tr>
<tr>
<td>D.5.2.2</td>
<td>$N_b$ is the basic concrete breakout strength in tension of a single anchor in cracked concrete.</td>
<td>$N_b = k_c \sqrt{f'} h_{ef}^{1.5}$ (D-7)</td>
</tr>
<tr>
<td></td>
<td>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.</td>
<td>$N_b = 24(4000)^{0.5}(h_{ef})^{1.5}$</td>
</tr>
<tr>
<td></td>
<td>Use 5/8 in. diameter bolts</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Edge distances</th>
<th>Spacing</th>
<th>Effective area $A_{se}$ in.²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2</td>
<td>0.142</td>
<td></td>
</tr>
<tr>
<td>5/8</td>
<td>0.226</td>
<td></td>
</tr>
</tbody>
</table>
**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).**

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
</table>
| 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. | $A_{Nc} = (1.5h_{ef} + 18)^2$  
  $= (1.5 \times 9 + 18)^2$  
  $= 992.5$ in.$^2$. |
| D.5.2.1      | Determine $A_{Nc,co}$ | $A_{Nc,co} = 9 \times h_{ef}^2$  
  $= 729$ in.$^2$. |
| D.5.2.1      | Determine the ratio of $A_{Nc}/A_{Nc,co}$. | $A_{Nc}/A_{Nc,co} = 992.3/729 = 1.36$ |
| D.5.2.5      | Determine edge effect factor $\psi_{ed,N}$. | $\psi_{ed,N} = 0.7 + 0.3 \times \left[ c_{a, min}/(1.5h_{ef}) \right]$  
  $= 0.7 + 0.3 \times [12/(1.5 \times 9)]$  
  $= 0.97$ |
| D.5.2.2      | Basic concrete breakout strength. | $N_b = 1.52(h_{ef})^{1.5}$  
  $= 41.0$ kips |
| D.5.2.1      | Nominal group concrete breakout strength $N_{cbg}$. | $N_{cbg} = (A_{Nc}/A_{Nc,co})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,NNb}$  
  $= 54.1$ kips $< 61.6$ kips (No good) |

**Because $N_{cbg}$ is less than $N_{cb,req}$ the effective embedment depth needs to be increased.**

Second iteration: Try $h_{ef} = 16$ in.

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
</table>
| 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 $A_{Nc,co}$ | $A_{Nc,co} = 9 \times h_{ef}^2$  
  $= 2304$ in.$^2$. |
| D.5.2.1      | Determine the ratio of $A_{Nc}/A_{Nc,co}$. | $A_{Nc}/A_{Nc,co} = 1764/2304 = 0.77$ |
| D.5.2.5      | Determine edge effect factor $\psi_{ed,N}$. | $\psi_{ed,N} = 0.7 + 0.3 \times \left[ c_{a, min}/(1.5h_{ef}) \right]$  
  $= 0.85$ |
| D.5.2.1      | Basic concrete breakout strength. | $N_b = 1.52(h_{ef})^{1.5}$  
  $= 97.3$ kips |
| D.5.2.1      | Nominal group concrete breakout strength $N_{cbg}$. | $N_{cbg} = (A_{Nc}/A_{Nc,co})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,NNb}$  
  $= 63.7$ kips $> 61.6$ kips OK for ductility |

**Use 5/8 in. diameter bolt, with 16 in. embedment depth.**

$\phi N_{cbg} = 0.75 \times 63.7$  
$= 47.8$ 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 $\phi N_{pn}$ 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.

$0.85 N_{pn} > N_{sa}$

**D.5.3.1** Determine $N_{pn}$:  
$N_{pn}$ is the nominal pullout strength in tension of a single anchor.

**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 $c_{a1}$).  
\[ c_{a1} \geq 0.4 h_{ef} \]
\[ 0.4 \times 16 = 6.4 \text{ in.} \]
\[ c_{a1} = 12 \text{ in.} \geq 6.4 \text{ in.} \]

**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)  
$\phi N_{pn} = \text{check with manufacturer}$

**Step 4**  
**D.5.4.1** Design concrete side-face blowout strength  
$\phi N_{sb} = N/A$

**D.4.1.2** Design strength of bolt in tension  
$\phi N_r = \min(\phi N_{sa}, \phi N_{cbg})$
\[ = \min(41.9, 47.8) = 41.9 \text{ kips} \]
\[ > N_{ua} = 28 \text{ kips} \]

**D.3.6.1** Check ductility  
$0.85 N_{cbg} \geq N_{sa}$
\[ 0.85 \times 63.7 \geq 52.4 \text{ kips} \]
\[ 54.1 > 52.4 \text{ kips} \]

Plate design: same as Example B1(b).
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.
- $\phi$-factors are based on Condition B in D.4.5 of the Code (no supplementary reinforcement).

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

**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.

Calculate the tension in the anchors.

Determine the required stud area \( A_{se,t} \) for tension. Assume ductile design and use the corresponding \( \phi \) factor in accordance with D.4.5. \( \phi = 0.8 \) for tension load.

From Appendix A, Table 2:

<table>
<thead>
<tr>
<th>Anchor diameter, in</th>
<th>Effective area (gross area for stud), in(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8</td>
<td>0.110</td>
</tr>
<tr>
<td>1/2</td>
<td>0.196</td>
</tr>
<tr>
<td>5/8</td>
<td>0.307</td>
</tr>
</tbody>
</table>

**ANCHORS**

**Tension:**

Determine the required stud area \( A_{se,t} \) for tension. Assume ductile design and use the corresponding \( \phi \) factor in accordance with D.4.5. \( \phi = 0.8 \) for tension load.

\[
T = N_{ula} = \phi N_{ula} = \phi N_{se,t} f_{uta}
\]

\( \phi = 0.80 \)

\( n = \text{two anchors} \)

\( f_{uta} = 65 \text{ ksi} \)

\[
A_{se,t,req} = \frac{N_{ula}}{\phi n f_{uta}} = \frac{14.0}{(0.8 \times 2 \times 65)} = 0.135 \text{ in.}^2
\]

Use four 1/2 in. diameter studs.

\[
A_{se} = A_{se,prov} = 0.196 \text{ in.}^2 \]

**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.
From above, $C_F$ and $T$ are already calculated.

**D.6.1.4** Determine friction force $V_{f_1}$.

*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.\(^2\)

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).

**D.6.1** Strength of front and back anchors, $V_{sa,1}$ and $V_{sa,2}$:

\[
A_{se,v} = A_{se,v,1} + A_{se,v,2} \\
A_{se,v,1} = A_{se,v,2} \\
A_{se,1} = nA_{se,1}f_{uta} = 2 \times 0.196 = 0.392 \text{ in.}^2 \\
V_{sa,1} = V_{sa,2} = nA_{se,1}f_{uta} = 2 \times 0.196 \times 65 = 25.5 \text{ kips} \tag{D-18}
\]

Total strength of front and back anchors:

\[
V_{sa} = V_{sa,1} + V_{sa,2} = 2 \times 25.5 = 51.0 \text{ kips}
\]

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 $f_{ya}$).

Total design steel strength of four anchors:

\[
\phi V_{sa} = 2 \times 19.1 = 38.2 \text{ kips}
\]

**D.6.1.4** Additional shear strength is provided from friction on the compression side of the plate. $\phi$ is taken as 0.75 from Condition B.

Design shear strength of the connection, four anchors plus frictional resistance:

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.

Design strength of anchors $\phi V_{sa,2}$ at Line 2:

\[
\phi V_{sa,2} = 19.1 \text{ kips}
\]

\[
\phi V_{sa,2} \geq V_{sa} = 12.4 \text{ kips} \quad \text{OK}
\]

**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.

\[
N_{sa} = 24\frac{f_{uta}}{A_{sa}} = 24\frac{0.196}{65} = 25.5 \text{ kips}
\]

**D.5.2.2** Calculate the required concrete breakout strength of anchors in tension to ensure that embedment is ductile.

\[
N_{c_{bg}} = 0.85N_{c_{bg,req}} = 25.5/0.85 = 29.9 \text{ kips}
\]

Concrete breakout strength for a group of anchors:

\[
N_{c_{bg}} = (A_{Nc}/A_{Nco})\psi_{rc,N}\psi_{ed,N}\psi_{ec,n}\psi_{ct,N}\psi_{N,NN} \tag{D-5}
\]

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.

\[
h_{ef} = 5.625 + t_{plate} = \text{burnoff (See Appendix A, Table 6)} \\
= 5.625 + 0.625 = 6.125 \text{ in.}
\]

\[
A_{Nco} = 9h_{ef}^2 = 9 \times (6.125)^2 = 337.6 \text{ in.}^2 \tag{D-6}
\]
Calculate $A_{Nc}$. There are no edge effects because $1.5 \times 6.125 = 9.2$ in. $< c_{d1} = 23$ in.

$$A_{Nc} = (2 \times 1.5 \times 6.125 + 5)(2 \times 1.5 \times 6.125) = 429.5 \text{ in.}^2$$

$$A_{Nc}/A_{Nco} = 429.5/337.6 = 1.3$$

Modification factors are 1.0 for:
- Eccentricity effects $\psi_{ec,N}$
- Edge effects $\psi_{ed,N}$
- Concrete cracking $\psi_{c,N}$
- Concrete corner splitting $\psi_{cp,N}$

For cast-in-place anchors, use $k_c = 24$.

Embedment is ductile in tension. Strength controlled by steel. This satisfies shear friction requirement from Step 1.

$$N_b = k_c \sqrt{\frac{h_{ef}}{2}} \cdot \frac{1.5}{h_{ef}}$$

$$N_b = 24(4000)^0.5(6.125)^1.5$$

$$N_b = 23.0 \text{ kips}$$

$$N_{cbg} = \left(\frac{A_{Nc}/A_{Nco}}{\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b}\right)$$

$$N_{cbg} = 1.3 \times 1.0 \times 1.0 \times 23.0$$

$$N_{cbg} = 29.9 \text{ kips} \geq N_{cbg,req} = 29.9 \text{ kips} \quad \text{OK}$$

Use four 1/2 in. diameter x 5-5/8 in. long anchors at 5 in. spacing.

STEP 3: Check pullout strength of anchor.

Concrete is cracked per problem statement.

$$A_{brg} = \pi \times (1.00^2 - 0.50^2)/4$$

$$A_{brg} = 0.59 \text{ in.}^2$$

$$N_{pm} = \psi_{c,P} A_{brg}$$

$$N_{pm} = 1.0 \times 32 \times 0.59$$

$$N_{pm} = 18.9 \text{ kips}$$

Ductile 1/2 in. diameter studs are OK for pullout.

STEP 4: Check concrete side-face blowout.

Check anchors closest to the edge for side-face blowout.

$$c_{d1} > 0.4h_{ef}$$

18 in. $> 0.4 \times 6.125 = 2.45$ in.

Side-face blowout $N_{sb}$ need not be checked.
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.

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.6.2.1</td>
<td>For a plate with welded studs, Mode 2 can be used to calculate the concrete breakout strength. Therefore, $c_a = 18 + 5 = 23$ in. The failure cone passes through the bottom of the slab, so the $A_{nc}$ equation uses the full slab thickness ($h_a = 18$ in. $&lt; 23$ in.).</td>
<td></td>
</tr>
</tbody>
</table>

- $V_{cbg} = (A_{Vc}/A_{Vco}) \psi_{ec,V} \psi_{ed,V} \psi_{c,V} V_b \quad \text{(D-21)}$
- $A_{Vc} = (3c_a + s)(h_a) = (3 \times 23 + 5)(18) = 1332 \text{ in.}^2$
- $A_{Vco} = 4.5c_a^2 = 4.5 \times 23^2 = 2381 \text{ in.}^2 \quad \text{(D-22)}$
- $A_{Vc}/A_{Vco} = 0.56$

- Modification factors:
  - Eccentricity on anchor group $\psi_{ec,V}$
  - Second edge effects $\psi_{ed,V}$
  - Assumed crack $\psi_{c,V}$

- Note: By definition, $l_e$ is limited to $8d_o = 4$ in.

| D.6.2.5 | Eccentricity on anchor group $\psi_{ec,V}$ |
| D.6.2.6 | Second edge effects $\psi_{ed,V}$ |
| D.6.2.7 | Assumed crack $\psi_{c,V}$ |

- For $V_b = 8(l_e/d_o)^{0.2} \sqrt{l_e} \sqrt{f_{cu}} (c_a)^{1.5} \quad \text{(D-24)}$
- $V_b = 8(4/0.5)^{0.2} \sqrt{0.5} \sqrt{4000} (23)^{1.5}$
- $V_b = 59.8 \text{ kips}$
### CONCRETE CAPACITY DESIGN (CCD) METHOD—EMBEDMENT DESIGN EXAMPLES

<table>
<thead>
<tr>
<th>CODE SECTION</th>
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<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>STEP 5 (cont.)</td>
<td>Nominal concrete breakout strength:</td>
<td>$V_{cbg,2} = 0.56 \times 1.0 \times 1.0 \times 1.0 \times 59.8 = 33.5 \text{ kips}$</td>
</tr>
<tr>
<td>D.3.6.1 Check ductility, two anchors on Line 2:</td>
<td>$0.85V_{cbg,2} = 0.85 \times 33.5 = 28.5 &gt; V_{sa,2} = 25.5 \text{ kips}$</td>
<td>Mode 2 is ductile.</td>
</tr>
<tr>
<td>D.4.5 Check for strength:</td>
<td>$\phi = 0.75$</td>
<td></td>
</tr>
<tr>
<td>D.6.2.1 Design group concrete breakout strength:</td>
<td>$\phi V_{cbg} = 0.75 \times 33.5 = 25.1 &gt; V_{ua} = 8.2 \text{ kips}$</td>
<td>OK for strength</td>
</tr>
</tbody>
</table>

#### 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) | |
| Two anchors: | $k_{cp} = 2$ for $h_{ef} > 2.5 \text{ in.}$ | |
| | $N_{cbg} = 29.9 \text{ kips}$ | |
| | $V_{cpg} = 2 \times 29.9 = 59.8 \text{ kips}$ | |
| | $0.85 V_{cpg} = 0.85 \times 59.8 = 50.8 > 25.5 \text{ kips}$ | Ductile |

#### STEP 7: Summary

- **TENSION**
  - Stud:
    - Diameter $d_o = 1/2 \text{ in.}$
    - Length $L = 5-5/8 \text{ in.}$
    - Effective depth $h_{ef} = 6.13 \text{ in.}$
    - Base plate thickness $t = 5/8 \text{ in.}$
    - Anchors are ductile.
  - Applied load $N_{ua} = 14.0 \text{ kips}$
  - Steel strength $\phi N_{sa} = 0.8 \times 25.5 = 20.4 \text{ kips}$
  - Concrete breakout strength $\phi N_{cbg} = 0.75 \times 29.9 = 22.4 \text{ kips}$
  - Concrete pullout strength $\phi N_{pu} = 0.75 \times 2 \times 18.9 = 28.4 \text{ kips}$

- **SHEAR: Mode 2 (frictional resistance counted)**
  - Applied load, $12.4 - 4.2 = 8.2 \text{ kips}$ $V_{sa} = 8.2 \text{ kips}$
  - Steel strength $\phi V_{sa} = 0.75 \times 25.5 = 19.1 \text{ kips}$
  - Concrete breakout strength (nonductile) $\phi V_{cbg} = 0.75 \times 33.5 = 25.1 \text{ kips}$
  - Concrete pryout strength $\phi V_{cpg} = 0.75 \times 59.8 = 44.9 \text{ kips}$
  - Design strength of stud in shear $\phi V_{n} = 19.1, 25.1, 44.9 > V_{sa} = 8.2 \text{ kips}$ | OK |
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.

\[ \frac{N_{au}}{\phi N_n} + \frac{V_{au}}{\phi V_n} \leq 1.2 \quad \text{(D-30)} \]

Mode 2:
Design shear strength in shear:
\[ V_{au} > 0.2 \times \text{design shear strength} \text{— interaction check is required.} \]

If the frictional resistance was ignored, then the aforementioned interaction equation becomes:

and the anchors will have to be redesigned.

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 \( \phi \)-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 (\( \phi = 0.9 \) 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 (\( M_p = M_y = ZF_y \)). 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.
- \( \phi \)-factors are based on Condition B in D.4.5 of the Code (no supplementary reinforcement).

---

*ASTM F 1554-00 specification, Grade 105, Class 1A, 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 than 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 \( \leq 1.9f_{ya} \) or 125 ksi. Refer also to Table 1 for other materials.
### DESIGN PROCEDURE

#### 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 2t 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
Determined the required rod area $A_{s,t}$ for tension. Assume ductile design and use the corresponding $\phi$-factor in accordance with D.4.5.

#### D.4.5
Try 1/2 in. diameter rods. Gross cross-sectional area is 0.196 in.$^2$

<table>
<thead>
<tr>
<th>Anchor diameter, in.</th>
<th>Effective area (gross area for stud), in.$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8</td>
<td>0.078</td>
</tr>
<tr>
<td>1/2</td>
<td>0.142</td>
</tr>
<tr>
<td>5/8</td>
<td>0.226</td>
</tr>
</tbody>
</table>

- **Because this is a surface-mounted plate with cast-in headed bolts, direct shear is applicable.**

#### D.6.1.2(b)
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**
  \[
  V_{sa} = 0.60nA_{s,tv}f_{uta} \\
  = 0.60 \times 2 \times 0.142 \times 120 \\
  = 40.9 \text{ kips}
  \]
- **Strength reduction factor**
  \[
  \phi = 0.75
  \]
- **Design strength of four anchors**
  \[
  \phi V_{sa} = 0.75 \times 40.9 = 30.68 \text{ kips}
  \]

#### 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.)**

#### D.4.5.c
- **Strength reduction factor**
  \[
  \phi = 0.75
  \]
- **Design strength provided by friction**
  \[
  \phi V_{f} = 0.75 \times 7 = 5.25 \text{ kips}
  \]
  \[
  \phi V_{sa} + \phi V_{f} = 30.68 + 5.25 = 35.93 \text{ kips} > 12.4 \text{ kips} \quad \text{OK}
  \]
### Design Procedure Calculation

#### STEP 2: Design base plate.

**ASIC LFRD Chapter F**
- Nominal flexural strength of base plate, per AISC LRFD Code, is $M_n = M_p = F_yZ$. The tension in the anchor is based on the applied moment, not the full tensile capacity of the anchor.

- The $\phi$-factor for flexure is $\phi_b = 0.90$.

- Required plate thickness per AISC:

  \[
  \phi_b M_n = \phi_b F_y Z = 0.9 F_y \left( \frac{bt^2}{4} \right) = 0.9(36)(7)\frac{t^2}{4} = 56.7t^2 \\
  \]

  \[
  t = \frac{56.7}{17.5} = 0.55 \text{ 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} = n A_{se,t} f_u = 2 \times 0.142 \times 120 = 34.1 \text{ kips} \\
  
  \]

**D.3.6.1**
- Calculate the concrete breakout strength of anchors in tension so that embedment is ductile.

  \[
  N_{cbg} = \frac{N_{cbg,req}}{0.85} = \frac{34.1}{0.85} = 40.6 \text{ kips} \\
  
  \]

**D.5.2**
- Concrete breakout strength for a group of anchors

  \[
  N_{cbg} = (A_{Nc}/A_{Nco}) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} N_b \\
  A_{Nco} = 9 \times 8.5^2 = 650 \text{ in.}^2 \\
  A_{Nc} = (2 \times 1.5 \times 8.5 + 5)(2 \times 1.5 \times 8.5) = 778 \text{ in.}^2 \\
  A_{Nc}/A_{Nco} = 778/650 = 1.20 \\
  
  \]

**D.5.2.4**
- Modification factors are 1.0 for:

  - Eccentricity effects $\psi_{ec,N}$
  - Edge effects $\psi_{ed,N}$
  - Concrete cracking $\psi_{c,N}$
  - Concrete corner splitting $\psi_{cp,N}$

**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 \frac{F_y}{f'c} h_{ef}^{1.5} = 24 \sqrt{(4000)} h_{ef}^{1.5} = 1518 h_{ef}^{1.5} \text{ lb} = 1.52 \times 8.5^{1.5} = 37.6 \text{ 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 \text{ kips} \text{ 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.
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.

\[ N_{pm} = \psi_c p N_p \]  
(D-14)

\[ N_p = 8 A_{brg} f'_c \]  
= 8 \times 4 A_{brg}  
= 32 A_{brg}  
(D-15)

Concrete is cracked per problem statement.

\[ \psi_c = 1.0 \]

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.

\[ A_H = (3F^2/2)\tan30\text{ degrees} \]
\[ A_H = (3 \times 0.875^2/2)(0.577) = 0.663 \text{ in.}^2 \]
\[ A_D = \pi \times 0.5^2/4 = 0.196 \text{ in.}^2 \]

Pullout capacity for two anchors

\[ N_{pm} = 1.0 \times 32 \times 0.47 \]
\[ = 15.0 \text{ kips each anchor} \]

Determine required bearing area

\[ 0.85 N_{pm} = 0.85 \times 2 \times 15.0 \]
\[ = 25.5 < N_{sa} = 34.1 \text{ kips} \]
No good

\[ A_{brg,req} = 34.1/(2 \times 32 \times 0.85) \]
\[ = 0.625 \text{ in.}^2 \] 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.

\[ A_H = \pi \times 1.167^2/4 = 1.07 \text{ in.}^2 \]
\[ A_{brg} = A_H - A_D \]
\[ = 1.07 - 0.196 \]
\[ = 0.874 > 0.625 \text{ in.}^2 \]

Check:

\[ N_{pm} = 1 \times 32 \times 0.874 \]
\[ = 27.9 \text{ kips each anchor} \]
\[ = 55.9 \text{ kips for two anchors} \]

\[ 0.85 N_{pm} = 0.85 \times 55.9 = 47.5 < N_{sa} = 34.1 \text{ kips} \]
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.

\[ c_{sf} > 0.4 h_{sf} \]
18 in. > 0.4 \times 6.125 = 2.45 in.

Side-face blowout \( N_{sh} \) need not be checked.
**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.

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<thead>
<tr>
<th>Concrete breakout</th>
<th>Steel strength</th>
<th>Applied load</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_{a1}$</td>
<td>$0.85V_{cbg,1}$</td>
<td>$V_{sa,1}$</td>
</tr>
<tr>
<td>18 in.</td>
<td>22.4 kips</td>
<td>20.5 kips</td>
</tr>
</tbody>
</table>

Concrete breakout              Steel strength               Applied load

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

<table>
<thead>
<tr>
<th>Concrete breakout</th>
<th>Steel strength</th>
<th>Applied load</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_{a1}$</td>
<td>$0.85V_{cbg,2}$</td>
<td>$V_{sa,2}$</td>
</tr>
<tr>
<td>23 in.</td>
<td>24.9 kips</td>
<td>20.5 kips</td>
</tr>
</tbody>
</table>

Ductile—OK for strength; calculated below

See note above.

<table>
<thead>
<tr>
<th>Concrete breakout</th>
<th>Steel strength</th>
<th>Applied load</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_{a1}$</td>
<td>$0.85V_{cbg,1}$</td>
<td>$V_{sa,1}$</td>
</tr>
<tr>
<td>18 in.</td>
<td>22.4 kips</td>
<td>20.5 kips</td>
</tr>
</tbody>
</table>

Ductile—OK for strength.
## DESIGN PROCEDURE

<table>
<thead>
<tr>
<th>Mode 1:</th>
<th>Because the plate is not rigidly connected to the anchor, check the anchors nearest to the edge using 1/2 the applied shear. Therefore, ( c_{a1} = 18 ) in. Note that the depth of ( A_{Vc} ) is limited by ( h_a ).</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eccentricity on anchor group ( \psi_{ec,V} )</td>
<td>( \psi_{ec,V} = 1.0 ) (no eccentricity)</td>
</tr>
<tr>
<td>Second edge effect ( \psi_{ed,V} )</td>
<td>( \psi_{ed,V} = 1.0 \ c_{a2} &gt; 1.5c_{a1} )</td>
</tr>
<tr>
<td>Second edge effect, ( \psi_{c,V} ) assumed cracked.</td>
<td>( \psi_{c,V} = 1.0 )</td>
</tr>
</tbody>
</table>

**Note:** By definition, \( l_e \) is limited to 8\( d_o = 4 \) in.

### Step 2:

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 \( h_a \).

### Component Calculations

**Strength:** From Step 1, \( V_{sa} = 40.9/2 = 20.5 \) kips (two anchors)

**Ductility check:**

\[
0.85V_{cbg} = 0.85(26.4) = 22.4 > V_{sa} = 20.5 \text{ kips}
\]

Ductile, OK

<table>
<thead>
<tr>
<th>Mode 2:</th>
<th>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 ).</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eccentricity on anchor group ( \psi_{ec,V} )</td>
<td>( \psi_{ec,V} = 1.0 ) (no eccentricity)</td>
</tr>
<tr>
<td>Second edge effect ( \psi_{ed,V} )</td>
<td>( \psi_{ed,V} = 1.0 \ c_{a2} &gt; 1.5c_{a1} )</td>
</tr>
<tr>
<td>Second edge effect ( \psi_{c,V} ) assumed cracked.</td>
<td>( \psi_{c,V} = 1.0 )</td>
</tr>
</tbody>
</table>

**Note:** By definition, \( l_e \) is limited to 8\( d_o = 4 \) in.

### Component Calculations

**Strength:** From Step 1, \( V_{sa} = 40.9/2 = 20.5 \) kips (two anchors)

**Ductility check:**

\[
0.85V_{cbg} = 0.85(29.2) = 24.9 > V_{sa} = 20.5 \text{ kips}
\]

Ductile, OK
**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).)

<table>
<thead>
<tr>
<th>$V_{cpq}$</th>
<th>$k_{cp} \times N_{cbg}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k_{cp} = 2$ for $h_{ef} &gt; 2.5$ in.</td>
<td></td>
</tr>
<tr>
<td>$N_{cbg} = 45.1$ kips</td>
<td></td>
</tr>
<tr>
<td>$V_{cpq} = 2 \times 45.1 = 90.2$ kips</td>
<td></td>
</tr>
<tr>
<td>$\phi V_{cpq} = 0.75 \times 90.2 = 67.7$ kips &gt; 12.4 kips</td>
<td>OK</td>
</tr>
</tbody>
</table>

**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) |
| Steel strength | $\phi N_{sa} = 0.8 \times 34.1 = 27.3$ kips |
| Concrete breakout strength | $\phi N_{cbg} = 0.75 \times 45.1 = 33.8$ kips |
| Concrete pullout strength | $\phi N_{pn} = 0.75 \times 55.9 = 41.9$ kips |
| Concrete side-face blowout strength | $c_{a1} > 0.4h_{ef}$ so this is not applicable |
| Design strength of stud in tension | $\phi N_n = \min(\phi N_{sa}, \phi N_{cbg}, \phi N_{pn})$ |
| | = $\min(27.3, 33.8, 41.9)$ |
| | = 27.3 kips > $N_{ua} = 17.5$ kips | OK |

**SHEAR: Mode 2 (frictional resistance considered)**

| Step 1 Applied load | $V_{sa} = 12.4 - 5.25 = 7.2$ kips |
| Steel strength, two anchors | $\phi V_{sa} = 0.75 \times 20.5 = 15.4$ kips |
| Concrete breakout strength (nonductile) | $\phi V_{cbg} = 0.75 \times 29.2 = 21.9$ kips |
| Concrete pryout strength | $\phi V_{cpq} = 0.75 \times 90.2 = 67.7$ kips |
| Design strength of stud in shear | $\phi V_n = \min(\phi V_{sa}, \phi V_{cbg}, \phi V_{cpq})$ |
| | = $\min(15.4, 21.9, 67.7)$ |
| | = 15.4 kips > $V_{sa} = 7.2$ kips | OK |
STEP 9: Check for tension-shear interaction.

D.7.3 Tension-shear interaction, Mode 2. \[\frac{17.5}{27.3} + \frac{7.2}{15.4} = 1.11 < 1.2\] 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 $\phi$-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 ($\phi = 0.9$ 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 ($M_p = ZF_y$). 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
  - $c_{a1} = 18\text{ in.}$
  - $c_{a2} > 24\text{ in.}$
  - $h_a = 18\text{ in.}$
- Base plate
  - 12 x 12 in.
- Bolt spacing
  - $s = 8\text{ in.}$
- Concrete
  - $f'_{c1} = 4000\text{ psi (concrete)}$
  - $f'_{c2} = 9000\text{ psi (grout)}$
- Bolt material (F 1554 Gr. 36 anchor rods*):
  - $f_{ya} = 36\text{ ksi}$
  - $f_{uta} = 58\text{ ksi}$
- Plate
  - $F_y = 36\text{ ksi}$
- Loads
  - $V_{ua} = \pm 7\text{ kips}$
  - $e = 18\text{ in. (height of stub column above concrete surface)}$
  - $M_u = V_{ua} \times e = 126.0\text{ in.-kips}$
  - $P_u = 3.0\text{ kips}$
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.
- $\phi$-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 ($f_{uta} = 58\text{ ksi} < 1.9f_{ya} = 1.9 \times 36 = 64\text{ ksa}$).
STEP 1: Design for moment and tension.

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.

\[ d = \frac{12 \text{ in.} - 10 \text{ in.}}{2} = 1.0 \text{ in.} \]

\[ d = 12 \text{ in.} - d_e - \frac{12 \text{ in.} - 8 \text{ in.}}{2} = 9 \text{ in.} \]

\[ N_{u,2} = N_{u,M} + P_a/2 \]

\[ N_{u,2} = 14 \text{ kips} + (3 \text{ kips/2}) \text{ (in two bolts)} \]

\[ N_{u,2} = 15.5 \text{ kips} \]

Likewise, the compression on the bearing area can be reduced directly by the tension force even though forces are centered at different locations. \( \phi \) is 0.75 for tension strength in steel.

\[ N_{u,2} = \phi N_{u,a} \]

\[ N_{u,2} = \phi N_{u,a} \]

\[ N_{u,2} = \phi N_{u,a} \]

\[(D-3)\]
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 $\phi$-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.

\[
A_{s,et,req} = \frac{N_{u,2}}{\phi_A f_{ut,a}}
\]
\[
A_{s,et,req} = 15.5 / 0.75(2)58
\]
\[
A_{s,et,req} = 0.18 \text{ in.}^2
\]

Use 3/4 in. diameter threaded rods.

\[
A_D = 0.44 \text{ in.}^2 \text{ (nominal area)}
\]
\[
A_{s,e,t} = 0.334 \text{ in.}^2 \text{ (effective area)}
\]

\[
\phi N_{s,u} = \phi A_{s,e,t} f_{ut,a}
\]
\[
\phi N_{s,u} = 0.75(0.334)(2)58
\]
\[
\phi N_{s,u} = 29.06 \text{ (two bolts)}
\]
\[
\phi N_{s,u} > N_{u,2}
\]
\[
29.06 \text{ kips} > 15.5 \text{ kips} \quad \text{OK}
\]

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:
STEP 2 (cont.)

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

SHEAR FORCE TOWARD EDGE

\[
\begin{align*}
V_u & \quad \text{(Shear force towards edge)} \\
V_{ua} & \quad \text{(Anchor force)} \\
V_{val/2} & \quad \text{(Half of vertical anchor force)}
\end{align*}
\]

PLAN

C) Shear resisted by two anchors closest to edge:

SHEAR FORCE AWAY FROM EDGE

\[
\begin{align*}
N_u & \quad \text{(Normal force)} \\
V_u & \quad \text{(Shear force away from edge)} \\
V_{ua} & \quad \text{(Anchor force)} \\
V_{val/2} & \quad \text{(Half of vertical anchor force)}
\end{align*}
\]

SECTION A-A

D) Shear resisted by two anchors farthest from edge
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.

**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)

\[
V_{ua} \leq \phi V_n
\]

\[
\phi V_n = \phi(0.8) V_{sa} + \phi(0.4 C_F) \tag{D-19}
\]

\[
\phi(0.8)0.6nA_{se f_{ult}} = 0.65(0.8)(0.62)(0.334)58
\]

\[
\phi(0.8)0.6nA_{se f_{ult}} = 12.09 \text{ kips}
\]

\[
\phi(0.4 C_F) = 0.70(0.4(12.5)) = 3.50 \text{ kips}
\]

\[
V_{ua} \leq \phi(0.8)0.6nA_{se f_{ult}} + \phi(0.4 C_F) \tag{D-19}
\]

\[
7 \leq 12.09 + 3.50
\]

\[
7 \leq 15.39 \text{ kips} \quad \text{OK}
\]

**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.

\[
x = (b - s)/2 - d_e - t_f
\]

\[
y = (s/2) - (t_{web}/2)
\]

\[
x = (12 \text{ in.} - 8 \text{ in.})/2 - 1 \text{ in.} - 0.27 \text{ in.} = 0.73 \text{ in.}
\]

\[
y = 8 \text{ in.}/2 - 0.115 \text{ in.} = 3.88 \text{ in.}
\]

\[
b_{eff} = 4t = 4(1 \text{ in.}) = 4 \text{ in.}
\]

\[
M_y = N_{as,bolt}(x) = 7.8(0.73 \text{ in.})
\]

\[
M_y = 5.7 \text{ kip-in.}
\]

\[
M_x = N_{as,bolt}(y) = 7.8(3.88 \text{ in.})
\]

\[
M_x = 30.1 \text{ kip-in.}
\]
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 \( C_F/\alpha_{brg,pl} \).

\[
\phi M_n = 0.9 F_{y}/Z = 0.9 F_{y}(d_{ef}/4) = 32.4 \text{ kips}
\]

\[
\phi M_n \geq M_u = 34.2 \text{ kip}-\text{in.} \geq 30.6 \text{ kip-in.} \quad \text{OK}
\]

Use 1 in. thick plate.

\[
C_F = 12.5 \text{ kips}
\]

\[
t = \max(t,d_e) \sqrt{\frac{2 C_F}{0.9 F_{y} A_{brg,pl}}}
\]

\[
t = 1.0 \sqrt{\frac{2(12.5)}{0.9(36)13.53}}
\]

\[
t = 0.24 \text{ in.} < 1.0 \text{ in.} \quad \text{OK}
\]

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.

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:

\( d_o = 3/4 \text{ in.} \)

\( th = 2 \text{ in.} \)

\( E = 13.25 \text{ in.} \)

\( L = 18.5 \text{ in.} \) (for a double nut)

\[
N_{sa} = 0.334 \times 2 \times 58 = 38.7 \text{ kips}
\]

\[
N_{sa,2} = 15.5 \text{ kips}
\]

\[
N_{sa} < 0.85 N_{cbg} \quad \text{(for steel to control)}
\]

\[
N_{cbg,req} > 38.7 \text{ kips}/0.85 = 45.5 \text{ kips}
\]
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.)

**Basic concrete breakout strength $N_b$** of a single anchor in tension in cracked concrete.

- **Tensile strength controlled by steel.**
- **Check strength:**
  - $\phi = 0.70$ for tension strength in concrete against tension breakout.
- **Tension-shear interaction remains as computed in Step 9.**

**STEP 5: Check pullout strength of anchor.**

- **Calculate the pullout strength of the anchor in tension in accordance with** D.5.3.

- **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.

- **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 times 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.

**Calculation:**

- $h_{ef} = 12$ in.
- $N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}N_b$ (D-5)

- $A_{Nc} = (1.5h_{ef} + c_{at})(1.5h_{ef} + s_1 + 1.5h_{ef})$

- $c_{at} = 8$ 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.$^2 \leq nA_{Nco} = 2(1296) = 2592$ in.$^2$  OK

- $\psi_{ec,N} = 1$ (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)

- $N_b = 16\sqrt{\frac{h_{ef}^5}{\pi}} 11$ in. $< h_{ef} < 25$ in. (D-8)

- $N_b = 16\sqrt{4000(12)^{5/3}} = 63.65$ kips

- $N_{cbg} = (1144/1296)(1.0)(0.833)(1.0)(1.0)63.65$ kips

- $N_{cbg} = 46.8$ kips $> (N_{sa}/0.85) = 45.5$ kips

- Ductile. OK.

- Embedment is ductile for tension

- $\phi N_{cbg} = 0.7(46.80$ kips $) \geq \phi N_{sa}$

- $\phi N_{cbg} = 32.76$ kips $> 29.06$ kips

- **Use four rods 3/4 in. diameter x 18.5 in. long with $h_{ef} = 12$ in.**
**STEP 5 (cont.)**

<table>
<thead>
<tr>
<th>D.4.4</th>
<th>( \phi ) is 0.70 for pullout strength and 0.75 for tension strength of steel.</th>
</tr>
</thead>
<tbody>
<tr>
<td>( N_{pn} = \psi_{c}PA_{br}8f_{c}^{'} )</td>
<td>( N_{pn} = 1.0(1.1)(8)(4000) = 35.2 ) kips</td>
</tr>
<tr>
<td>( N_{np} &gt; N_{ns}/0.85 ) (for ductile response)</td>
<td>( nN_{pn} = 2(35.2) = 70.4 ) kips &gt; 45.5 kips</td>
</tr>
<tr>
<td>( nN_{pn} &gt; \phi N_{sa} )</td>
<td>Ductile, OK</td>
</tr>
<tr>
<td>( \phi N_{pq} = 0.7(70.4) = 49.3 ) kips</td>
<td>49.3 kips &gt; ( \phi N_{sa} = 29.06 ) kips</td>
</tr>
</tbody>
</table>

\[ N_{pn} = \psi_{c}PA_{br}8f_{c}^{'} \]

\[ N_{pn} = 1.0(1.1)(8)(4000) = 35.2 \] kips

\[ nN_{pn} = 2(35.2) = 70.4 \] kips > 45.5 kips

\[ \phi N_{pq} = 0.7(70.4) = 49.3 \] kips

\[ 49.3 \] kips > \( \phi N_{sa} = 29.06 \) kips

**STEP 6: Check concrete side-face blowout.**

<table>
<thead>
<tr>
<th>D.5.4</th>
<th>Check if side-face blowout needs to be investigated using the Code limits given in Section D.5.4.</th>
</tr>
</thead>
<tbody>
<tr>
<td>( c_{at} = 8 ) in. &gt; 0.4( h_{ef} ) = 0.4(12) = 4.8 in.</td>
<td>OK. Ignore side-face blowout.</td>
</tr>
</tbody>
</table>

**STEP 7: Check concrete shear breakout.**

<table>
<thead>
<tr>
<th>D.6.2</th>
<th>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.</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.6.2.1</td>
<td>Two bolts failure cone: Option (a)</td>
</tr>
<tr>
<td>D.6.2.2</td>
<td>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_{sa} = 16 ) in. &lt; 1.5( c_{a1} = 1.5 \times 16 = 24 ) in. Limit cone depth to 16 in.</td>
</tr>
</tbody>
</table>

\[ V_{cbg,req} \geq 23.2 \] kips/0.85 = 27.3 kips (two bolts)

\[ V_{cbg} \]

\[ (AV_{c}/AV_{co}) \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{v,V} V_{b} \] (D-21)

\[ V_{b} = 7(\ell_{e}/d_{o})^{0.2} \sqrt{A_{V_{c}}/A_{V_{co}}}^{1.5} \] (D-23)

\[ A_{V_{c}} = (2(1.5c_{a1}) + s)h_{a} \]

\[ A_{V_{co}} = 4.5c_{a1}^{2} = 4.5(16)^{2} = 1152 \] in.²

\[ \psi_{ec,V} = 1.0 \]

\[ \psi_{ed,V} = 1.0 \]

\[ \psi_{c,V} = 1.0 \]

\[ \ell_{e} = \min(h_{ef}, 8d_{o}) = \min(12, 8(0.75)) \]

\[ \ell_{e} = \min(12, 6) = 6 \] in.

\[ V_{b} = 7(6/0.75)^{0.2} \sqrt{1.5} \sqrt{4000} (16)^{1.5} \]

\[ V_{b} = 37.2 \] kips

\[ V_{cbg} = \frac{896}{1152} \]

\[ V_{cbg} = 28.9 \] kips > 27.3 kips

**Ductile**
### CODE SECTION | DESIGN PROCEDURE | CALCULATION
--- | --- | ---
**STEP 7 (cont.)** | | |
D.6.2.1 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. |

\[
A_{Vc} = (2(1.5c_{a1}) + s_1)1.5c_{a1}
\]

\[
A_{Vc} = (2(1.5\times 8) + 8)12 = 384 \text{ in.}^2
\]

\[
A_{Vco} = 4.5c_{a1}^2 = 4.5 \times 8^2 = 288 \text{ in.}^2
\]

\[
V_b = 7(6/0.75)^{0.2} \sqrt{0.75 \times 4000 \times 8}^{1.5}
\]

\[
V_b = 13.2 \text{ kips}
\]

\[
V_{cbg} = (384/288) \times 1.0 \times 1.0 \times 1.0 \times 13.2
\]

\[
V_{cbg} = 17.5 \text{ kips} < 27.3 \text{ kips}
\]

Nonductile

\[
\phi V_{cbg} = \phi (0.6) V_{cbg}
\]

\[
\phi V_{cbg} = 0.7(0.6)17.5 \text{ kips}
\]

\[
\phi V_{cbg} = 7.4 \text{ kips} > V_{ua} = 7.0 \text{ kips}
\]

OK

**STEP 8: Check concrete shear pryout.**

D.6.3 | For pryout, check the tension cone for the two bolts closest to the edge. Note N\(_{cbg}\) computed in Step 4. |

\[
N_{cbg} = 46.8 \text{ kips (two bolts)}
\]

\[
V_{cpg} = k_{cp}N_{cbg}
\]

\[
V_{cpg} = 2(46.8) = 93.6 \text{ kips} > 27.3 \text{ kips}
\]

OK for pryout, but because nonductile in shear, apply the 0.6 penalty.

\[
\phi V_{cpg} = \phi (0.6) V_{cpg}
\]

\[
\phi V_{cpg} = 0.7(0.6)93.6 \text{ kips}
\]

\[
\phi V_{cpg} = 39.3 \text{ kips} > V_{ua} = 7.0 \text{ kips}
\]

OK

**STEP 9: Check for tension-shear interaction.**

D.7 | 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. |

\[
N_{ua} = 7.0 \text{ kips (applied load)}
\]

\[
N_{ua} = 7.0 \text{ kips} - 3.5 \text{ kips} = 3.5 \text{ kips}
\]

(if applied load is reduced by friction)

\[
N_{ua,2} = 15.5 \text{ kips}
\]

\[
\phi V_{ua} = 15.39 \text{ kips (including friction)}
\]

\[
\phi N_n = \phi N_{ua} = 29.06 \text{ kips}
\]

\[
(N_{ua,2}/\phi N_n) + (V_{ua}/\phi V_{ua}) = (15.5/29.06) + (7/15.39) = 1.11 < 1.20
\]

OK for the case where friction is not considered.

\[
(N_{ua,2}/\phi N_n) + (V_{ua}/\phi V_{ua}) = (15.5/29.06) + (3.5/15.39) = 0.82 < 1.20
\]

OK for the case where friction is considered.
### DESIGN PROCEDURE

<table>
<thead>
<tr>
<th>Step 1</th>
<th>Design steel tensile strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Step 4</td>
<td>Design concrete tension breakout strength</td>
</tr>
<tr>
<td>Step 5</td>
<td>Design concrete tension pullout strength</td>
</tr>
<tr>
<td>Step 6</td>
<td>Concrete side-face blowout strength</td>
</tr>
<tr>
<td>Step 7</td>
<td>Design steel shear strength (includes grout and compression effects)</td>
</tr>
<tr>
<td>Step 7</td>
<td>Design steel shear strength (excludes compression effects)</td>
</tr>
<tr>
<td>Step 7</td>
<td>Design concrete shear breakout strength (includes the 0.6 penalty because concrete controls)</td>
</tr>
<tr>
<td>Step 7</td>
<td>Design concrete shear pryout strength (includes the 0.6 penalty because concrete control)</td>
</tr>
<tr>
<td>D.4.1.2</td>
<td>Design strength of anchor, shear (conservatively considering the case that excludes compression effects)</td>
</tr>
</tbody>
</table>

### CALCULATION

<table>
<thead>
<tr>
<th>Given</th>
<th>Applied load on the embed attachment:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$V_{ua} = 7$ kips</td>
</tr>
<tr>
<td></td>
<td>$e = 18$ in. (height of stub column above surface of concrete)</td>
</tr>
<tr>
<td></td>
<td>$M_u = V_{ua} \times e = 126.0$ in.-kips</td>
</tr>
<tr>
<td></td>
<td>$P_u = 3.0$ kips</td>
</tr>
<tr>
<td>Resulting applied load on critical two bolts:</td>
<td>$N_{ua,2} = 15.5$ kips</td>
</tr>
<tr>
<td>Vua</td>
<td>= 7 kips</td>
</tr>
<tr>
<td>e</td>
<td>= 18 in. (height of stub column above surface of concrete)</td>
</tr>
<tr>
<td>Mu</td>
<td>= Vua \times e = 126.0 in.-kips</td>
</tr>
<tr>
<td>Pu</td>
<td>= 3.0 kips</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step 1</th>
<th>Design steel tensile strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\phi N_{sa} = 29.06$ kips (two bolts)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step 4</th>
<th>Design concrete tension breakout strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\phi N_{cbg} = 32.76$ kips</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step 5</th>
<th>Design concrete tension pullout strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\phi nN_{pm} = 49.3$ kips</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step 6</th>
<th>Concrete side-face blowout strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\phi N_{sb} = N/A$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step 2</th>
<th>Design strength of stud in tension</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.4.1.2</td>
<td>$\phi N_n = \min(\phi N_{sa}, \phi N_{cbg}, \phi nN_{pm})$</td>
</tr>
<tr>
<td></td>
<td>= $\min(29.06, 32.76, 49.3)$</td>
</tr>
<tr>
<td></td>
<td>= 29.06 kips &gt; N_{ua,2} = 15.5 kips OK</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step 7</th>
<th>Design steel shear strength (includes grout and compression effects)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\phi V_n = 15.39$ kips</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step 7</th>
<th>Design steel shear strength (excludes compression effects)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\phi V_n = 12.09$ kips</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step 7</th>
<th>Design concrete shear breakout strength (includes the 0.6 penalty because concrete controls)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\phi V_{cbg} = 7.4$ kips</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step 7</th>
<th>Design concrete shear pryout strength (includes the 0.6 penalty because concrete control)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\phi V_{cpg} = 39.3$ kips</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>D.4.1.2</th>
<th>Design strength of anchor, shear (conservatively considering the case that excludes compression effects)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\phi V_n = \min(\phi V_n, \phi V_{cbg}, \phi V_{cpg})$</td>
</tr>
<tr>
<td></td>
<td>= $\min(12.09, 7.4, 39.3)$</td>
</tr>
<tr>
<td></td>
<td>= 7.4 kips &gt; V_{ub} = 7 kips OK</td>
</tr>
</tbody>
</table>

---


2$\phi$-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.

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
\( c_{a1} = 10 \text{ in.} \)
\( c_{a2} = 10 \text{ in.} \)
\( h_a = 7\frac{1}{2} \text{ in.} \)
\( s_1, s_2 = 6 \text{ in.} \)
\( d_o = 1\frac{1}{2} \text{ in.} \)
\( h_{ef} = 5 \text{ in.} \)

Slab reinforcement: No. 5 bars

Concrete
\( f_{c'} = 4000 \text{ psi} \)

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

Plate
\( F_y = 36 \text{ ksi} \)

Load
\( N_{ua} = 18 \text{ kips} \)

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

Assumptions:
- Concrete is cracked.
- \( \phi \)-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 \( \leq \) 1.9 \times 51.0 = 96.9 ksi).
### CODE SECTION

#### DESIGN PROCEDURE

- **D.8.1** Minimum center-to-center spacing for cast-in anchors (anchors are not torqued)
  \[ s \geq 4d_o \]
  \[ d_o = 1/2 \text{ in.} \]
  \[ 4d_o = 4 \times (1/2) = 2.0 \text{ in.} \]
  \[ s = 6 \text{ in.} > 2.0 \text{ in.} \]

- **D.8.2** Minimum edge distance for cast-in anchors
  \[ c_{a,\text{min}} = c_{a1} = c_{a2} = 10 \text{ in.} \]

- **7.7** Minimum cover for No. 5 bar and smaller
  \[ c_{a,\text{min}} = 10 \text{ in.} > 1.5 \text{ in.} \]

- **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."

- **Step 3** Minimum slab thickness: No Code requirement for cast-in anchors
  
### CALCULATION

- **STEP 1**: Check the required spacing to preclude splitting failure.
  \[ s \geq 4d_o \]

- **STEP 2**: Check for minimum edge distance to preclude splitting failure.

- **STEP 3**: Check for minimum slab thickness to preclude splitting failure.

- **STEP 4**: Summary.

### Table

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>STEP 1</strong></td>
<td>Check the required spacing to preclude splitting failure.</td>
<td></td>
</tr>
<tr>
<td>D.8.1</td>
<td>Minimum center-to-center spacing for cast-in anchors  (anchors are not torqued)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>[ s \geq 4d_o ]</td>
<td>Given</td>
</tr>
<tr>
<td></td>
<td>[ d_o = 1/2 \text{ in.} ]</td>
<td></td>
</tr>
<tr>
<td></td>
<td>[ 4d_o = 4 \times (1/2) = 2.0 \text{ in.} ]</td>
<td></td>
</tr>
<tr>
<td></td>
<td>[ s = 6 \text{ in.} &gt; 2.0 \text{ in.} ]</td>
<td>OK</td>
</tr>
<tr>
<td><strong>STEP 2</strong></td>
<td>Check for minimum edge distance to preclude splitting failure.</td>
<td></td>
</tr>
<tr>
<td>D.8.2</td>
<td>Minimum edge distance for cast-in anchors</td>
<td></td>
</tr>
<tr>
<td></td>
<td>[ c_{a,\text{min}} = c_{a1} = c_{a2} = 10 \text{ in.} ]</td>
<td></td>
</tr>
<tr>
<td>7.7</td>
<td>Minimum cover for No. 5 bar and smaller</td>
<td></td>
</tr>
<tr>
<td></td>
<td>[ c_{a,\text{min}} = 10 \text{ in.} &gt; 1.5 \text{ in.} ]</td>
<td></td>
</tr>
<tr>
<td><strong>STEP 3</strong></td>
<td>Check for minimum slab thickness to preclude splitting failure.</td>
<td></td>
</tr>
</tbody>
</table>
| 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. |
| **STEP 4**   | Summary.                                      |
| Step 1       | Minimum spacing for cast-in anchors           |
| D.8.1        | \[ s = 6 \text{ in.} \geq 4d_o = 4 \times 1/2 = 2.0 \text{ in.} \] |
| Step 2       | Minimum edge distance                         |
| D.8.2        | \[ c_{a,\text{min}} = 10 \text{ in.} \]      |
| Step 2       | Minimum cover for No. 5 bar and smaller       |
| 7.7          | \[ \text{Cover required} = 1.5 \text{ in.} < 10 \text{ in.} \] |
| Step 3       | \( h_{ef} \)                                 |
| D.8.5        | \[ h_{ef} = 5 \text{ in.} \]                 |
| Step 3       | Minimum slab thickness: No Code requirement for cast-in anchors |
| D.8.5        |                                             |
Example B4(b)—Four-stud rigid embedded plate in thin slab, tension only

Given:
Concrete edges
\( c_{a1} = c_{a2} = 10 \text{ in.} \)
\( s_1 = s_2 = 6 \text{ in.} \)
\( h_a = 7\frac{1}{2} \text{ in.} \)

Concrete
\( f'_c = 4000 \text{ psi} \)

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

Plate
3 x 3 x 5/8 in. thick
\( F_y = 36 \text{ ksi} \)

Load
\( N_{ua} = 18 \text{ kips} \)

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

Assumptions:
- Concrete is cracked.
- \( \phi \)-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: \( f_{uta} \leq 1.9f_{ya} \) (65 \( \leq \) 1.9 \( \times \) 51.0 = 96.9 ksi).
STEP 1: Determine required steel area of the anchor.

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.4.1.1</td>
<td>Equate the external factored load to the internal design strength and solve for the required steel area of the stud.</td>
<td>Equation no.</td>
</tr>
<tr>
<td>D.5.1.2</td>
<td>Assume embedment will be designed as ductile in accordance with D.3.6.1 (in Step 2). Therefore, $\phi = 0.85$ for tension.</td>
<td></td>
</tr>
<tr>
<td>D.4.5</td>
<td>Calculate nominal steel strength of selected stud.</td>
<td></td>
</tr>
<tr>
<td>D.5.1.2</td>
<td>Material properties are given. See footnote on previous page.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Anchor diameter, in.</th>
<th>Effective area $A_{se}$, in.$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4</td>
<td>0.049</td>
</tr>
<tr>
<td>3/8</td>
<td>0.110</td>
</tr>
</tbody>
</table>

STEP 2: Determine required embedment length for the anchor to prevent concrete breakout failure in tension.

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.5.2</td>
<td>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 $\phi$-factor for steel used previously.</td>
<td></td>
</tr>
<tr>
<td>D.3.6.1</td>
<td>The requirements for a ductile design are given in D.3.6.1. To prevent concrete breakout for tension load requires that $0.85N_{cbg} &gt; N_{sa}$.</td>
<td></td>
</tr>
<tr>
<td>D.5.2.1</td>
<td>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.$^2$.</td>
<td>From Step 1: $N_{sa} = 28.6$ kips</td>
</tr>
<tr>
<td>D.5.2.2</td>
<td>$k_c = 24$ for post-installed anchors</td>
<td></td>
</tr>
<tr>
<td>D.5.2.2</td>
<td>Determine required embedment length $h_{ef,req}$.</td>
<td></td>
</tr>
<tr>
<td>D.5.2.2</td>
<td>Total length of 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.</td>
<td>Use 3/8 x 5 in. long stud.</td>
</tr>
</tbody>
</table>

Use four 3/8 in. diameter studs, $A_{se} = 0.110$ in.$^2 > 0.081$ in.$^2$ (required)

<table>
<thead>
<tr>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$ (D-5)</td>
</tr>
<tr>
<td>$N_{cbg} = 33.65$ kips</td>
</tr>
<tr>
<td>$f_{y,a} = 65 \leq 1.9f_{y,a}$ or $125$ ksi</td>
</tr>
<tr>
<td>$f_{y,a} = 65 \leq 1.9f_{y,a} = 1.9 \times 51 = 96.9$ ksi OK</td>
</tr>
</tbody>
</table>

Use four 3/8 in. diameter studs, $A_{se} = 0.110$ in.$^2 > 0.081$ in.$^2$ (required)

From Step 1: $N_{sa} = 28.6$ kips

$0.85N_{cbg} = N_{sa}$

$N_{cbg} = \frac{N_{sa}}{0.85}$

$= 28.6/0.85$

$= 33.65$ kips

Concrete breakout strength for an anchor group:

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

$N_{cbg} = 33.65$ kips

$33.65 = 1.96 \times 1.0 \times 1.0 \times 1.4 \times 1.0 \times 1.52h_{ef}^{1.5}$

$h_{ef,req} = 4.02$ in.

$N_{cbg} = 33.65$ kips

$33.65 = 1.96 \times 1.0 \times 1.0 \times 1.4 \times 1.0 \times 1.52h_{ef}^{1.5}$

$h_{ef,req} = 4.02$ in.

Use 3/8 x 5 in. long stud.
**STEP 2 (cont.)**

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
</table>
| D.3.6.1      | Calculate $N_{c_{bg}}$ using $h_{ef, provided}$ | $h_{ef, provided} = 5 - 0.187 + 0.375 -$ burnoff (0.125 in.)
  $= 5.06$ in. > $4.02$ in.  OK |
| D.4.1.2      | Final check:    |             |
  a) Ductility:  | $0.85N_{c_{bg}} \geq N_{sa}$  |
  b) Strength:   | $0.85 \times 47.47 = 40.35 > 28.6$ kips  OK |

**STEP 3: Check pullout strength of anchor.**

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.5.3</td>
<td>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</td>
<td>$N_{pn} = \psi_{c,P} N_P$ (D-14)</td>
</tr>
<tr>
<td>D.3.6.1</td>
<td>Concrete is cracked per problem statement</td>
<td></td>
</tr>
</tbody>
</table>
| D.5.3.4      | Calculate pullout strength of anchor. | $4N_p = 4A_{brg} f'_c'$
  $= 4A_{brg} \times 8 \times 4$
  $= 128A_{brg}$ |
| D.5.3.5      | $\psi_{c,P} = 1.0$ for cracked concrete. |             |
|              | 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: $0.85N_{pn} \geq N_{sa}$ | $N_{pn} = 1.0 \times 128 \times 0.33$
  $= 42.4$ kips |
  $0.85N_{pn} = 0.85 \times 42.4$
  $= 36.04 > N_{sa} = 28.6$ kips |
  Therefore, ductile  OK |

**STEP 4: Check concrete side-face blowout.**

<table>
<thead>
<tr>
<th>CODE SECTION</th>
<th>DESIGN PROCEDURE</th>
<th>CALCULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.5.4</td>
<td>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.</td>
<td>N/A</td>
</tr>
</tbody>
</table>

**STEP 5: Summary.**

<table>
<thead>
<tr>
<th>Given</th>
<th>Applied load</th>
<th>$N_{sa} = 18$ kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>Step 1</td>
<td>Steel strength</td>
<td>$\phi N_{sa} = 0.8 \times 28.6 = 22.8$ kips</td>
</tr>
<tr>
<td>Step 2</td>
<td>Concrete breakout strength</td>
<td>$\phi N_{c_{bg}} = 0.75 \times 47.47 = 35.6$ kips</td>
</tr>
<tr>
<td>Step 3</td>
<td>Concrete pullout strength</td>
<td>$\phi N_{pn} = 0.75 \times 42.4 = 36.04$ kips</td>
</tr>
<tr>
<td>Step 4</td>
<td>Concrete side-face blowout strength</td>
<td>$\phi N_{sb} = N/A$</td>
</tr>
</tbody>
</table>
| D.4.1.2      | Design strength of stud | $\phi N_n = \min(\phi N_{sa}, \phi N_{c_{bg}}, \phi N_{pn})$
  $= \min(22.88, 35.6, 36.04)$
  $= 22.88$ kips $> N_{sa} = 18$ kips  OK |

**STEP 6: Check plate thickness.**

| AISC         | 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.*
### Table 1—Materials for headed and threaded anchors *

<table>
<thead>
<tr>
<th>Material</th>
<th>Grade or type</th>
<th>Diameter, in.</th>
<th>Grade</th>
<th>Tensile strength, minimum, ksi</th>
<th>Yield strength, minimum, ksi</th>
<th>Elongation, minimum, %</th>
<th>Reduction of area, minimum, %</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded studs</td>
<td>AWS D1.1:2006 ASTM A 29-05/A108-03</td>
<td>B 1010</td>
<td>1/4 to 1</td>
<td>65</td>
<td>51</td>
<td>0.2%</td>
<td>20</td>
<td>2 in.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>B 1020</td>
<td>1/4 to 1</td>
<td>65</td>
<td>51</td>
<td>0.2%</td>
<td>20</td>
<td>2 in.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Over 2-1/4 to 4</td>
<td>115</td>
<td>95</td>
<td>0.2%</td>
<td>16</td>
<td>4D</td>
<td>50 Ductile</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Over 4 to 7</td>
<td>100</td>
<td>75</td>
<td>0.2%</td>
<td>18</td>
<td>4D</td>
<td>50 Ductile</td>
</tr>
<tr>
<td>ASTM F 1554-04 (HD, T) †</td>
<td></td>
<td></td>
<td>36</td>
<td>1/4 to 4</td>
<td>58</td>
<td>36</td>
<td>0.2%</td>
<td>23</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>55</td>
<td>≤2 ‡</td>
<td>75</td>
<td>55</td>
<td>0.2%</td>
<td>21</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>105</td>
<td>1/4 to 3</td>
<td>125</td>
<td>105</td>
<td>0.2%</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td></td>
<td>B7 Over 2-1/4 to 4</td>
<td>115</td>
<td>95</td>
<td>0.2%</td>
<td>16</td>
<td>4D</td>
<td>50 Ductile</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Over 4 to 7</td>
<td>100</td>
<td>75</td>
<td>0.2%</td>
<td>18</td>
<td>4D</td>
<td>50 Ductile</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Over 1-1/4 to 3</td>
<td>105</td>
<td>81</td>
<td>0.2%</td>
<td>14</td>
<td>4D</td>
<td>35 Ductile</td>
</tr>
<tr>
<td>ASTM A 193-06a (T)</td>
<td></td>
<td></td>
<td>1/4 to 4</td>
<td>60</td>
<td></td>
<td>—</td>
<td>18</td>
<td>2 in.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Over 2-1/4 to 4</td>
<td>115</td>
<td>95</td>
<td>0.2%</td>
<td>16</td>
<td>4D</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Over 4 to 7</td>
<td>100</td>
<td>75</td>
<td>0.2%</td>
<td>18</td>
<td>4D</td>
</tr>
<tr>
<td>ASTM A 307-04 (Gr. A; HD) (Gr. C; T)</td>
<td></td>
<td></td>
<td>1/4 to 4</td>
<td>58</td>
<td>36</td>
<td>—</td>
<td>23</td>
<td>2 in.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Over 2-1/4 to 4</td>
<td>115</td>
<td>95</td>
<td>0.2%</td>
<td>16</td>
<td>4D</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Over 4 to 7</td>
<td>100</td>
<td>75</td>
<td>0.2%</td>
<td>18</td>
<td>4D</td>
</tr>
<tr>
<td>ASTM A 36-05 (T)</td>
<td></td>
<td></td>
<td>1/4 to 4</td>
<td>58</td>
<td>36</td>
<td>—</td>
<td>23</td>
<td>2 in.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Over 2-1/4 to 4</td>
<td>115</td>
<td>95</td>
<td>0.2%</td>
<td>16</td>
<td>4D</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Over 4 to 7</td>
<td>100</td>
<td>75</td>
<td>0.2%</td>
<td>18</td>
<td>4D</td>
</tr>
<tr>
<td>ASTM A 449-04b (HD, T)</td>
<td></td>
<td></td>
<td>1/4 to 1</td>
<td>120</td>
<td>92</td>
<td>0.2%</td>
<td>14</td>
<td>4D</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Over 1 to 1-1/2</td>
<td>105</td>
<td>81</td>
<td>0.2%</td>
<td>14</td>
<td>4D</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Over 1-1/4 to 3</td>
<td>90</td>
<td>58</td>
<td>0.2%</td>
<td>12</td>
<td>4D</td>
</tr>
</tbody>
</table>

*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), these 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 *

<table>
<thead>
<tr>
<th>Diameter, d0, in.</th>
<th>Threads per in., n0h</th>
<th>Effective area, A0, in.²</th>
<th>Nominal steel strength, Nsa, kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.250</td>
<td>20</td>
<td>0.049</td>
<td>3.2</td>
</tr>
<tr>
<td>0.375</td>
<td>16</td>
<td>0.110</td>
<td>7.2</td>
</tr>
<tr>
<td>0.500</td>
<td>13</td>
<td>0.196</td>
<td>12.8</td>
</tr>
<tr>
<td>0.625</td>
<td>11</td>
<td>0.307</td>
<td>19.9</td>
</tr>
<tr>
<td>0.750</td>
<td>10</td>
<td>0.442</td>
<td>28.7</td>
</tr>
<tr>
<td>0.875</td>
<td>9</td>
<td>0.601</td>
<td>39.1</td>
</tr>
<tr>
<td>1.000</td>
<td>8</td>
<td>0.785</td>
<td>51.1</td>
</tr>
<tr>
<td>1.125</td>
<td>7</td>
<td>0.994</td>
<td>64.6</td>
</tr>
<tr>
<td>1.250</td>
<td>7</td>
<td>1.227</td>
<td>79.8</td>
</tr>
<tr>
<td>1.375</td>
<td>6</td>
<td>1.485</td>
<td>96.5</td>
</tr>
<tr>
<td>1.500</td>
<td>6</td>
<td>1.767</td>
<td>114.9</td>
</tr>
<tr>
<td>1.750</td>
<td>5</td>
<td>2.405</td>
<td>156.3</td>
</tr>
<tr>
<td>2.000</td>
<td>4.5</td>
<td>3.142</td>
<td>204.2</td>
</tr>
</tbody>
</table>

*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 AD for studs.

§Use tensile stress area Ase for threaded anchors (Note: Ase is the same as tensile stress area in AISC). Ase = 0.7854 × (d0 - (0.9743/n0h))².

Nsa = AD × futa; futa = 65 ksi; see Table 1 for other materials.

Nusa = AD × futsb; futsb = 58 ksi; see Table 1 for other materials.

Note: For A108, and futsb = 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)

<table>
<thead>
<tr>
<th>Nominal anchor diameter, d₀, in.</th>
<th>Gross area of bolt, A_D, in.²</th>
<th>Tensile area, A_{s_e}, in.²</th>
<th>N_{sa} = A_{s_e} × f_{u,sa}</th>
<th>Required embedment depth h_{ef} for ductile behavior, in.*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.250</td>
<td>0.049</td>
<td>0.032</td>
<td>1.8</td>
<td>1.4</td>
</tr>
<tr>
<td>0.375</td>
<td>0.110</td>
<td>0.078</td>
<td>4.5</td>
<td>2.5</td>
</tr>
<tr>
<td>0.500</td>
<td>0.196</td>
<td>0.142</td>
<td>8.2</td>
<td>3.8</td>
</tr>
<tr>
<td>0.625</td>
<td>0.307</td>
<td>0.226</td>
<td>13.1</td>
<td>5.2</td>
</tr>
<tr>
<td>0.750</td>
<td>0.442</td>
<td>0.334</td>
<td>19.4</td>
<td>6.7</td>
</tr>
<tr>
<td>0.875</td>
<td>0.601</td>
<td>0.462</td>
<td>26.8</td>
<td>8.3</td>
</tr>
<tr>
<td>1.000</td>
<td>0.785</td>
<td>0.606</td>
<td>35.1</td>
<td>10.0</td>
</tr>
<tr>
<td>1.125</td>
<td>0.994</td>
<td>0.763</td>
<td>44.3</td>
<td>11.6</td>
</tr>
<tr>
<td>1.250</td>
<td>1.227</td>
<td>0.969</td>
<td>56.2</td>
<td>13.6</td>
</tr>
<tr>
<td>1.375</td>
<td>1.485</td>
<td>1.16</td>
<td>67.3</td>
<td>15.4</td>
</tr>
<tr>
<td>1.500</td>
<td>1.767</td>
<td>1.41</td>
<td>81.5</td>
<td>17.5</td>
</tr>
<tr>
<td>1.750</td>
<td>2.405</td>
<td>1.90</td>
<td>110.2</td>
<td>21.3</td>
</tr>
<tr>
<td>2.000</td>
<td>3.142</td>
<td>2.50</td>
<td>144.9</td>
<td>25.6</td>
</tr>
</tbody>
</table>

\[ h_{ef} = \left( \frac{N_{sa}}{0.85 \times 24 \times f_{c}^{0.5}} \right)^{2/3}. \]

\[ A_{s_e} \text{ taken from Table 2.} \]

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)

<table>
<thead>
<tr>
<th>Nominal anchor diameter, d₀, in.</th>
<th>Gross area of bolt, A_D, in.²</th>
<th>Tensile area, A_{s_e}, in.²</th>
<th>N_{sa} = A_{s_e} × f_{u,sa}</th>
<th>Required embedment depth h_{ef} for ductile behavior, in.*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.250</td>
<td>0.049</td>
<td>0.032</td>
<td>3.3</td>
<td>2.1</td>
</tr>
<tr>
<td>0.375</td>
<td>0.110</td>
<td>0.078</td>
<td>8.2</td>
<td>3.8</td>
</tr>
<tr>
<td>0.500</td>
<td>0.196</td>
<td>0.142</td>
<td>14.9</td>
<td>5.6</td>
</tr>
<tr>
<td>0.625</td>
<td>0.307</td>
<td>0.226</td>
<td>23.7</td>
<td>7.7</td>
</tr>
<tr>
<td>0.750</td>
<td>0.442</td>
<td>0.334</td>
<td>35.1</td>
<td>10.0</td>
</tr>
<tr>
<td>0.875</td>
<td>0.601</td>
<td>0.462</td>
<td>48.5</td>
<td>12.3</td>
</tr>
<tr>
<td>1.000</td>
<td>0.785</td>
<td>0.606</td>
<td>63.6</td>
<td>14.8</td>
</tr>
<tr>
<td>1.125</td>
<td>0.994</td>
<td>0.763</td>
<td>80.1</td>
<td>17.3</td>
</tr>
<tr>
<td>1.250</td>
<td>1.227</td>
<td>0.969</td>
<td>101.8</td>
<td>20.2</td>
</tr>
<tr>
<td>1.375</td>
<td>1.485</td>
<td>1.16</td>
<td>121.8</td>
<td>22.8</td>
</tr>
<tr>
<td>1.500</td>
<td>1.767</td>
<td>1.41</td>
<td>147.6</td>
<td>25.9</td>
</tr>
<tr>
<td>1.750</td>
<td>2.405</td>
<td>1.90</td>
<td>199.4</td>
<td>31.7</td>
</tr>
<tr>
<td>2.000</td>
<td>3.142</td>
<td>2.50</td>
<td>262.3</td>
<td>38.1</td>
</tr>
</tbody>
</table>

\[ h_{ef} = \left( \frac{N_{sa}}{0.85 \times 24 \times f_{c}^{0.5}} \right)^{2/3}. \]

\[ A_{s_e} \text{ taken from Table 2.} \]

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

<table>
<thead>
<tr>
<th>Nominal anchor diameter, d₀, in.</th>
<th>Gross area of bolt, A_D, in.²</th>
<th>Tensile area, A_{s_e}, in.²</th>
<th>N_{sa} = A_{s_e} × f_{u,sa}</th>
<th>Required embedment depth h_{ef} for ductile behavior, in.*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.250</td>
<td>0.049</td>
<td>0.032</td>
<td>3.2</td>
<td>2.0</td>
</tr>
<tr>
<td>0.375</td>
<td>0.110</td>
<td>0.078</td>
<td>7.2</td>
<td>3.5</td>
</tr>
<tr>
<td>0.500</td>
<td>0.196</td>
<td>0.142</td>
<td>12.8</td>
<td>5.1</td>
</tr>
<tr>
<td>0.625</td>
<td>0.307</td>
<td>0.226</td>
<td>19.9</td>
<td>6.8</td>
</tr>
<tr>
<td>0.750</td>
<td>0.442</td>
<td>0.334</td>
<td>28.7</td>
<td>8.7</td>
</tr>
<tr>
<td>0.875</td>
<td>0.601</td>
<td>0.462</td>
<td>39.1</td>
<td>10.7</td>
</tr>
<tr>
<td>1.000</td>
<td>0.785</td>
<td>0.606</td>
<td>51.1</td>
<td>12.8</td>
</tr>
<tr>
<td>1.125</td>
<td>0.994</td>
<td>0.763</td>
<td>64.6</td>
<td>15.0</td>
</tr>
<tr>
<td>1.250</td>
<td>1.227</td>
<td>0.969</td>
<td>79.8</td>
<td>17.2</td>
</tr>
<tr>
<td>1.375</td>
<td>1.485</td>
<td>1.16</td>
<td>96.5</td>
<td>19.5</td>
</tr>
<tr>
<td>1.500</td>
<td>1.767</td>
<td>1.41</td>
<td>114.9</td>
<td>21.9</td>
</tr>
<tr>
<td>1.750</td>
<td>2.405</td>
<td>1.90</td>
<td>156.3</td>
<td>27.0</td>
</tr>
<tr>
<td>2.000</td>
<td>3.142</td>
<td>2.50</td>
<td>204.2</td>
<td>32.2</td>
</tr>
</tbody>
</table>

\[ h_{ef} = \left( \frac{N_{sa}}{0.85 \times 24 \times f_{c}^{0.5}} \right)^{2/3}. \]

\[ A_{s_e} \text{ taken from Table 2.} \]
### Table 4(a)—Anchor head and nut (square head) dimensions *

<table>
<thead>
<tr>
<th>Nominal anchor diameter ( d_o ), in.</th>
<th>Gross, area of bolt, ( A_{Dr} ) in.</th>
<th>Tensile area, ( A_{se} ) in.</th>
<th>Width ( F ), in.</th>
<th>Width ( C ), in.</th>
<th>Height ( H ), in.</th>
<th>Gross area of head ( A_{H} ) based on width ( F ), in.</th>
<th>Net bearing area ( A_{brg} ) in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.250</td>
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<td>0.032</td>
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<td>1.563</td>
<td>0.500</td>
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*Dimensions taken from AISC Steel Design Manual.

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

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<tr>
<th>Nominal anchor diameter ( d_o ), in.</th>
<th>Gross, area of bolt, ( A_{Dr} ) in.</th>
<th>Tensile area, ( A_{se} ) in.</th>
<th>Width ( F ), in.</th>
<th>Width ( C ), in.</th>
<th>Height ( H ), in.</th>
<th>Gross area of head ( A_{H} ) based on width ( F ), in.</th>
<th>Net bearing area ( A_{brg} ) in.</th>
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<tbody>
<tr>
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</tbody>
</table>

*Dimensions taken from AISC Steel Design Manual.

**See Table 2 for definition of \( A_{se} \).

\( A_{H} = F^2 \) or \( A_{H} = 1.5F^2\tan 30\circ \).

\( A_{brg} = A_{H} - A_{Dr} \)
### Table 4(c)—Anchor head and nut (heavy hex) dimensions

<table>
<thead>
<tr>
<th>Nominal anchor diameter d₀, in.</th>
<th>Anchor areas</th>
<th>Gross, area of bolt, ( A_D ), ( \text{in.}^2 )</th>
<th>Tensile area, ( A_{se} ), ( \text{in.}^2 )</th>
<th>Width ( F ), in.</th>
<th>Width ( C ), in.</th>
<th>Height ( H ), in.</th>
<th>Gross area of head ( A_H ),(^\ddagger ) based on width ( F ), in.(^2 )</th>
<th>Net bearing area ( A_{bg} ),(^\S ) in.(^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.250</td>
<td>0.049</td>
<td>0.032</td>
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</table>

\(^{\ddagger}\)Dimensions taken from AISC Steel Design Manual.

\(^{\S}\)See Table 2 for definition of \( A_{se} \).

\(^{\ddagger}\)\( A_H = A_D \) or \( A_H = 1.5F\tan 30^\circ \).

\(^{\S}\)\( A_{bg} = A_H - A_D \).
### Table 5—Hardened washer dimensions

<table>
<thead>
<tr>
<th>Nominal anchor diameter $d_o$, in.</th>
<th>Anchor areas</th>
<th>Washer dimensions</th>
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<tr>
<td></td>
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<td>Gross, area of bolt, $A_D$, in.$^2$</td>
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<tr>
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<td>0.334</td>
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<td>0.462</td>
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**SAE dimensions**

<table>
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<th>Anchor areas</th>
<th>Washer dimensions</th>
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<tr>
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<td>Gross, area of bolt, $A_D$, in.$^2$</td>
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<td>1.125</td>
<td>0.994</td>
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</tr>
<tr>
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</tr>
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</tr>
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<td>2.50</td>
</tr>
</tbody>
</table>

**U.S. Standard dimensions**

<table>
<thead>
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<th>Nominal anchor diameter $d_o$, in.</th>
<th>Anchor areas</th>
<th>Washer dimensions</th>
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</thead>
<tbody>
<tr>
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<td></td>
<td>Gross, area of bolt, $A_D$, in.$^2$</td>
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<tr>
<td>0.250</td>
<td>0.125</td>
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<tr>
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</tr>
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<td>0.312</td>
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<tr>
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<td>0.875</td>
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<td>0.250</td>
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</tbody>
</table>

*Hardened washers to SAE dimensions.

†Material ASTM F 436.

‡Refer to Table 2 for definition of $A_{tens}$.

§$A_H = \pi (OD)^2/4$.

$A_{brg} = A_H - A_D$.

### Table 6—Stud dimensions

**(Steel: ASTM A 108; $f_{ut} = 65$ ksi; $f_y = 51$ ksi)**

<table>
<thead>
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<th>Diameter $d_o$, in.</th>
<th>Stud dimensions</th>
<th>Areas</th>
</tr>
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<td>$H$, in.</td>
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<td>0.187</td>
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<tr>
<td>0.375</td>
<td>0.125</td>
<td>0.281</td>
</tr>
<tr>
<td>0.500</td>
<td>0.125</td>
<td>0.312</td>
</tr>
<tr>
<td>0.625</td>
<td>0.187</td>
<td>0.312</td>
</tr>
<tr>
<td>0.750</td>
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<td>0.375</td>
</tr>
<tr>
<td>0.875</td>
<td>0.187</td>
<td>0.375</td>
</tr>
<tr>
<td>1.000</td>
<td>0.250</td>
<td>0.500</td>
</tr>
</tbody>
</table>

*Taken from Nelson Stud catalog.

$A_{stud} = \pi (d_o)^2/4$.

$A_{brg} = \pi (H_d)^2/4$. 

$A_{brg} = \pi (H_d - d_o)^2/4$. 

[Diagram of a stud and washer]
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 \(1.4N_p\) (where \(N_p\) 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 normal weight concrete only.

D.3.5 The values of \(f_c'\) 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 \(f_c'\) 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 \(0.60\phi N_n\) and \(0.60\phi V_n\), where \(\phi\) is given in D.4.4 or D.4.5, and \(N_n\) and \(V_n\) 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 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,

\[\phi N_n \geq N_{ua}\]  \hspace{1cm} (D-1)
\[ \phi V_n \geq V_{ua} \]  

(D-2)

D.4.1.2 In Eq. (D-1) and (D-2), \( \phi N_n \) and \( \phi V_n \) are the lowest design strengths determined from all appropriate failure modes. \( \phi N_n \) is the lowest design strength in tension of an anchor or group of anchors as determined from consideration of \( \phi N_{sa}, \phi N_{sb}, \) either \( \phi N_{eb} \) or \( \phi N_{ebg}, \) and either \( \phi N_{cb} \) or \( \phi N_{cbg}. \) \( \phi V_n \) is the lowest design strength in shear of an anchor or a group of anchors as determined from consideration of: \( \phi V_{sa}, \) either \( \phi V_{eb} \) or \( \phi V_{ebg}, \) and either \( \phi V_{cp} \) or \( \phi V_{cpg}. \)

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 \( \phi \) 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 .................................................. 0.70
   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

<table>
<thead>
<tr>
<th>Condition A</th>
<th>Condition B</th>
</tr>
</thead>
<tbody>
<tr>
<td>i) Shear loads</td>
<td>0.75</td>
</tr>
<tr>
<td>ii) Tension loads</td>
<td>Cast-in headed studs</td>
</tr>
<tr>
<td></td>
<td>or headed bolts</td>
</tr>
<tr>
<td></td>
<td>Post-installed</td>
</tr>
</tbody>
</table>

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 \( \phi \) 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

<table>
<thead>
<tr>
<th>Condition</th>
<th>Nominal strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>i) Shear loads</td>
<td>0.85</td>
</tr>
<tr>
<td>ii) Tension loads</td>
<td>Cast-in headed studs</td>
</tr>
<tr>
<td></td>
<td>or headed bolts</td>
</tr>
<tr>
<td></td>
<td>Post-installed</td>
</tr>
</tbody>
</table>

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 \( \phi \) 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

\[
N_{sa} = nA_{se}f_{uta} \quad \text{(D-3)}
\]

where \( n \) is the number of anchors in the group, and \( f_{uta} \) shall not be taken greater than the smaller of \( 1.9f_{yu} \) 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

\[
N_{cb} = \frac{A_{Ne}}{A_{Nco}}\psi_{c,N}\psi_{c,N}\psi_{cp,N}N_b \quad \text{(D-4)}
\]

(b) for a group of anchors

\[
N_{cbg} = \frac{A_{Ne}}{A_{Nco}}\psi_{c,N}\psi_{c,N}\psi_{cp,N}N_b \quad \text{(D-5)}
\]

Factors \( \psi_{c,N} \), \( \psi_{ed,N} \), 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. \( A_{Ne} \) 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. \( A_{Nco} \) 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 \( 1.5h_{ef} \)

\[
A_{Nco} = 9h_{ef}^2 \quad \text{(D-6)}
\]

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

\[
N_b = k_c\sqrt{f'}c h_{ef}^{5/3} \quad \text{(D-8)}
\]

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

\[
\psi_{ed,N} = \frac{1}{1 + \frac{2e_N}{3h_{ef}}} \leq 1.0 \quad \text{(D-9)}
\]

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 \( \psi_{ed,N} \) shall be computed for each axis individually and the product of these factors used as \( \psi_{ed,N} \) in Eq. (D-5).

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

\[
\psi_{ed,N} = 1 \text{ if } c_{a,min} \geq 1.5h_{ef} \quad \text{(D-10)}
\]

\[
\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \text{ if } c_{a,min} < 1.5h_{ef} \quad \text{(D-11)}
\]

D.5.2.6 For anchors located in a region of a concrete member where analysis indicates no cracking \( (f_2 < f_r) \) under the load combinations specified in 9.2 or C.2 with load factors taken as unity, the following modification factor shall be permitted:

\[
\psi_{c,N} = 1.25 \text{ for cast-in anchors; and } \psi_{c,N} = 1.4 \text{ for post-installed anchors, where the value of } k_c \text{ used in Eq. (D-7) is 17.}
\]

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

\[
\psi_{cp,N} = 1.0 \text{ if } c_{a,min} \geq c_{ac} \quad \text{(D-12)}
\]

\[
\psi_{cp,N} = \frac{c_{a,min}}{c_{ac}} \text{ if } c_{a,min} < c_{ac} \quad \text{(D-13)}
\]

where the critical distance \( c_{ac} \) is defined in D.8.6.
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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 1.5$h_{ef}$ 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

$$N_{pn} = \psi_{c,P}N_p$$

(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

$$N_p = 8A_{brg}f'_c$$

(D-15)

D.5.3.5 For an anchor located in a region of a concrete member where analysis indicates no cracking ($f_t < f_c$) 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 ($e_{a1} < 0.4h_{ef}$), the nominal side-face blowout strength, $N_{sb}$, shall not exceed

$$N_{sb} = 160c_{a1}\sqrt{A_{brg}}f'_c$$

(D-16)

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

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

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

(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

$$V_{sa} = nA_{se}f_{uta}$$

(D-18)

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

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

$$V_{sa} = n0.6A_{se}f_{uta}$$

(D-19)

where $n$ is the number of anchors in the group and $f_{uta}$ shall not be taken greater than the smaller of 1.9$f_{ya}$ 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 0.40$C_F$.

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

$$V_{cb} = \frac{A_{Ve}}{A_{Ve0}} \psi_{ed, V} \psi_{c, V} \psi_{V} V_{b}$$  \hspace{1cm} (D-20)

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

$$V_{cbg} = \frac{A_{Ve}}{A_{Ve0}} \psi_{ec} \psi_{ed, V} \psi_{c, V} \psi_{V} V_{b}$$  \hspace{1cm} (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 $\psi_{wed, V}$ 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 $\psi_{ec}, \psi_{ed, V}$, 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. $A_{Ve}$ 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 $A_{Ve}$ 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. $A_{Ve0}$, where $n$ is the number of anchors in the group.

$A_{Ve0}$ 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 $A_{Ve0}$ as the base of a half-pyramid with a side length parallel to the edge of $3c_{a1}$ and a depth of $1.5c_{a1}$

$$A_{Ve0} = 4.5(c_{a1})^2$$  \hspace{1cm} (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

$$V_{b} = 7\left(\frac{\ell_e}{d_o}\right)^{0.2} d_o^{1.5} \sqrt[1.5]{(c_{a1})^{1.5}}$$  \hspace{1cm} (D-23)

where $\ell_e$ 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 $\ell_e$ 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

$$V_{b} = 8\left(\frac{\ell_e}{d_o}\right)^{0.2} d_o^{1.5} \sqrt[1.5]{(c_{a1})^{1.5}}$$  \hspace{1cm} (D-24)

where $\ell_e$ 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 $c_{a2} \leq 1.5h_{ef}$.

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

$$\psi_{ec, V} = \frac{1}{\left(1 + \frac{2c_{a1}}{3c_{a1}}\right)} \leq 1$$  \hspace{1cm} (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

$$\psi_{ed, V} = 1.0 \text{ if } c_{a2} \geq 1.5c_{a1}$$  \hspace{1cm} (D-26)

$$\psi_{ed, V} = 0.7 + 0.3 \frac{c_{a2}}{1.5c_{a1}} \text{ if } c_{a2} < 1.5c_{a1}$$  \hspace{1cm} (D-27)

D.6.2.7 For anchors located in a region of a concrete member where analysis indicates no cracking ($f_y < f_i$) 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 \( V_{cp,g} \), shall not exceed

(a) for a single anchor

\[ V_{cp} = k_{cp}N_{eb} \]  \( (D-28) \)

(b) for a group of anchors

\[ V_{cp,g} = k_{cp}N_{egb} \]  \( (D-29) \)

where

\[ k_{cp} = 1.0 \] for \( h_{ef} < 2.5 \) in.; and

\[ k_{cp} = 2.0 \] for \( h_{ef} \geq 2.5 \) in.

\( N_{eb} \) and \( N_{egb} \) 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 \( \phi N_n \) shall be as required in D.4.1.2. The value of \( \phi V_n \) 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

\[ \frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2 \]  \( (D-30) \)

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 \( \phi \) 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 embeddings 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 \( 4\phi \int f^c_0 \) 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 \( \phi \) 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 embeddings with embedded base plates—For embeddings 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: 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 embeddings

D.12.1 Grouted embeddings 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 embeddings shall be tested to verify embedment strength. Grouted embeddings 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 embeddings shall be tested for the installed condition by testing randomly selected grouted embeddings 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

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 \( K_{00} \) associated with the 5% fractile, \( S - K_{00} S_x \), depends on the number of tests, \( n \), used to compute \( S \) and \( S_x \). Values of \( K_{00} \) 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 embeddings, 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, B18.2.1, D.1 B18.2.1, D.2

![Fig. RD.1—Types of fasteners.](image-url)
and A18.2.6 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. RD2.6(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 heavy-weight 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 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 $f_{yu}$, 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 and A 490 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 $\phi$-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-
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 \( \kappa \) 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 \( \kappa \) method. It is consistent with a breakout prism angle of approximately 35 degrees (Fig. RD.4.2.2(a) and (b)).

**RD.4.4** The \( \phi \)-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 \( \phi \)-factors for use with \( f_{ut} \) appear low, they result in a level of safety consistent with the use of higher \( \phi \)-factors applied to \( f_y \). The smaller \( \phi \)-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, \( \phi = 0.70 \).

**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 \( \phi \)-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 \( \phi \)-factors are presented in D.4.5 for use with the load factors of 2006 Appendix C. The \( \phi \)-factors for use with the load factors of the ACI 318-99, Appendix C were determined in a manner consistent with the other \( \phi \)-factors of the ACI 318-99 Appendix C. These \( \phi \)-factors are presented in D.4.4 for use with the load factors of 2006 Section 9.2. Since investigative studies for \( \phi \)-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 \( \phi \)-factors appears in this section.

Even though the \( \phi \)-factor for plain concrete in Appendix C uses a value of 0.65, the basic factor for brittle concrete failures (\( \phi = 0.75 \)) was chosen based on results of probabilistic studies\(^{D.18} \) that indicated the use of \( \phi = 0.65 \) 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 \( \phi = 0.65 \) value would be overly conservative. Comparison with other design procedures and probabilistic studies\(^{D.18} \) indicated that the choice of \( \phi = 0.75 \)
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was justified. For applications with supplementary reinforcement and more ductile failures (Condition A), the 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 $1.3f_y$ 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 $A_{s_{f_{y,\text{b}}}}$ rather than $A_{s_{f_{y,a}}}$ 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 $A_{s_{f_{y,\text{b}}}}$ since the 1986 edition of their specifications. The use of Eq. (D-3) with Section 9.2 load factors and the $\phi$-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 $f_{y,\text{b}}$ is to ensure that, under service load conditions, the anchor does not exceed $f_{y,a}$. The limit on $f_{y,\text{b}}$ of $1.9f_y$ was determined by converting the LRFD provisions to corresponding service load conditions. For Section 9.2, the average load factor of 1.4 (from 1.2D + 1.6L) divided by the highest $\phi$-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 1.4D + 1.7L), divided by the highest $\phi$-factor (0.80 for tension), results in a limit $f_{ut}/f_y$ of 1.55/0.8 = 1.94. For consistent results, the service-ability limitation of $f_{y,\text{b}}$ was taken as $1.9f_y$. If the ratio of $f_{y,\text{b}}$ to $f_{y,a}$ exceeds this value, the anchoring may be subjected to service loads above $f_{y,a}$. Although not a concern for standard structural steel anchors (maximum value of $f_{ut}f_{y,a}$ 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 $A_{N_c}/A_{N_{co}}$ and $\psi_{ed,N}$ in Eq. (D-4) and (D-5).

Figure RD.5.2.1(a) shows $A_{N_{co}}$ and the development of Eq. (D-6). $A_{N_c}$ 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 $A_{N_c}$ is the total projected area for a group of anchors, and $A_{N_{co}}$ 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 $A_{N_c}$ 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 $k_e$ 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 $k_{e,h}$ values for cracked concrete.(D.13,D.24) Higher $k_{e,h}$ 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. ≤ $h_{ef} ≤ 25$ 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 $1.5h_{ef}$ 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 $A_{N_c}/A_{N_{co}}$ 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 $h_{ef} = 1.5 h_{ef}$, 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 $h_{ef} = c_{a,max}/1.5$, results in $h_{ef} = h_{ef}^{5/3} = 4$ 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
The critical edge distance for headed studs, headed bolts, expansion anchors, and undercut anchors is 1.5\( h_{df} \).

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

\[
A_{Nco} = 2(1.5\ h_{df}) \cdot 2(1.5\ h_{df}) = 9h_{df}^2
\]

\( \text{(a)} \)

\[
A_{Nc1} = (c_{ao} + 1.5h_{df})(2 \times 1.5h_{df})
\]
if \( c_{ao} < 1.5h_{df} \)

\( \text{(b)} \)

\[
A_{Nc2} = (c_{ao} + s_j + 1.5h_{df})(2 \times 1.5h_{df})
\]
if \( c_{ao} < 1.5h_{df} \) and \( s_j < 3h_{df} \)

The actual failure surface for limiting \( h_{fu} \) shows the intersection of the assumed failure surface for limiting \( h_{fu} \) with the concrete surface.

**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 \( A_{Ne}/A_{Nco} \). 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.

**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, \( \psi_{c,N} = 1 \)) with \( \psi_{ed,N} k_e = 24 \) 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 \( \psi_{ed,N} k_e \) 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 \( k_e \) 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 \( k_e \) 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 \( k_e \) 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 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 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.
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. 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 < 0.4d_y). 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.”

The limitation of 1.9f_{ya} 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.

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 ψ_{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 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.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 $A_{Vco}$; (b) Projected area for single anchors and groups of anchors and calculation of $A_{Vc}$; (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...
increase with the failure surface, which is proportional to 
$$(c_1)^2.$$ Instead, the capacity increases proportionally to 
$$(c_1)^{1.5}$$ 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 deter-
mined from test data reported in Reference D.12 at the 5%
fractile adjusted for cracking.

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 
this, the basic shear strength value for such anchors is increased. 
Limits are imposed to ensure sufficient rigidity. The design 
of supplementary reinforcement is discussed in Refer-
ences D.11 , D.14, and D.15.

For anchors influenced by three or more edges 
where any edge distance is less than 1.5$c_1$, 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 $\kappa$ methodD.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 $c_{a2}/1.5$, $h_a/1.5$, 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 $c_{a1}$ is the largest of $c_{a2}/1.5$ 
in each direction, $h_a/1.5$, and $1/3$ of the maximum spacing 
between anchors for anchor groups. For the example shown in Fig. 
RD.6.2.4, Point A shows the controlling intersection.

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 $e'_s$ for 
calculating the $\psi_{e_s}V$ 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

$$\left(\frac{N_{ua}}{N_n}\right)^{\frac{5}{3}} + \left(\frac{V_{ua}}{V_n}\right)^{\frac{5}{3}} \leq 1.0$$

where $\zeta$ varies from 1 to 2. The current trilinear recommendation is a simplification of the expression where $\zeta = \frac{5}{3}$ (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.

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 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 $\phi$-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 $\phi$-values in the AISC-LRFD Steel Manual. The $\phi$-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 $\phi = 0.90$ and $F_v = 0.6F_c$. For these elements, the same $\phi$-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...
than distance approach considers the effect of the tension anchors and positive for tension and negative for compression. This external loads acting across the initial shear fracture planes of the Code for concrete-to-concrete friction, and is given in Reference D.21, is as follows:

\[ K_c = 1.6 \]

data. The different \( K_c \) 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 \( N_{sa} \), 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.

**References**


D.6. Lotze, D., and Klingner, R.E., “Behavior of Multiple-Anchor Attachments to Concrete from the Perspective of


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