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Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D

Reported by ACI Committee 355

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American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
U.S.A.
Phone: 248-848-3700
Fax: 248-848-3701

www.concrete.org

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Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D

Reported by ACI Committee 355

Donald F. Meinheit[*]
Chair

J. Bret Turley[*]
Secretary

Tarek S. Aziz	Werner A. F. Fuchs	Richard E. Klingner[*]	Jake Olsen
Ranjit L. Bandyopadhyay[*†]	Branko Galunic	Anthony J. Lamanna	Alan D. Price
Peter J. Carrato[*]	Brian C. Gerber	Harry B. Lancelot III	John F. Silva[*]
Harry A. Chambers	Michael Gong	Nam-Ho Lee[*]	Patrick J. E. Sullivan
Ronald A. Cook[*]	Herman L. Graves III	Lee W. Mattis	Harry Wiewel
Rolf Eligehausen	Christopher Heinz[*]	Robert R. McGlohn[*]	Richard E. Wollmershauser
Sam S. Eskildsen[*]	Bruce I. Ireland		

[*]Committee 355 members who were major contributors to the development of this guide.
[†]Deceased.

This guide presents worked examples using the design provisions in ACI 318 Appendix D. Not all conditions are covered in these examples. The essentials of direct tension, direct shear, combined tension and shear, and the common situation of eccentric shear, as in a bracket or corbel, are presented.

Keywords: anchorage; combined tension and shear; design examples; eccentric shear; embedded bolts; headed-stud anchors; post-installed anchors; shear; tension.

CONTENTS

ACI 355.3R-11 was adopted and published March 2011.

CHAPTER 1—INTRODUCTION

1.1—Introduction

This guide was prepared by the members of ACI 355, Anchorage to Concrete, to provide design examples that demonstrate the provisions of ACI 318-05 Appendix D. Appendix D, which was first introduced in ACI 318-02, contains design provisions for determining the strength of anchors based on the Concrete Capacity Design (CCD) method for concrete breakout failure. The CCD method has its origins in research work done at the University of Stuttgart in Germany (Eligehausen et al. 1987; Eligehausen and Fuchs 1988; Rehm et al. 1992) and was formalized at the University of Texas at Austin in the 1990s (Fuchs et al. 1995). The CCD method calculates the concrete breakout strength using a model that is based on a breakout prism having an angle of approximately 35 degrees, rather than the traditional 45-degree cone model used since the early 1970s.

Appendix D design provisions are for both cast-in-place anchors and prequalified post-installed mechanical anchors. Separate design equations are frequently provided because cast-in-place anchors behave differently than post-installed anchors. The provisions for post-installed anchors are only intended for those post-installed anchors that are qualified under comprehensive testing protocols. The testing and evaluation requirements in ACI 355.2 are the standard for qualifying post-installed anchors used in design with Appendix D. Similar procedures, which are expected to be completed soon, are under development for adhesive anchors and concrete screw anchors.

Table 1.1—List of anchor failure modes

Tension failure mode	Shear failure mode
Steel strength of anchor	Steel strength of anchor
Concrete breakout strength	Concrete breakout strength
Concrete side-face blowout strength	Concrete pryout strength
Pullout and pull-through strength	

1.2—Discussion on design example problems

The example problems presented in this guide were developed using the code provisions in Appendix D of ACI 318-05, which were current at the time the examples were developed. The new provisions of ACI 318-08 will alter the calculations and results in these examples. Commentary in this guide describes how the new ACI 318-08 provisions modify the design results. The ACI 318-08 Appendix D provisions clarify issues when dealing with earthquake forces, ductile failure, anchor reinforcement, and supplemental reinforcement.

The design approach used in the example problems follows a basic outline of evaluating each potential failure mode in tension and shear for the anchor using the provisions of Appendix D of ACI 318-05. The provisions include modification factors that account for the effects of edges, eccentricity, and the presence or lack of cracking in the concrete, to determine the nominal strengths for each failure mode. The types of failure modes considered are shown in Table 1.1.

In addition to the failure modes in Table 1.1, minimum edge distance, anchor spacing, and thickness of the concrete member are checked to preclude the splitting of concrete. The calculated nominal strengths for each failure mode are modified by the appropriate modification factors. The minimum calculated design strength becomes the controlling design strength of the anchor or group of anchors.

1.3—Commentary on seismic requirements for Appendix D of ACI 318-02 and ACI 318-05

ACI 318-02 and ACI 318-05 use the terminology "low," "moderate," and "high" to describe the levels of seismic risk. The design strength of anchors that include earthquake forces and that are located in regions of moderate or high seismic risk are required to be controlled by failure in tension, shear, or both, of a ductile steel element. In addition, the design strengths for steel and concrete are reduced by a factor of 0.75. The nonductile, concrete failure modes include all the concrete breakout modes in tension and shear, plus the pullout and pull-through failure modes in tension. Nonductile failure can occur if the steel behaves in a brittle fashion. It is not always possible, due to geometric or material constraints, to design the anchorage for a ductile failure. Therefore, code provisions allow the attachment, which the anchor connects to the structure, to be considered as the ductile steel element.

Design Examples 1, 2, 11, and 12 demonstrate the provisions of Appendix D when earthquake forces are involved. They show the design of the anchors governed by the steel strength of a ductile steel element, according to Section D.3.3.4 of

Table 1.2—Description of design example problems

Design example problem	Description of problem
Example 1—Single headed anchor in tension away from edges	A single cast-in anchor under tension loading unaffected by edges and located in a high seismic region.
Example 2—Single hooked anchor in tension away from edges	Same problem as Example 1 but the anchor is an L-bolt with similar geometry; the intent is to show the inherent lower capacity of the L-bolt compared with the headed bolt.
Example 3—Single post-installed anchor in tension away from edges	Determines the optimum post-installed anchor embedment and diameter to support a tension load using anchor qualification testing data.
Example 4—Group of headed studs in tension near an edge	A group of headed anchors welded to plate supporting a tension load near free edge of concrete.
Example 5—Single headed bolt in shear near an edge	Determines the service wind load that can be applied to a single cast-in anchor near a free edge by calculating the design shear strength with reductions due to edge effects.
Example 6—Single headed bolt in tension and shear near an edge	The same anchor geometry of Example 5 is subjected a reversible shear load and a tension load. Edge effects and tension/shear interaction are evaluated.
Example 7—Single post-installed anchor in tension and shear near an edge	A post-installed anchor near a free edge of concrete is subjected to shear and tension using anchor qualification testing data. Edge effects and tension/shear interaction are evaluated.
Example 8—Group of cast-in anchors in tension and shear with two free edges and supplemental reinforcement	A column base plate with oversize holes is anchored with a group of cast-in anchors. Tension and shear loads are applied. Edge effects, tension/shear interaction, and considerations for supplemental reinforcement are evaluated.
Example 9—Group of headed studs in tension near an edge with eccentricity	A similar group of headed anchors welded to a plate shown in Example 4 is supporting an eccentric tension load near a free edge of concrete. Unequal force distribution and edge effects in the anchors are considered.
Example 10—Multiple headed anchor connection subjected to seismic moment and shear	A group of eight welded headed anchors supports an embedded plate with an eccentric shear load that produces unequal force distribution among the anchor group. The free edges require consideration of edge effects. Supplemental reinforcement and tension/shear interaction are evaluated.
Example 11—Multiple post-installed anchor connection subjected to seismic moment and shear	A group of six post-installed undercut anchors supports a plate with an eccentric seismic shear load that produces unequal force distribution among the anchor group. Single anchor and group strength are evaluated using sample anchor qualification testing data to provide ductile failure. Tension/shear interaction is evaluated.
Example 12—Multiple headed anchor connection subjected to seismic moment and shear	A group of six headed anchors welded to a plate supports an eccentric seismic shear load that produces unequal force distribution among the anchor group. Plastic design is used to evaluate the anchor and plate strength. Tension/shear interaction is evaluated.
Example 13—Group of tension anchors on a pier with shear lug	A concrete pier supporting a column base with cast-in anchors is evaluated for large shear and tension forces. Concrete breakout strength is exceeded and the pier reinforcing steel is used to transfer the tension force. Shear lug design using provisions of ACI 349 is included.

ACI 318-05, thus avoiding the potential problem of brittle failure associated with concrete breakout.

1.4—Commentary on seismic requirements for Appendix D of ACI 318-08

Several changes were made from the previous seismic requirements for Appendix D stated in Section 1.3 of this guide. The levels of seismic risk from previous ACI 318 editions have been correlated in ACI 318-08 to the corresponding design methods, categories, and zones shown in the model building codes. The seismic reduction factor of 0.75 that is applied to the design strength of ductile steel has been eliminated. For Examples 1, 2, 11, and 12, which are discussed in Section 1.3 of this guide, the removal of this reduction factor for steel would indicate that a brittle concrete breakout failure would control the design in most cases. Nonductile failure modes are allowed to control seismic design in ACI 318-08 by imposing an additional reduction factor of 0.4 to the concrete breakout strength. This factor, when combined with the required seismic reduction factor of 0.75, which is associated with concrete failure modes, results in a total reduction of 0.3, and reduces the concrete breakout design strength from the ACI 318-05 levels. The intent of reducing the permissible strength is to force the anchorage system to resist the earthquake load elastically and avoid brittle failure in the concrete.

1.5—Commentary on notation and definitions for Appendix D ACI 318-08

A new definition, "anchor reinforcement" has been defined in ACI 318-08 Appendix D to include the situation where reinforcing steel is specifically designed to transfer all the anchorage forces into the structure without considering the concrete breakout strength. This anchor reinforcement design approach occurs in cases where the concrete breakout strength is insufficient due to geometric restraints. Example 13 provides information on designing the anchorage using anchor reinforcement. Adding this new definition helped to distinguish the term from supplemental reinforcement. Supplemental reinforcement present in the direction of the load can provide restraint and improve ductility for the anchorage. Although supplemental reinforcement is not explicitly designed to transfer the load, it has been experimentally shown to improve ductility, thereby allowing an increase in the design strength of the connection through an increase in the phi factor. Examples 8 and 10 demonstrate the use of supplemental reinforcement.

A new term $\psi_{c,V}$ has been included in ACI 318-08 Appendix D to provide a modification factor to increase the basic concrete breakout strength in shear when the thickness of the section, h_a, is less than $1.5c_{a1}$.

The term for anchor diameter was changed from d_o to d_a in ACI 318-08 Appendix D.

1.6—Anchor designs featured in example problems

Table 1.2 contains a brief description of the anchor designs featured in each example problem.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

Notations are defined in Chapter 2 of ACI 318-05 and in Examples 1 through 13 of this guide.

A_{brg} = net bearing area of the head of stud, anchor bolt, or headed deformed bar, in.^{2}

A_{Nc} = projected concrete failure area of a single anchor or group of anchors, for calculation of strength in tension, in.^{2}

A_{Nco} = projected concrete failure area of a single anchor, for calculation of strength in tension if not limited by edge distance or spacing, in.^{2}

A_{se} = effective cross-sectional area of anchor, in.^{2}

A_{Vc} = projected concrete failure area of a single anchor or group of anchors, for calculation of strength in shear, in.^{2}

A_{Vco} = projected concrete failure area of a single anchor, for calculation of strength in shear, if not limited by corner influences, spacing, or member thickness, in.^{2}

c_{ac} = critical edge distance required to develop the basic concrete breakout strength of a post-installed anchor in uncracked concrete without supplementary reinforcement to control splitting, in.

$c_{a,max}$ = maximum distance from center of an anchor shaft to the edge of concrete, in.

$c_{a,min}$ = minimum distance from center of an anchor shaft to the edge of concrete, in.

c_{a1} = distance from the center of an anchor shaft to the edge of concrete in one direction, in. If shear is applied to anchor, c_{a1} is taken in the direction of the applied shear. If tension is applied to the anchor, c_{a1} is the minimum edge distance

c'_{a1} = limiting value of c_{a1} when anchors are located less than $1.5c_{a1}$ from three or more edges (Fig. RD.6.2.4), Appendix D

c_{a2} = distance from center of an anchor shaft to the edge of concrete in the direction perpendicular to c_{a1}, in.

d_o = outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt, 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, in.; e'_N is always positive

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_a = thickness of member in which an anchor is located, measured parallel to anchor axis, in.

h_{ef} = effective embedment depth of anchor, in.

h'_{ef} = limiting value of h_{ef} when anchors are located less than $1.5h_{ef}$ from three or more edges (Fig. RD.5.2.3), Appendix D

k_c = coefficient for basic concrete breakout strength in tension

k_{cp} = coefficient for pryout strength

ℓ_d = development length in tension of deformed bar, deformed wire, plain and deformed welded wire reinforcement, or pretensioned strand, in.

ℓ_{dh} = development length in tension of deformed bar or deformed wire with a standard hook, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency], plus inside radius of bend and one bar diameter), in.

ℓ_e = load-bearing length of anchor for shear, in.

N_b = basic concrete breakout strength in tension of a single anchor in cracked concrete, lb

N_{cb} = nominal concrete breakout strength in tension of a single anchor, lb

N_{cbg} = nominal concrete breakout strength in tension of a group of anchors, lb

N_n = nominal strength in tension, lb

N_p = pullout strength in tension of a single anchor in cracked concrete, lb

N_{pn} = nominal pullout strength in tension of a single anchor, lb

N_{sa} = nominal strength of a single anchor or group of anchors in tension as governed by the steel strength, lb

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

s = center-to-center spacing of anchors, in.

V_b = basic concrete breakout strength in shear of a single anchor in cracked concrete, lb

V_{cb} = nominal concrete breakout strength in shear of a single anchor, lb

V_{cbg} = nominal concrete breakout strength in shear of a group of anchors, lb

V_{cp} = nominal concrete pryout strength of a single anchor, lb

V_{cpg} = nominal concrete pryout strength of a group of anchors, lb

V_{sa} = nominal strength in shear of a single anchor or group of anchors as governed by the steel strength, lb

V_{ua} = factored shear force applied to a single anchor or group of anchors, lb

ϕ = strength reduction factor

$\psi_{c,N}$ = factor used to modify tensile strength of anchors based on presence or absence of cracks in concrete

$\psi_{c,P}$ = factor used to modify pullout strength of anchors based on presence or absence of cracks in concrete

$\psi_{c,V}$ = factor used to modify shear strength of anchors based on presence or absence of cracks in concrete and presence or absence of supplementary reinforcement

$\psi_{cp,N}$ = factor used to modify tensile strength of post-installed anchors intended for use in uncracked concrete without supplementary reinforcement

$\psi_{ec,N}$ = factor used to modify tensile strength of anchors based on eccentricity of applied loads

$\psi_{ed,N}$ = factor used to modify tensile strength of anchors based on proximity to edges of concrete member

$\psi_{ed,V}$ = factor used to modify shear strength of anchors based on proximity to edges of concrete member

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, "ACI Concrete Terminology," http://terminology.concrete.org.

Refer to Chapter 3 for definitions used in this guide.

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Chapter 3 of ACI 355.3R—Reprint of ACI 318-05 Appendix D

APPENDIX D — ANCHORING TO CONCRETE

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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, hooked bolts (J- or L-bolt), headed studs, expansion anchors, or undercut anchors.

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 percent, or reduction in area of less than 30 percent, or both.

Cast-in anchor — A headed bolt, headed stud, or hooked bolt 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 a 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 steel element — An element with a tensile test elongation of at least 14 percent and reduction in area of at least 30 percent. A steel element meeting the requirements of ASTM A 307 shall be considered ductile.

COMMENTARY

RD.1 — Definitions

Brittle steel element and ductile steel element The 14 percent elongation should be measured over the gage length specified in the appropriate ASTM standard for the steel.

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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.

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 percent confidence that there is 95 percent probability of the actual strength exceeding the nominal strength.

Hooked bolt — A cast-in anchor anchored mainly by mechanical interlock from the 90-deg bend (L-bolt) or 180-deg bend (J-bolt) at its lower end, having a minimum e_h of $3d_o$.

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.

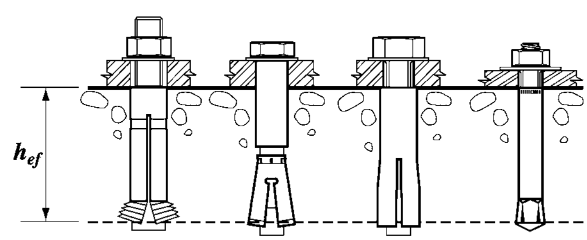
Post-installed anchor — An anchor installed in hardened concrete. Expansion anchors and undercut anchors are examples of post-installed anchors.

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.

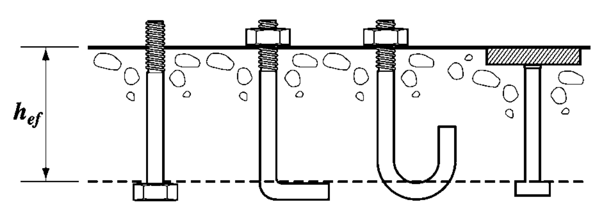
COMMENTARY

Effective embedment depths for a variety of anchor types are shown in Fig. RD.1.

Five percent fractile — The determination of the coefficient K_{05} associated with the 5 percent fractile, $\bar{x} - K_{05}s_s$, depends on the number of tests, n, used to compute the sample mean, $\bar{x}$, and sample standard deviation, s_s. Values of K_{05} range, for example, from 1.645 for $n = \infty$, to 2.010 for $n = 40$, and 2.568 for $n = 10$. With this definition of the 5 percent fractile, the nominal strength in D.4.2 is the same as the characteristic strength in ACI 355.2.



(a) Post-installed anchors



(b) Cast-in-place anchors

Fig. RD.1—Types of anchors.

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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 elements. Specialty inserts are not within the scope of this appendix.

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 anchors in concrete used to transmit structural loads by means of tension, shear, or a combination of tension and shear between (a) connected structural elements; or (b) safety-related attachments and structural elements. 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. Specialty inserts, through bolts, multiple anchors connected to a single steel plate at the embedded end of the anchors, adhesive or grouted 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.

D.2.3 — Headed studs and headed bolts having a geometry that has been demonstrated to result in a pullout strength in uncracked concrete equal or exceeding $\mathbf{1.4}\boldsymbol{N_p}$ (where $\boldsymbol{N_p}$ is given by Eq. (D-15)) are included. Hooked bolts that have a geometry that has been demonstrated to result in a pullout strength

COMMENTARY

RD.2 — Scope

RD.2.1 — Appendix D is restricted in scope to structural anchors that transmit structural loads related to strength, stability, or life safety. Two types of applications are envisioned. The first is connections between structural elements where the failure of an anchor or an anchor group could result in loss of equilibrium or stability of any portion of the structure. The second is where safety-related attachments that are not part of the structure (such as sprinkler systems, heavy suspended pipes, or barrier rails) are attached to structural elements. The levels of safety defined by the combinations of load factors and ϕ factors are appropriate for structural applications. Other standards may require more stringent safety levels during temporary handling.

RD.2.2 — The wide variety of shapes and configurations of specialty inserts makes it difficult to prescribe generalized tests and design equations for many insert types. Hence, they have been excluded from the scope of Appendix D. Adhesive anchors are widely used and can perform adequately. At this time, however, such anchors are outside the scope of this appendix.

RD.2.3 — Typical cast-in headed studs and headed bolts with geometries consistent with ANSI/ASME B1.1,[D.1] B18.2.1,[D.2] and B18.2.6[D.3] have been tested and proven to behave predictably, so calculated pullout values are acceptable. Post-installed anchors do not have predictable pullout capacities, and therefore are required to be tested. For a

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without the benefit of friction in uncracked concrete equal or exceeding **1.4N_p** (where N_p is given by Eq. (D-16)) are included. Post-installed anchors that meet the assessment requirements of ACI 355.2 are included. The suitability of the post-installed anchor for use in concrete shall have been demonstrated by the ACI 355.2 prequalification tests.

D.2.4 — Load applications that are predominantly high cycle fatigue or impact loads are not covered by this appendix.

D.3 — General requirements

D.3.1 — Anchors and anchor groups 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.

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.

D.3.3 — When anchor design includes seismic loads, the additional requirements of D.3.3.1 through D.3.3.5 shall apply.

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post-installed anchor to be used in conjunction with the requirements of this appendix, the results of the ACI 355.2 tests have to indicate that pullout failures exhibit an acceptable load-displacement characteristic or that pullout failures are precluded by another failure mode.

RD.2.4 — The exclusion from the scope of load applications producing high cycle fatigue or extremely short duration impact (such as blast or shock wave) are not meant to exclude seismic load effects. D.3.3 presents additional requirements for design when seismic loads are included.

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 capacities of ductile anchor groups.

RD.3.3 — Post-installed structural anchors are required to be qualified for moderate or high seismic risk zone usage by demonstrating the capacity to undergo large displacements through several cycles as specified in the seismic simulation tests of ACI 355.2. Because ACI 355.2 excludes plastic hinge zones, Appendix D is not applicable to the design of anchors in plastic hinge zones under seismic loads. In addition, the design of anchors in zones of moderate or high seismic risk is based on a more conservative approach by the introduction of 0.75 factor on the design strength ϕN_n and ϕV_n, and by requiring the system to have adequate ductility. Anchorage capacity should be governed by ductile yielding of a steel element. If the anchor cannot meet these ductility requirements, then the attachment is required to be designed so as to yield at a load well below the anchor capacity. In designing attachments for adequate ductility, the ratio of yield to ultimate load capacity should be considered. A connection element could yield only to result in a secondary failure as one or more

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D.3.3.1 —The provisions of Appendix D do not apply to the design of anchors in plastic hinge zones of concrete structures under seismic loads.

D.3.3.2 — In regions of moderate or high seismic risk, or for structures assigned to intermediate or high seismic performance or design categories, post-installed structural anchors for use under D.2.3 shall have passed the Simulated Seismic Tests of ACI 355.2.

D.3.3.3 — In regions of moderate or high seismic risk, or for structures assigned to intermediate or high seismic performance or design categories, the design strength of anchors shall be taken as $0.75\phi N_n$ and $0.75\phi V_n$, where ϕ 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.3.3.4 — In regions of moderate or high seismic risk, or for structures assigned to intermediate or high seismic performance or design categories, anchors shall be designed to be governed by tensile or shear strength of a ductile steel element, unless D.3.3.5 is satisfied.

D.3.3.5 — Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a load level corresponding to anchor forces no greater than the design strength of anchors specified in D.3.3.3.

D.3.4 — All provisions for anchor axial tension and shear strength apply to normalweight concrete. If lightweight concrete is used, provisions for N_n and V_n shall be modified by multiplying all values of $\sqrt{f_c'}$ affecting N_n and V_n, by 0.75 for all-lightweight concrete and 0.85 for sand-lightweight concrete. Linear interpolation shall be permitted when partial sand replacement is used.

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.

COMMENTARY

elements strain harden and fail if the ultimate load capacity is excessive when compared to the yield capacity.

Under seismic conditions, the direction of shear loading may not be predictable. The full shear load should be assumed in any direction for a safe design.

RD.3.3.1 —Section 3.1 of ACI 355.2 specifically states that the seismic test procedures do no simulate the behavior of anchors in plastic hinge zones. The possible higher level of cracking and spalling in plastic hinge zones are beyond the damage states for which Appendix D is applicable.

RD.3.5 — A limited number of tests of cast-in-place and post-installed anchors in high-strength concrete[D.7] indicate that the design procedures contained in this appendix become unconservative, particularly for cast-in anchors in concrete with compressive strengths in the range of 11,000 to 12,000 psi. Until further tests are available, an upper limit on f_c' of 10,000 psi has been imposed in the design of cast-in-place anchors. This is consistent with Chapters 11 and 12. The companion ACI 355.2 does not require testing of

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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 percent 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, except as required in D.3.3,

$$\phi N_n \geq N_{ua} \quad \text{(D-1)}$$

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

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

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

D.4.2 — The nominal strength for any anchor or group of anchors shall be based on design models that result in predictions of strength in substantial agreement with results of comprehensive tests. The materials used in

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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.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 RD.4.1(b) Comprehensive discussions of anchor failure modes are included in References D.8 to D.10. Any model that complies with the requirements of D.4.2 and D.4.3 can be used to establish the concrete-related strengths. For anchors such as headed bolts, headed studs, and post-installed anchors, the concrete breakout design methods of D.5.2 and D.6.2 are acceptable. The anchor strength is also dependent on the pullout strength of D.5.3, the side-face blowout strength of D.5.4, and the minimum spacings and edge distances of D.8. The design of anchors for tension recognizes that the strength of anchors is sensitive to appropriate installation; installation requirements are included in D.9. Some post-installed anchors are less sensitive to installation errors and tolerances. This is reflected in varied ϕ factors based on the assessment criteria of ACI 355.2.

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

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the tests shall be compatible with the materials used in the structure. The nominal strength shall be based on the 5 percent 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.

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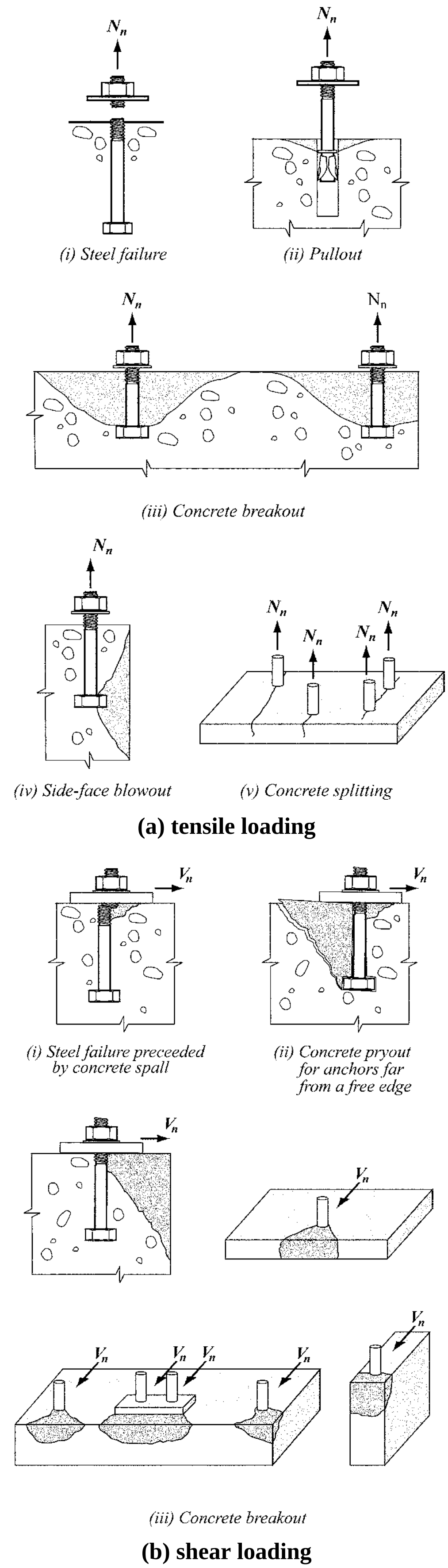

Fig. RD.4.1 — Failure modes for anchors.

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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.

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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.8, D.11, and D.12 provide substantial information on design of such reinforcement. The effect of such supplementary reinforcement 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 in order to include the effects of supplementary reinforcement.

For anchors exceeding the limitations of D.4.2.2, or for situations where geometric restrictions limit breakout capacity, 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 breakout capacity.

The 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)

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.

RD.4.2.2 — The method for concrete breakout design included as "considered to satisfy" D.4.2 was developed from the Concrete Capacity Design (CCD) Method,[D.9,D.10] which was an adaptation of the κ Method[D.13,D.14] and is considered to be accurate, relatively easy to apply, and capable of extension to irregular layouts. The CCD Method predicts the load capacity 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 breakout strength calculations are based on a model suggested in the κ Method. It is consistent with a breakout prism angle of approximately 35 degrees [Fig. RD.4.2.2(a) and (b)].

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.

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

a) Anchor governed by strength of a ductile steel element

i) Tension loads 0.75
ii) Shear loads 0.65

b) Anchor governed by strength of a brittle steel element

i) Tension loads 0.65
ii) Shear loads 0.60

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

	Condition A	Condition B
i) Shear loads	0.75	0.70
ii) Tension loads		
Cast-in headed studs, headed bolts, or hooked bolts	0.75	0.70

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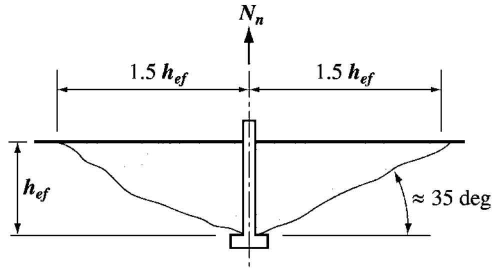

Fig. RD.4.2.2(a)—Breakout cone for tension

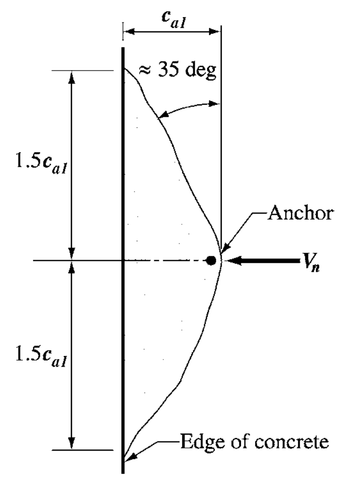

Fig. RD.4.2.2(b)—Breakout cone for shear

RD.4.4 — The ϕ factors for steel strength are based on using f_{uta} to determine the nominal strength of the anchor (see D.5.1 and D.6.1) rather than f_{ya} as used in the design of reinforced concrete members. Although the ϕ factors for use with f_{uta} appear low, they result in a level of safety consistent with the use of higher ϕ factors applied to f_{ya}. The smaller ϕ factors for shear than for tension do not reflect basic material differences but rather account for the possibility of a non-uniform distribution of shear in connections with multiple anchors. It is acceptable to have a ductile failure of a steel element in the attachment if the attachment is designed so that it will undergo ductile yielding at a load level no greater than 75 percent of the minimum design strength of an anchor (See D.3.3.4). 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.8, D.11, D.12, and D.15. Further discussion of strength reduction factors is presented in RD.4.5.

The ACI 355.2 tests for sensitivity to installation procedures determine the category appropriate for a particular anchoring device. In the ACI 355.2 tests, the effects of variability in anchor torque during installation, tolerance on drilled hole size, energy level used in setting anchors, and

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Post-installed anchors with category as determined from ACI 355.2		
Category 1 (Low sensitivity to installation and high reliability)	0.75	0.65
Category 2 (Medium sensitivity to installation and medium reliability)	0.65	0.55
Category 3 (High sensitivity to installation and lower reliability)	0.55	0.45

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

a) Anchor governed by strength of a ductile steel element

i) Tension loads 0.80
ii) Shear loads 0.75

b) Anchor governed by strength of a brittle steel element

i) Tension loads 0.70
ii) Shear loads 0.65

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

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

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for anchors approved for use in cracked concrete, increased crack widths are considered. The three categories of acceptable post-installed anchors are:

Category 1 — low sensitivity to installation and high reliability;

Category 2 — medium sensitivity to installation and medium reliability; and

Category 3 — high sensitivity to installation and lower reliability.

The capacities of anchors under shear loads are not as sensitive to installation errors and tolerances. Therefore, for shear calculations of all anchors, ϕ = 0.75 for Condition A and ϕ = 0.70 for Condition B.

RD.4.5 — As noted in R9.1, the 2002 code incorporated the load factors of ASCE 7-98 and the corresponding strength reduction factors provided in the 1999 Appendix C into 9.2 and 9.3, except that the factor for flexure has been increased. Developmental studies for the ϕ factors to be used for Appendix D were based on the 1999 9.2 and 9.3 load and strength reduction factors. The resulting ϕ factors are presented in D.4.5 for use with the load factors of the 2002 Appendix C. The ϕ factors for use with the load factors of the 1999 Appendix C were determined in a manner consistent with the other ϕ factors of the 1999 Appendix C. These ϕ factors are presented in D.4.4 for use with the load factors of 2002 9.2. Since developmental studies for ϕ factors to be used with Appendix D, for brittle concrete failure modes, were performed for the load and strength reduction factors now given in Appendix C, the discussion of the selection of these ϕ factors appears in this section.

Even though the ϕ factor for plain concrete in Appendix C uses a value of 0.65, the basic factor for brittle concrete failures (ϕ = 0.75) was chosen based on results of probabilistic studies[D.16] that indicated the use of ϕ = 0.65 with mean values of concrete-controlled failures produced adequate safety levels. Because the nominal resistance expressions used in this appendix and in the test requirements are based on the 5 percent fractiles, the ϕ = 0.65 value would be overly conservative. Comparison with other design procedures and probabilistic studies[D.16] indicated that the choice of ϕ = 0.75 was

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Post-installed anchors with category as determined from ACI 355.2		
Category 1 (Low sensitivity to installation and high reliability)	0.85	0.75
Category 2 (Medium sensitivity to installation and medium reliability)	0.75	0.65
Category 3 (High sensitivity to installation and lower reliability)	0.65	0.55

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.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 $1.9f_{ya}$ and 125,000 psi.

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justified. For applications with supplementary reinforcement and more ductile failures (Condition A), the ϕ factors are increased. The value of $\phi = 0.85$ is compatible with the level of safety for shear failures in concrete beams, and has been recommended in the PCI Design Handbook[D.17] and by ACI 349.[D.15]

RD.5 — Design requirements for tensile loading

RD.5.1 — Steel strength of anchor in tension

RD.5.1.2 — The nominal tension strength of anchors is best represented by $A_{se}f_{uta}$ rather than $A_{se}f_{ya}$ because the large majority of anchor materials do not exhibit a well-defined yield point. The American Institute of Steel Construction (AISC) has based tension strength of anchors on $A_{se}f_{uta}$ since the 1986 edition of their specifications. The use of Eq. (D-3) with 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.[D.18]

The limitation of $1.9f_{ya}$ on f_{uta} is to ensure that under service load conditions the anchor does not exceed f_{ya}. The limit on f_{uta} of $1.9f_{ya}$ was determined by converting the LRFD provisions to corresponding service level conditions. For Section 9.2, the average load factor of 1.4 (from $1.2D + 1.6L$) divided by the highest ϕ factor (0.75 for tension) results in a limit of f_{uta}/f_{ya} of 1.4/0.75 = 1.87. For Appendix C, the average load factor of 1.55 (from $1.4D + 1.7L$), divided

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by the highest ϕ factor (0.80 for tension), results in a limit of f_{uta}/f_{ya} of 1.55/0.8 = 1.94. For consistent results, the serviceability limitation of f_{uta} was taken as $1.9f_{ya}$. If the ratio of f_{uta} to f_{ya} exceeds this value, the anchoring may be subjected to service loads above f_{ya} under service loads. Although not a concern for standard structural steel anchors (maximum value of f_{uta}/f_{ya} is 1.6 for ASTM A 307), the limitation is applicable to some stainless steels.

The effective cross-sectional area of an anchor should be provided by the manufacturer of expansion anchors with reduced cross-sectional area for the expansion mechanism. For threaded bolts, ANSI/ASME B1.1[D.1] defines A_{se} as:

$$A_{se} = \frac{\pi}{4}\left(d_o - \frac{0.9743}{n_t}\right)^2$$

where n_t is the number of threads per in.

D.5.2 — Concrete breakout strength of anchor in tension

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

(a) for a single anchor:

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

(b) for a group of anchors:

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

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

$$A_{Nco} = 9h_{ef}^2 \qquad \text{(D-6)}$$

RD.5.2 — Concrete breakout strength of anchor in tension

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

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

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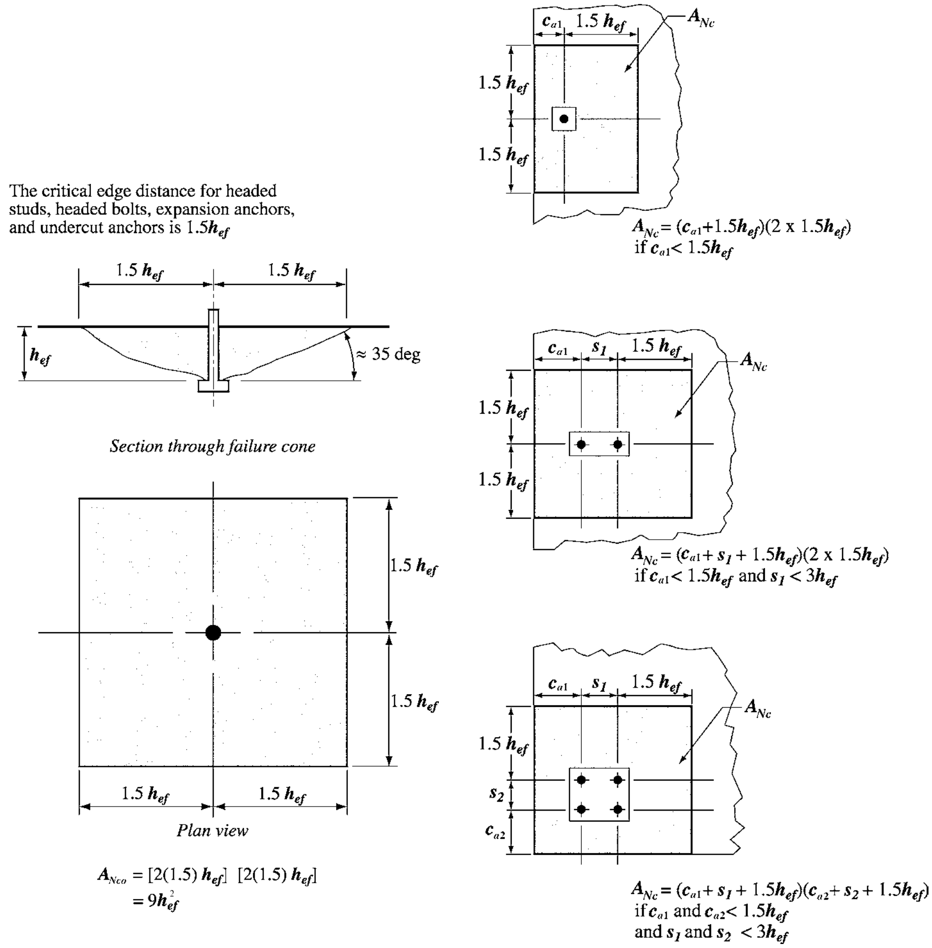

(a) (b)

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

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

$$N_b = k_c\sqrt{f_c'}h_{ef}^{1.5} \quad \text{(D-7)}$$

where

k_c = 24 for cast-in anchors; and

k_c = 17 for post-installed anchors.

The value of k_c for post-installed anchors shall be permitted to be increased above 17 based on ACI 355.2 product-specific tests, but shall in no case exceed 24.

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

$$N_b = 16\sqrt{f_c'}h_{ef}^{5/3} \quad \text{(D-8)}$$

RD.5.2.2 — The basic equation for anchor capacity was derived[D.9-D.11,D.14] assuming a concrete failure prism with an angle of about 35 degrees, considering fracture mechanics concepts.

The values of k_c in Eq. (D-7) were determined from a large database of test results in uncracked concrete[D.9] at the 5 percent fractile. The values were adjusted to corresponding k_c values for cracked concrete.[D.10,D.19] Higher k_c values for post-installed anchors may be permitted, provided they have been determined from product approval testing in accordance with ACI 355.2. For anchors with a deep embedment (h_{ef} > 11 in.), test evidence indicates the use of $h_{ef}^{1.5}$ can be overly conservative for some cases. Often such tests have been with selected aggregates for special applications. An alternative expression (Eq. (D-8)) is provided using $h_{ef}^{5/3}$ for evaluation of cast-in anchors with 11 in. ≤ h_{ef} ≤ 25 in. The limit of 25 in. corresponds to the upper range of test data. This expression can also be appropriate for some

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

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undercut post-installed anchors. However, for such anchors, the use of Eq. (D-8) should be justified by test results in accordance with D.4.2.

RD.5.2.3 — For anchors located less than $\mathbf{1.5}\boldsymbol{h_{ef}}$ from three or more edges, the tensile breakout strength computed by the CCD Method, which is the basis for Eq. (D-4) to (D-11), gives overly conservative results.[D.20] This occurs because the ordinary definitions of $\boldsymbol{A_{Nc}}/\boldsymbol{A_{Nco}}$ do not correctly reflect the edge effects. This problem is corrected by limiting the the value of $\boldsymbol{h_{ef}}$ used in Eq. (D-4) through (D-11) to $\boldsymbol{c_{a,max}}/\mathbf{1.5}$, where $\boldsymbol{c_{a,max}}$ is the largest of the influencing edge distances that are less than or equal to the actual $\mathbf{1.5}\boldsymbol{h_{ef}}$. In no case should $\boldsymbol{c_{a,max}}$ be taken less than one-third of the maximum spacing between anchors within the group. The limit on $\boldsymbol{h_{ef}}$ of at least one-third of the maximum spacing between anchors within the group prevents the designer from using a calculated strength based on individual breakout prisms for a group anchor configuration.

This approach is illustrated in Fig. RD.5.2.3. In this example, the proposed limit on the value of $\boldsymbol{h_{ef}}$ to be used in the computations where $\boldsymbol{h_{ef}} = \boldsymbol{c_{a,max}}/\mathbf{1.5}$, results in $\boldsymbol{h_{ef}} = \boldsymbol{h'_{ef}}$ = 4 in. For this example, this would be the proper value to be used for $\boldsymbol{h_{ef}}$ 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, which originates at the actual $\boldsymbol{h_{ef}}$, toward the surface of the concrete parallel to the applied tension load. The value of $\boldsymbol{h_{ef}}$ used in Eq. (D-4) to (D-11) is determined when either: (a) the outer boundaries of the failure surface first intersect a free edge; or (b) the intersection of the breakout surface between anchors within the group first intersects the surface of the concrete. For the example shown in Fig. RD.5.2.3, point "A" defines the intersection of the assumed failure surface for limiting $\boldsymbol{h_{ef}}$ with the concrete surface.

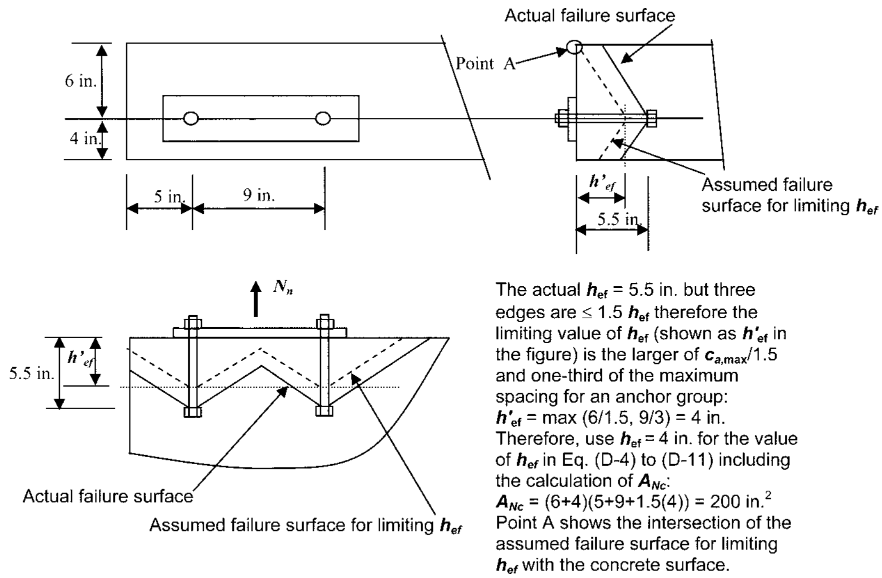

Fig. RD.5.2.3 — Tension in narrow members.

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

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

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

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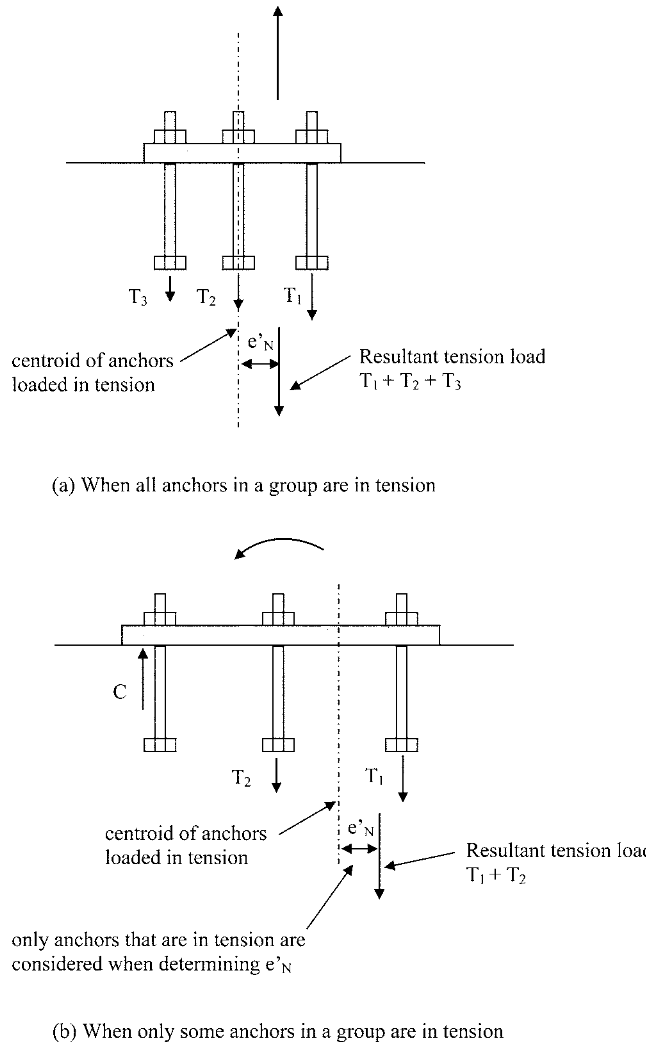

Fig. RD.5.2.4 — Definition of $\mathbf{e'_N}$ *for group anchors.*

RD.5.2.4 — Figure RD.5.2.4(a) shows a group of anchors that are all in tension but the resultant force is eccentric with respect to the centroid of the anchor group. Groups of anchors can also 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.

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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 at service load levels, the following modification factor shall be permitted:

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

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

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

Where the value of k_c used in Eq. (D-7) is taken from the ACI 355.2 product evaluation report for post-installed anchors qualified for use only in uncracked concrete, $\psi_{c,N}$ shall be taken as 1.0.

When analysis indicates cracking at service load levels, $\psi_{c,N}$ 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 ACI 355.2. 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.

D5.2.7 — 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:

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

$$\psi_{cp,N} = \frac{c_{a,min}}{c_{ac}} \geq \frac{1.5h_{ef}}{c_{ac}} \text{ if } c_{a,min} < c_{ac} \quad \text{(D-13)}$$

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

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

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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 capacity of the anchor is further reduced beyond that reflected in A_{Nc}/A_{Nco}. If the smallest side cover distance is greater than or equal to $1.5h_{ef}$, a complete prism can form and there is no reduction ($\psi_{ed,N} = 1$). If the side cover is less than $1.5h_{ef}$, the factor $\psi_{ed,N}$ is required to adjust for the edge effect.[D.9]

RD.5.2.6 — Post-installed and cast-in anchors that have not met the requirements for use in cracked concrete according to ACI 355.2 should be used in uncracked regions only. The analysis for the determination of crack formation should include the effects of restrained shrinkage (see 7.12.1.2). 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 about 0.012 in. should be provided.

The concrete breakout strengths given by Eq. (D-7) and (D-8) assume cracked concrete (that is, $\psi_{c,N}$ = 1.0) with $\psi_{c,N}k_c$ = 24 for cast-in-place, and 17 for post-installed (cast-in 40 percent 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 $\psi_{c,N}k_c$ factors of 30 for cast-in and 24 for post-installed (25 percent 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.

RD.5.2.7 — The design provisions in D.5 are based on the assumption that the basic concrete breakout strength can be achieved if the minimum edge distance, $c_{a,min}$, equals $1.5h_{ef}$. However, test results[D.21] 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 splitting failure may occur before reaching the concrete breakout strength defined in D.5.2.1. To account for this potential

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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.

D.5.2.8 — Where an additional plate or washer is added at the head of the anchor, it shall be permitted to calculate the projected area of the failure surface by projecting the failure surface outward $1.5h_{ef}$ from the effective perimeter of the plate or washer. The effective perimeter shall not exceed the value at a section projected outward more than the thickness of the washer or plate from the outer edge of the head of the anchor.

D.5.3 — Pullout strength of anchor in tension

RD.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:

$$N_{pn} = \psi_{c,P} N_p \qquad \text{(D-14)}$$

where $\psi_{c,P}$ is defined in D.5.3.6.

D.5.3.2 — For post-installed expansion and undercut anchors, the values of N_p shall be based on the 5 percent fractile of results of tests performed and evaluated according to ACI 355.2. It is not permissible to calculate the pullout strength in tension for such anchors.

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 and hooked anchors;[D.8,D.22] 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.

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. For single J- or L-bolts, it shall be permitted to evaluate the pullout strength in tension using D.5.3.5. Alternatively, it shall be permitted to use values of N_p based on the 5 percent fractile of tests performed and evaluated in the same manner as the ACI 355.2 procedures but without the benefit of friction.

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.

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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:

$$N_p = 8A_{brg}f_c' \quad \text{(D-15)}$$

D.5.3.5 — The pullout strength in tension of a single hooked bolt, N_p, for use in Eq. (D-14) shall not exceed:

$$N_p = 0.9f_c'e_hd_o \quad \text{(D-16)}$$

where $3d_o \leq e_h \leq 4.5d_o$.

D.5.3.6 — For an anchor located in a region of a concrete member where analysis indicates no cracking at service load levels, the following modification factor shall be permitted

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

where analysis indicates cracking at service load levels, $\psi_{c,P}$ shall be taken as 1.0.

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

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

$$N_{sb} = 160c_{a1}\sqrt{A_{brg}}\sqrt{f_c'} \quad \text{(D-17)}$$

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

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

$$N_{sbg} = \left(1 + \frac{s}{6c_{a1}}\right)N_{sb} \quad \text{(D-18)}$$

where s is spacing of the outer anchors along the edge in the group; and N_{sb} is obtained from Eq. (D-17) without modification for a perpendicular edge distance.

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RD.5.3.4 — Equation (D-15) corresponds to the load at which the concrete under the anchor head begins to crush.[D.8,D.15] 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.5.3.5 — Equation (D-16) for hooked bolts was developed by Lutz based on the results of Reference D.22. Reliance is placed on the bearing component only, neglecting any frictional component because crushing inside the hook will greatly reduce the stiffness of the connection, and generally will be the beginning of pullout failure. The limits on e_h are based on the range of variables used in the three tests programs reported in Reference D.22.

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.23. 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.

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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

$$V_{sa} = nA_{se}f_{uta} \quad \text{(D-19)}$$

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

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

$$V_{sa} = n0.6A_{se}f_{uta} \quad \text{(D-20)}$$

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

(c) for post-installed anchors where sleeves extend through the shear plane, V_{sa} shall be based on the results of tests performed and evaluated according to ACI 355.2. Alternatively, Eq. (D-20) shall be permitted to be used.

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.6.2 — Concrete breakout strength of anchor in shear

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

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

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

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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-19) and (D-20) with 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.[D.18]

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} a of $1.9f_{ya}$ was determined by converting the LRFD provisions to corresponding service level conditions as discussed in RD.5.1.2.

The effective cross-sectional area of an anchor should be provided by the manufacturer of expansion anchors with reduced cross-sectional area for the expansion mechanism. For threaded bolts, ANSI/ASME B1.1[D.1] defines A_{se} as:

$$A_{se} = \frac{\pi}{4}\left(d_o - \frac{0.9743}{n_t}\right)^2$$

where n_t is the number of threads per in.

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 (See 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-21) and (D-22), and $\psi_{ec,V}$ in Eq. (D-22). 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.

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

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}} \psi_{ec,V}\psi_{ed,V}\psi_{c,V}V_b \qquad \text{(D-22)}$$

(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-21) or (D-22), respectively, with the shear force assumed to act perpendicular to the edge and with $\psi_{ed,V}$ taken equal to 1.0.

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

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

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

$$A_{Vco} = 4.5(c_{a1})^2 \qquad \text{(D-23)}$$

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.

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Fig. RD.6.2.1(a) shows A_{Vco} and the development of Eq. (D-23). 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. Fig. 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-22) for anchor groups loaded in shear, both assumptions for load distribution illustrated in examples on the right side of Fig. RD.6.2.1(b) should be considered because the anchors nearest the edge could fail first or the whole group could fail as a unit with the failure surface originating from the anchors farthest from the edge. If the anchors are welded to a common plate, when the anchor nearest the front edge begins to form a failure cone, shear load would be transferred to the stiffer and stronger rear anchor. For this reason, anchors welded to a common plate do not need to consider the failure mode shown in the upper right figure of Fig. RD.6.2.1(b). The *PCI Design Handbook* approach[D.17] suggests in Section 6.5.2.2 that the capacity 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.11] 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 capacity 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 capacity 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.8.

For the case of anchors near a corner subjected to a shear force with components normal to each edge, a satisfactory solution is to check independently the connection for each component of the shear force. 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.11.

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

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The case of shear force parallel to an edge is shown in Fig. RD.6.2.1(c). A special case can arise with shear force parallel to the edge near a corner. In the example of a single anchor near a corner (See Fig. RD.6.2.1(d)), the provisions for shear force applied perpendicular to the edge should be checked in addition to the provisions for shear force applied parallel to the edge.

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

$$V_b = 7\left(\frac{\ell_e}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f_c'}(c_{a1})^{1.5} \qquad \text{(D-24)}$$

RD.6.2.2 — Like the concrete breakout tensile capacity, the concrete breakout shear capacity does not increase with the failure surface, which is proportional to $(c_{a1})^2$. Instead the capacity increases proportionally to $(c_{a1})^{1.5}$ due to size effect. The capacity is also influenced by the anchor stiffness and the anchor diameter.[D.9-D.11,D.14]

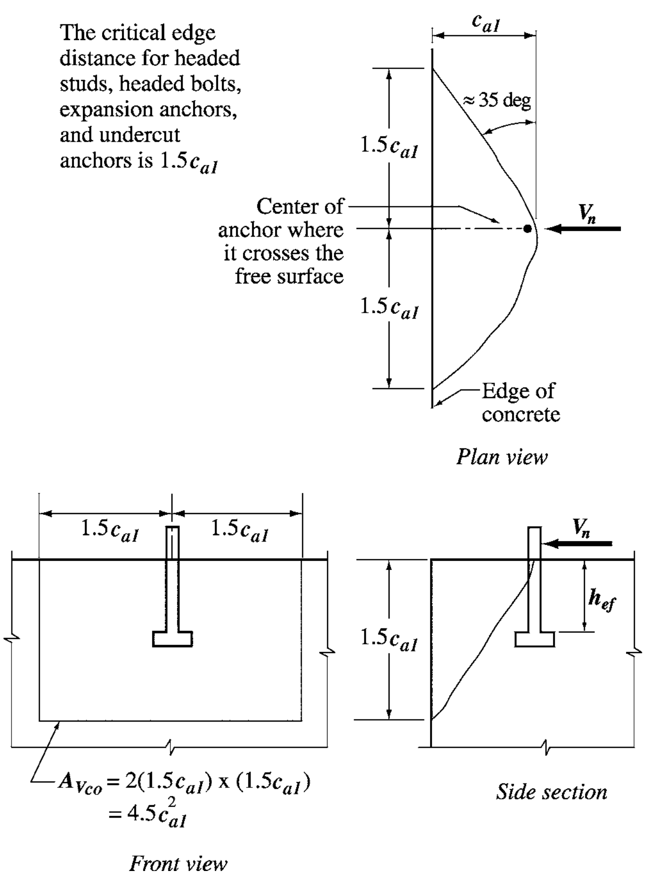

Fig. RD.6.2.1(a)—Calculation of A_{Vco}

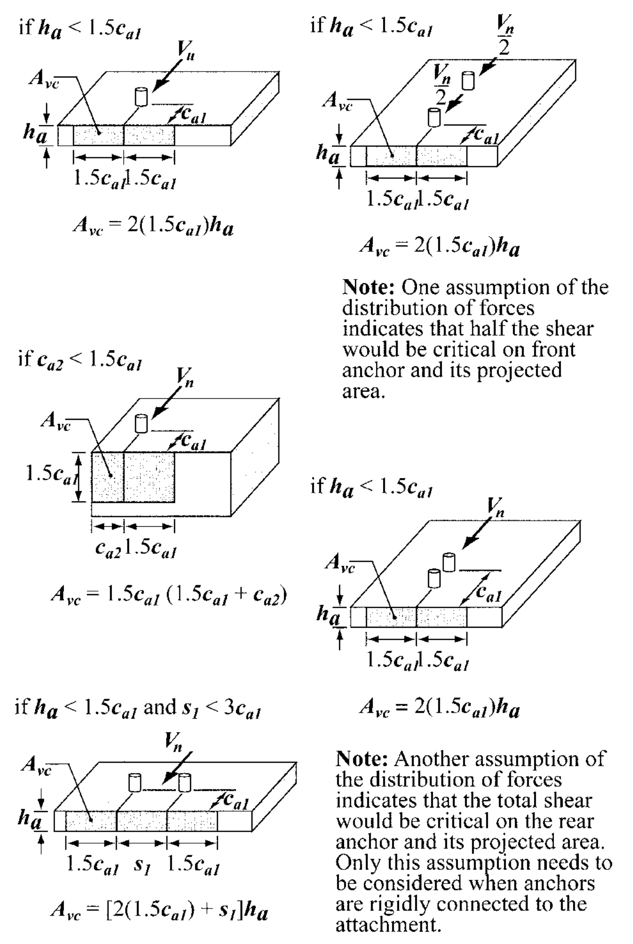

Fig. RD.6.2.1(b) — Projected area for single anchors and groups of anchors and calculation of A_{Vc}

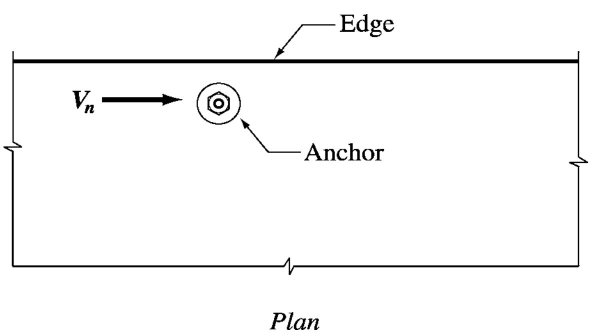

Fig. RD.6.2.1(c) — Shear force parallel to an edge

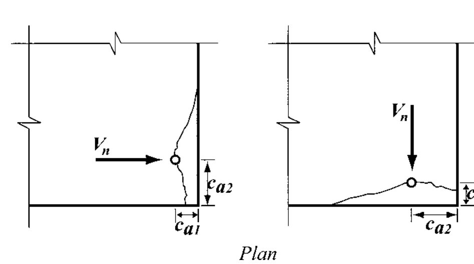

Fig. RD.6.2.1(d) — Shear force near a corner.

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

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

$\ell_e = 2d_o$ for torque-controlled expansion anchors with a distance sleeve separated from expansion sleeve, and

in no case shall ℓ_e exceed $8d_o$.

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

$$V_b = 8\left(\frac{\ell_e}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f_c'}(c_{a1})^{1.5} \qquad \text{(D-25)}$$

where ℓ_e is defined in D.6.2.2.

provided that:

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

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

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

D.6.2.4 — Where anchors are influenced by three or more edges, the value of c_{a1} used in Eq. (D-23) through (D-28) shall not exceed the greatest of: $c_{a2}/1.5$ in either direction, $h_a/1.5$; and one-third of the maximum spacing between anchors within the group.

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The constant, 7, in the shear strength equation was determined from test data reported in Reference D.9 at the 5 percent fractile adjusted for cracking.

RD.6.2.3 — For the special case of cast-in headed bolts continuously welded to an attachment, test data[D.24] show that somewhat higher shear capacity exists, possibly due to the stiff welding connection clamping the bolt more effectively than an attachment with an anchor gap. Because of this, the basic shear value for such anchors is increased. Limits are imposed to ensure sufficient rigidity. The design of supplementary reinforcement is discussed in References D.8, D.11, and D.12.

RD.6.2.4 — For anchors influenced by three or more edges where any edge distance is less than $1.5c_{a1}$, the shear breakout strength computed by the basic CCD Method, which is the basis for Eq. (D-21) though (D-28), gives safe but overly conservative results. These special cases were studied for the κ Method[D.14] and the problem was pointed out by Lutz.[D.20] Similarly, the approach used for tensile breakouts in D.5.2.3, a correct evaluation of the capacity is determined if the value of c_{a1} used in Eq. (D-21) to (D-28) is limited to the maximum of $c_{a2}/1.5$ in each direction, $h_a/1.5$, and one-third of the maximum spacing between anchors within the group. The limit on c_{a1} of at least one-third of the maximum spacing between anchors within the group prevents the designer from using a calculated strength based on individual breakout prisms for a group anchor configuration.

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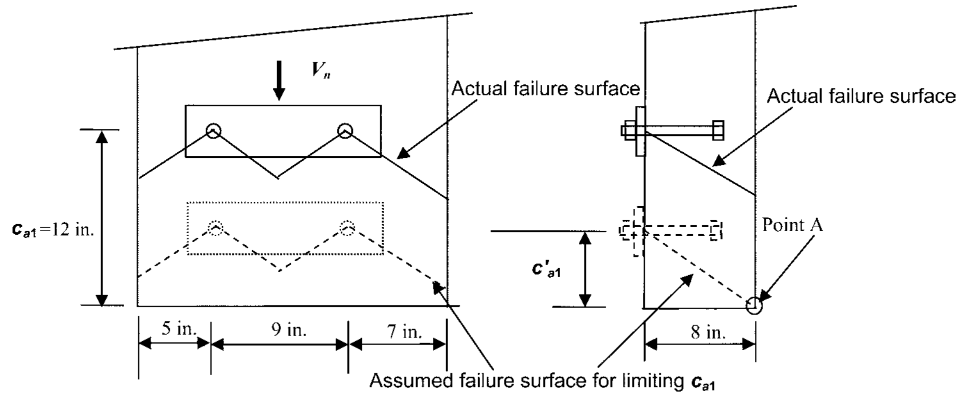

The actual c_{a1} = 12 in. but two orthogonal edges c_{a2} and h_a are ≤ 1.5 c_{a1} therefore the limiting value of c_{a1} (shown as c'_{a1} in the figure) is the larger of $c_{a2,max}$**/1.5**, h_a**/1.5** and one-third of the maximum spacing for an anchor group:
c'_{a1} = max (7/1.5, 8/1.5, 9/3) =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

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

This approach is illustrated in Fig. RD.6.2.4. In this example, the limit on the value of c_{a1} is the largest of c_{a2}**/1.5** in either direction, h_a**/1.5**, and one-third the maximum spacing between anchors for anchor groups results in c'_{a1} = 5.33 in. For this example, this would be the proper value to be used for c_{a1} in computing V_{cb} or V_{cbg}, even if the actual edge distance that the shear is directed toward is larger. The requirement of D.6.2.4 may be visualized by moving the actual concrete breakout surface originating at the actual c_{a1} toward the surface of the concrete in the direction of the applied shear load. The value of c_{a1} used in Eq. (D-21) to (D-28) is determined when either: (a) the outer boundaries of the failure surface first intersect a free edge; or (b) the intersection of the breakout surface between anchors within the group first intersects the surface of the concrete. For the example shown in Fig. RD.6.2.4, point "A" shows the intersection of the assumed failure surface for limiting c_{a1} with the concrete surface.

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

$$\psi_{ec,V} = \frac{1}{\left(1 + \frac{2e_V'}{3c_{a1}}\right)} \leq 1 \qquad \text{(D-26)}$$

If the loading on an anchor group is such that only some anchors are loaded in shear in the same direction, only those anchors that are loaded in shear in the same direction shall be considered when determining the eccentricity of e_V' for use in Eq. (D-26) and for the calculation of V_{cbg} in Eq. (D-22).

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

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

$$\psi_{ed,V} = 1.0 \quad \text{(D-27)}$$

if $c_{a2} \geq 1.5c_{a1}$

$$\psi_{ed,V} = 0.7 + 0.3\frac{c_{a2}}{1.5c_{a1}} \quad \text{(D-28)}$$

if $c_{a2} < 1.5c_{a1}$

D.6.2.7 — For anchors located in a region of a concrete member where analysis indicates no cracking at service loads, 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 at service load levels, 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.

D.6.3 — Concrete pryout strength of anchor in shear

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

(a) for a single anchor:

$$V_{cp} = k_{cp}N_{cb} \quad \text{(D-29)}$$

(b) for a group of anchors:

$$V_{cpg} = k_{cp}N_{cbg} \quad \text{(D-30)}$$

where:

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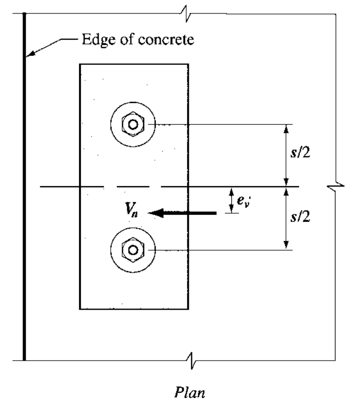

Fig. RD.6.2.5 — Definition of dimensions e_V'.

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

RD.6.3 — Concrete pryout strength of anchor in shear

Reference D.9 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.

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k_{cp} = **1.0** for h_{ef} < 2.5 in.; and
k_{cp} = **2.0** for $h_{ef} \geq 2.5$ in.

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

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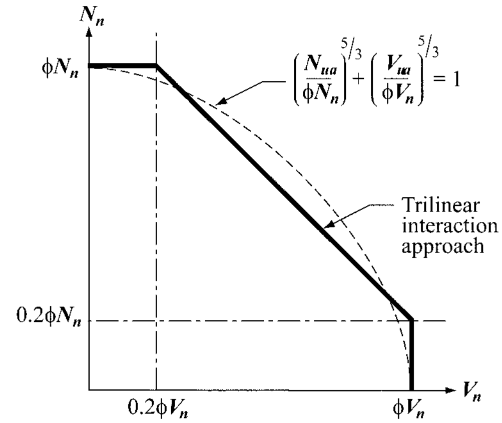


Fig. RD.7 — Shear and tensile load interaction equation.

D.7 — Interaction of tensile and shear forces

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

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

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

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

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2 \qquad \text{(D-31)}$$

RD.7 — Interaction of tensile and shear forces

The shear-tension interaction expression has traditionally been expressed as:

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

where ζ varies from 1 to 2. The current trilinear recommendation is a simplification of the expression where ζ **= 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.

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 ACI 355.2 shall be permitted.

D.8.1 — Unless determined in accordance with D.8.4, 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.

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

The minimum spacings, edge distances, and thicknesses are very 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 of values for minimum spacings, edge distances, and thicknesses of post-installed anchors should be the product-specific tests of ACI 355.2. In some cases, however, specific products are not known in the design stage. Approximate values are provided for use in design.

CODE

D.8.2 — Unless determined in accordance with D.8.4, 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 $6d_o$.

D.8.3 — Unless determined in accordance with D.8.4, 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 ACI 355.2, and shall not be less than 2.0 times the maximum aggregate size. In the absence of product-specific ACI 355.2 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 — For anchors where installation does not produce a splitting force and that will remain untorqued, if the edge distance or spacing is less than those specified in D.8.1 to D.8.3, calculations shall be performed by substituting for d_o a smaller value d_o' that meets the requirements of D.8.1 to D.8.3. Calculated forces applied to the anchor shall be limited to the values corresponding to an anchor having a diameter of d_o' .

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 ACI 355.2, 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.

COMMENTARY

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 — In some cases, it may be desirable to use a larger-diameter anchor than the requirements on D.8.1 to D.8.3 permit. In these cases, it is permissible to use a larger-diameter anchor provided the design strength of the anchor is based on a smaller assumed anchor diameter, d_o' .

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.

CODE

D.9 — Installation of anchors

D.9.1 — Anchors shall be installed in accordance with the project drawings and project specifications.

COMMENTARY

RD.9 — Installation of anchors

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

Notes

CHAPTER 4—DESIGN EXAMPLES

4.1—Example 1: Single headed anchor away from edges subject to seismic tension

Check the capacity of a single anchor, 1 in. diameter, F1554 Grade 36 headed bolt with heavy-hex head installed in the top of a 12-in. thick foundation without edge effects to resist a factored load of 17,000 lb tension (determined from ACI 318-05 Section 9.2.1). The embedment depth is 8 in. and the concrete compressive strength is 6000 psi, as shown in Fig. 4.1.

The structure supported on the foundation slab in this example is located in an area of high seismic risk. Structural loads on the foundation were calculated based on a seismic analysis. Assume normalweight concrete and that a crack forms in the plane of the anchor.

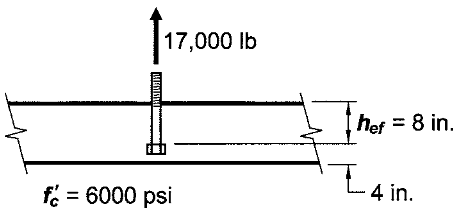


Fig. 4.1—Example 1: Single headed anchor in tension away from edges. (Note: Foundation reinforcement not shown for clarity.)

Step	Calculations and discussion	ACI 318-05 Section
1.	The factored design load is: N_{ua} = 17,000 lb	 9.2.1
2.	General requirement for anchor strength: $\phi N_n \geq N_{ua}$ where ϕN_n is the lowest design strength from all appropriate failure modes in tension as determined from consideration of: • N_{sa} — Steel failure • N_{cb} — Concrete breakout failure • N_{pn} — Pullout and pull-through failure • N_{sb} — Side-face blowout failure In regions of moderate or high seismic risk, the design strength of anchors is limited to $0.75\phi N_n$: $0.75\phi N_n \geq N_{ua}$	 Eq. (D-1) D.4.1.2 D.3.3.3

3.	Check strength of steel anchor and requirements for ductility. Since the anchor is located in a region of moderate or high seismic risk, the anchor strength should be governed by tensile or shear strength of a ductile steel element, unless the other parts of the attachment are designed to yield.	D.3.3.4 D.3.3.5
	ASTM F1554 Grade 36 material meets the requirements of the ductile steel element definition in D.1 (tensile elongation of at least 14% and reduction of area of at least 30%).	
	ASTM F1554 material has 23% elongation in 2 in. of length with 40% reduction in area (Table A.1).	
	The design strength for the steel anchor in regions of moderate or high seismic risk is:	
	$0.75\phi N_{sa} \geq N_{ua}$	
	and	
	ϕN_{sa} < capacity of concrete breakout, pullout, side-face blowout, or splitting failure in tension to satisfy ductility requirements	
	where	
	$\phi = 0.75$	D.4.4(a)
	and the steel bolt strength is:	
	$N_{sa} = nA_{se}f_{uta}$	Eq. (D-3)
	where the minimum value for f_{uta} = 58,000 psi is given for ASTM F1554 Grade 36 material (Table A.1). Note that f_{uta} should not be taken greater than the smaller of $1.9f_{ya}$ and 125,000 psi.	D.5.1.2
	$1.9f_{ya}$ = 1.9(36,000) = 68,400 psi > 58,000 psi – OK	
	n = 1 (single anchor)	
	ANSI/ASME B1.1 defines A_{se}:	
	$A_{se} = \frac{\pi}{4}\left(d_o - \frac{0.9743}{n_t}\right)^2$	RD.5.1.2
	Substituting 1 in. for d_o and 8 (number of threads) for n_t, A_{se} = 0.606 in.^{2}	
	N_{sa} = (1)(0.606 in.^{2})(58,000 psi) = 35,148 lb	
	ϕN_{sa} = (0.75)35,148 lb = 26,361 lb	
	Reduction for regions of moderate or high seismic risk	
	$0.75(\phi N_{sa})$ = 0.75(26,361 lb) = 19,771 lb	D.3.3.3
	19,771 lb > 17,000 lb (requirement of $0.75\phi N_{sa} > N_{ua}$ is met)	

4.	Using the embedment depth h_{ef} = 8 in., check the various concrete failure modes. The basic capacity for nominal concrete breakout strength in tension for a single anchor in regions of moderate or high seismic risk is: $0.75\phi N_{cb} \geq N_{ua}$ and $\phi N_{cb} > \phi N_{sa}$ (to ensure that the steel failure mode governs) where ϕ = 0.70 (for concrete breakout) ϕ = 0.75 (for ductile steel in tension)	D.4.4
	Determine the concrete breakout strength assuming Condition B (no supplementary reinforcement has been provided): $N_{cb} = \frac{A_{Nc}}{A_{Nco}}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$	Eq. (D-4)
	where A_{Nc}/A_{Nco} = 1.0 for single anchor remote from edges (a complete breakout prism is developed and no reduction is required)	RD.5.2.1
	$\psi_{ed,N}$ = 1.0 since anchor is remote from edges ($c_{a,min} \geq 1.5h_{ef}$)	D.5.2.5
	$\psi_{c,N}$ = 1.0 since for this example assumed cracking occurs in the concrete at service load levels	D.5.2.6
	$\psi_{cp,N}$ = 1.0 (cast-in anchors)	D.5.2.7
	The basic concrete breakout strength of a single anchor in tension is: $N_b = k_c\sqrt{f'_c}\,h_{ef}^{1.5}$	Eq. (D-7)
	where k_c = 24 for cast-in anchors. For this example, h_{ef} = 8 in., which is less than 11 in., so Eq. (D-8), $N_b = 16\sqrt{f'_c}\,h_{ef}^{5/3}$, does not apply. Substituting: $N_b = 24\sqrt{6000}(8)^{1.5}$ = 42,065 lb and ϕN_{cb} = 0.7(42,065 lb) = 29,445 lb Reduction for regions of moderate or high seismic risk:	
	$0.75\phi N_{cb}$ = 0.75(29,445 lb) = 22,084 lb	D.3.3.3
	22,084 lb $\geq N_{ua}$ and 22,084 lb > $0.75\phi N_{sa}$ (or 19,771 lb) Therefore, the anchor strength remains governed by ductile steel failure.	

5.	Check the anchor pullout strength: Check the bearing stress under bolt head to prevent a pullout failure mode, which is initiated by crushing of the concrete: $0.75\phi N_{pn} \geq N_{ua}$ and $\phi N_{pn} > \phi N_{sa}$ (to ensure ductile steel failure mode governs) where $\phi = 0.70$ $N_{pn} = \psi_{cp} N_p$ where $N_p = 8A_{brg} f_c'$ $\psi_{c,P} = 1.0$ considering concrete cracking at service loads For 1 in. diameter heavy hex bolt head, the bearing area of the head, $A_{brg} = 1.501$ in.^{2} (Table A.2(a)). Substituting: $N_p = 8(1.501 \text{ in.}^2)(6000 \text{ psi}) = 72{,}048$ lb and $\phi N_{pn} = 0.7(72{,}048 \text{ lb}) = 50{,}433$ lb Reduction for regions of moderate or high seismic risk $0.75(\phi N_{pn}) = 0.75(50{,}433 \text{ lb}) = 37{,}825$ lb $37{,}825 \text{ lb} \geq N_{ua}$ and $37{,}825 \text{ lb} > 0.75\phi N_{sa}$ (or 19,771 lb) Therefore, the anchor strength remains governed by ductile steel failure.	D.5.3 D.4.4(c)ii Eq. (D-14) Eq. (D-15) D.5.3.6 D.3.3.3
6.	Evaluate concrete side-face blowout failure mode: Since this anchor is located far from a free edge of concrete ($c_{a1} \geq 0.4h_{ef}$), concrete side-face blowout failure mode is not applicable.	D.5.4
7.	Evaluate splitting failure: This type of failure can occur in thin slabs where the anchors have torque applied. The minimum edge distance for cast-in anchors with torque applied is $6d_a$, where d_a is the bolt diameter. This failure mode is not applicable because the anchor is remote from an edge.	 D.8.2

8. *Summary:*

The lowest design strength for considering all failure modes shown in Table 4.1 is 19,771 lb for steel failure (controlled by the seismic reduction design capacity equation $0.75\phi N_{sa}$).

Table 4.1—Summary of design strengths and controlling failure mode

Failure mode	Anchor design strength	Calculated design strength, lb	Controlling failure mode
Steel	$0.75\phi N_{sa}$	19,771	← Controls
Concrete breakout	$0.75\phi N_{cb}$	22,084	—
Concrete pullout	$0.75\phi N_{pn}$	37,825	—
Concrete side-face blowout	Not applicable	Not applicable	—
Concrete splitting	Not applicable	Not applicable	—

Therefore, the ASTM F1554 Grade 36, 1 in. diameter anchor bolt with a heavy-hex head and an 8 in. embedment is adequate to resist the 17,000 lb tension factored load in a moderate or high seismic region.

If the seismic provisions of ACI 318-08 are followed, the results would be those shown in Table 4.2.

Table 4.2—Summary of design strengths and controlling mode using ACI 318-08

Failure mode	Anchor design strength (D.3.3.3 for concrete failure modes)	Calculated design strength, lb	Reduced permissible design strength for nonductile failure modes (D.3.3.6)	Calculated permissible design strength,* lb
Steel	ϕN_{sa}	26,361	—	—
Concrete breakout	$0.75\phi N_{cb}$	22,084 (controls†)	$0.4\,(0.75\phi N_{cb})$	8834
Concrete pullout	$0.75\phi N_{pn}$	37,825	—	—
Concrete side-face blowout	Not applicable	Not applicable	—	—
Concrete splitting	Not applicable	Not applicable	—	—

*If the anchor embedment depth or concrete strength cannot be increased enough for the concrete breakout design strength to be larger than the steel design strength, then as an alternate to the ductile design requirements of D.3.3.4 and D.3.3.5, reduced permissible design strength is allowed for a concrete failure mode using the 0.4 reduction factor.

†$0.75\phi N_{cb} < \phi N_{sa}$; therefore, this is a nonductile design.

Using the provisions of ACI 318-08, the steel design strength is greater (26,361 lb), resulting in the concrete breakout strength controlling the design. Since the 0.75 seismic reduction factor is no longer applied to the steel design strength in ACI 318-08, the concrete failure mode strengths (N_{cb}, N_{pn}, and N_{sb}) have to be greater than the steel design strength for ductile design. If brittle concrete failure is to control the seismic design, the concrete design strengths have to be reduced by the 0.4 seismic factor. Significant strength reductions occur if concrete controls.

4.2—Example 2: Single hooked anchor away from edges subjected to seismic tension

Check the capacity of a single anchor, 1 in. diameter, ASTM F1554 Grade 36 standard hooked bolt (L-bolt) installed in the top of a 12 in. thick foundation without edge effects to resist a factored load of 17,000 lb tension. The embedment depth is 8 in. and the concrete compressive strength is 6000 psi, as shown in Fig. 4.2. The structure supported on the foundation slab in this example is located in an area of high seismic risk. Structural loads on the foundation were calculated based on a seismic analysis. Assume normalweight concrete and that a crack forms in the plane of the anchor.

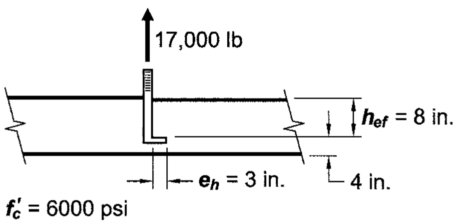


Fig. 4.2—Example 2: Single hooked anchor in tension away from edges. (Note: Foundation reinforcement not shown for clarity.)

Step	Calculations and discussion	ACI 318-05 Section
1.	The factored design load is: N_{ua} = 17,000 lb	 9.2.1
2.	General requirement for anchor strength: $\phi N_n \geq N_{ua}$ where ϕN_n is the lowest design strength from all appropriate failure modes in tension as determined from consideration of ϕN_{sa}, ϕN_{cb}, ϕN_{pn}, and ϕN_{sb}. In regions of moderate or high seismic risk, the design strength of anchors is limited to $0.75\phi N_n$.	 Eq. (D-1) D.4.1.2 D.3.3.3

3.	Check strength of steel anchor and requirements for ductility. Since the anchor is located in a region of moderate or high seismic risk, the anchor strength should be governed by tensile or shear strength of a ductile steel element. ASTM F1554 Grade 36 material meets the requirements of the ductile steel element definition in D.1 (tensile elongation of at least 14% and reduction of area of at least 30%). ASTM F1554 material has 23% elongation in 2 in. of length with 40% reduction in area (Table A.1). The design strength for the steel anchor in regions of moderate or high seismic risk is: $0.75\phi N_{sa} \geq N_{ua}$ and ϕN_{sa} < capacity of concrete breakout, pullout, side-face blowout, or splitting failure in tension to satisfy ductility requirements where	D.3.3.4
	$\phi = 0.75$	D.4.4(a)
	and the steel bolt strength is:	
	$N_{sa} = nA_{se} f_{uta}$	Eq. (D-3)
	where the minimum value for f_{uta} = 58,000 psi is given for ASTM F1554 Grade 36 material (Table A.1). Note that f_{uta} should not be taken greater than the smaller of $1.9f_{ya}$ and 125,000 psi. $1.9f_{ya}$ = 1.9(36,000) = 68,400 psi > 58,000 psi – OK n = 1 (single anchor) ANSI/ASME B1.1 defines A_{se}:	D.5.1.2
	$A_{se} = \frac{\pi}{4}\left(d_o - \frac{0.9743}{n_t}\right)^2$	RD.5.1.2
	Substituting 1 in. for d_o and 8 (number of threads) for n_t, A_{se} = 0.606 in.^{2} N_{sa} = (1)(0.606 in.^{2})(58,000 psi) = 35,148 lb ϕN_{sa} = (0.75)35,148 lb = 26,361 lb Reduction for regions of moderate or high seismic risk	
	$0.75\phi N_{sa}$ = 0.75(26,361 lb) = 19,771 lb	D.3.3.3
	19,771 lb > 17,000 lb (requirement of $0.75\phi N_{sa} > N_{ua}$ is met)	

4. Using the embedment depth h_{ef} = 8 in., check the various concrete failure modes. The basic capacity for nominal concrete breakout strength in tension for a single anchor in regions of moderate or high seismic risk is:

$0.75\phi N_{cb} \geq N_{ua}$ and $\phi N_{cb} > \phi N_{sa}$ (to ensure that the steel failure mode governs)

where

ϕ = 0.70 (for concrete breakout)
ϕ = 0.75 (for ductile steel in tension) D.4.4

Determine the concrete breakout strength assuming Condition B (no supplementary reinforcement has been provided):

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

where

A_{Nc}/A_{Nco} = 1.0 for single anchor remote from edges (a complete breakout prism is developed and no reduction is required) RD.5.2.1
$\psi_{ed,N}$ = 1.0 since anchor is remote from edges ($c_{a,min} \geq 1.5h_{ef}$) D.5.2.5
$\psi_{c,N}$ = 1.0 since for this example assumed cracking occurs in the concrete at service load levels D.5.2.6
$\psi_{cp,N}$ = 1.0 (cast-in anchors) D.5.2.7

The basic concrete breakout strength of a single anchor in tension is:

$$N_b = k_c\sqrt{f'_c}\,h_{ef}^{1.5}$$ Eq. (D-7)

where k_c = 24 for cast-in anchors.

For this example, h_{ef} = 8 in., which is less than 11 in., so Eq. (D-8), $N_b = 16\sqrt{f'_c}\,h_{ef}^{5/3}$, does not apply.

Substituting:

$$N_b = 24\sqrt{6000}(8)^{1.5} = 42{,}065 \text{ lb}$$

and

ϕN_{cb} = 0.7(42,065 lb) = 29,445 lb

Reduction for regions of moderate or high seismic risk:

$0.75\phi N_{cb}$ = 0.75(29,445 lb) = 22,084 lb D.3.3.3

22,084 lb $\geq N_{ua}$ and 22,084 lb > $0.75\phi N_{sa}$ (or 19,771 lb)

Therefore, the anchor strength remains governed by ductile steel failure.

5.	Check the anchor pullout strength: To prevent a pullout failure mode, the requirement for pullout strength of the hooked bolt checks the bearing stress inside the hook. This failure mode is initiated by crushing of the concrete: $0.75\phi N_{pn} \geq N_{ua}$ and $\phi N_{pn} > \phi N_{sa}$ (to ensure ductile steel failure mode governs) where $\phi = 0.70$ $N_{pn} = \psi_{cp} N_p$ where $N_p = 0.9 f_c' e_h d_o$ $\psi_{c,P} = 1.0$ considering concrete cracking at service loads $e_h = 3$ in. $d_o = 1$ in. and the hook length falls within the range $3d_o \leq e_h \leq 4.5d_o$ Substituting: $N_p = 0.9(6000 \text{ psi})(3 \text{ in.})(1 \text{ in.}) = 16{,}200$ lb and $\phi N_{pn} = 0.7(16{,}200 \text{ lb}) = 11{,}340$ lb Reduction for regions of moderate or high seismic risk $0.75\phi N_{pn} = 0.75(11{,}340 \text{ lb}) = 8505$ lb $0.75\phi N_{pn} < N_{ua}$ and $\phi N_{pn} < \phi N_{sa}$ Strength requirements are not met. Anchor pullout strength is less than the factored load and less than the steel capacity due to pullout capacity of the hook. Extending the hook will increase the capacity but the hook length is limited to 4.5 times anchor diameter or 4-1/2 in. Extend hook to the maximum allowed amount. Substituting 4-1/2 in. for e_h, recalculate N_p and ϕN_{pn} $N_p = 0.9(6000 \text{ psi})(4.5 \text{ in.})(1 \text{ in.}) = 24{,}300$ lb and $\phi N_{pn} = 0.7(24{,}300 \text{ lb}) = 17{,}010$ lb Reduction for regions of moderate or high seismic risk $0.75\phi N_{pn} = 0.75(17{,}010 \text{ lb}) = 12{,}757$ lb Anchor pullout strength with seismic reduction is less than factored load, 17,000 lb, and less than the steel capacity, 19,771 lb.	D.5.3 D.4.4(c)ii Eq. (D-14) Eq. (D-16) D.5.3.6 D.5.3.5 D.3.3.3 D.5.3.5 D.3.3.3
6.	Evaluate concrete side-face blowout failure mode: Since this anchor is located far from a free edge of concrete ($c_{a1} \geq 0.4h_{ef}$), concrete side-face blowout failure mode is not applicable; however, Section D.5.4 for side-face blowout only applies to headed anchors.	D.5.4

7. Evaluate splitting failure:

This type of failure can occur in thin slabs where the anchors have torque applied. The minimum edge distance for cast-in anchors with torque applied is $6d_a$, where d_a is the bolt diameter. This failure mode is not applicable because the anchor is remote from an edge. D.8.2

8. *Summary:*

Table 4.3—Summary of design strengths and controlling failure mode

Failure mode	Anchor design strength	Calculated design strength, lb	Controlling failure mode
Steel	$0.75\phi N_{sa}$	19,771	—
Concrete breakout	$0.75\phi N_{cb}$	22,084	—
Concrete pullout	$0.75\phi N_{pn}$	12,757	← Controls
Concrete side-face blowout	Not applicable	Not applicable	—
Concrete splitting	Not applicable	Not applicable	—

9. The ASTM F1554 Grade 36, 1 in. diameter hooked bolt with an 8 in. embedment using the maximum allowed effective hook length of 4-1/2 in. with the 0.75 seismic reduction is inadequate to resist the 17,000 lb tension factored load as shown in Table 4.3 and cannot be used in a moderate to high seismic region because the ductile steel failure mode does not govern the capacity. In a nonseismic or low seismic region, the hooked bolt with 4-1/2 in. hook is adequate to resist the factor load of 17,000 lb with the pullout strength (17,010 lb) being the governing mode of failure.

This is a nonductile anchor; if it had to be used with a current geometrical condition, the capacity of the concrete would have to be reduced per ACI 318-08, D.3.3.6 requirements.

If the provisions of ACI 318-08 are followed, the results would be those shown in Table 4.4.

Table 4.4—Summary of design strengths and controlling mode using ACI 318-08 Appendix D

Failure mode	Anchor design strength (D.3.3.3 for concrete failure modes)	Calculated design strength, lb	Reduced permissible design strength for nonductile failure modes (D.3.3.6)	Calculated permissible design strength,* lb
Steel	ϕN_{sa}	26,361	—	—
Concrete breakout	$0.75\phi N_{cb}$	22,084	—	—
Concrete pullout	$0.75\phi N_{pn}$	12,757 (controls†)	$0.4\ (0.75\phi N_{pn})$	5102
Concrete side-face blowout	Not applicable	Not applicable	—	—
Concrete splitting	Not applicable	Not applicable	—	—

*Since the concrete pullout strength cannot be increased, then as an alternate to the ductile design requirements of D.3.3.4 and D.3.3.5, reduced permissible design strength is allowed for a concrete failure mode using the 0.4 reduction factor.
†$0.75\phi N_{pn} < \phi N_{sa}$; therefore, this is a nonductile design.

Using the provisions of ACI 318-08, the steel design strength increases to 26,361 lb, as shown in Table 4.4, but the reduced permissible design strength controlled by concrete pullout drops to 5102 lb. The 0.75 seismic reduction factor is not applied to the steel design strength in ACI 318-08, so the concrete failure mode strengths should be larger to avoid being reduced by the 0.4 factor. Significant strength reductions occur if concrete controls.

4.3—Example 3: Single post-installed anchor in tension away from edges

Determine the minimum diameter of a post-installed torque-controlled expansion anchor for installation in the bottom of an 8 in. slab with a concrete compressive strength of f_c' = 4000 psi to support a 3000 lb service dead load, as shown in Fig. 4.3. The anchor will be in the tension zone (cracking at service load level is assumed), away from edges and other anchors in normal-weight concrete.

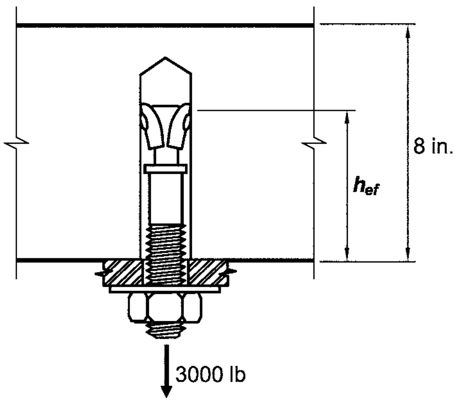


Fig. 4.3—Example 3: Single post-installed anchor in tension away from edges.

Refer to Table A.3 of this document for sample anchor installation and performance data. The data in Table A.3 are not from any specific anchor and should not be used for design in accordance with ACI 318-05, Appendix D. However, it is similar to what would be expected from testing and an evaluation report prepared by an independent testing and evaluation agency for the manufacturer in accordance with ACI 355.2.

Note: This example demonstrates code provisions and is not intended to promote the use of a single anchor to resist a tension load. The use of two anchors or providing an attachment that has multiple anchors to resist the load will provide redundancy in the design.

Step	Calculations and discussion	ACI 318-05 Section
1.	Determine factored design load: $N_{ua} = 1.4D$ N_{ua} = 1.4(3000) = 4200 lb	 9.2.1
2.	Determine fastener material: Select a qualified post-installed torque-controlled expansion anchor with a bolt conforming to the requirements of ASTM F1554 Grade 55. Design information resulting from anchor prequalification testing according to Table 4.2 of ACI 355.2-04, for the selected torque-controlled expansion anchor, is given in Table A.3 of this guide. For ASTM F1554 Grade 55: f_{uta} = 75,000 psi f_{ya} = 55,000 psi Elongation at 2 in. = 21% minimum Reduction of area = 30% minimum ACI 318 Appendix D requires 14% minimum elongation and 30% minimum reduction of area to qualify as a ductile steel anchor.	 D.1

3.	Steel strength requirement under tension loading:	D.5.1
	$\phi N_n \geq N_{ua}$ = 4200 lb	D.4.1.1
	where ϕN_n is the lowest of ϕN_{sa}, ϕN_{pn}, ϕN_{sb}, and ϕN_{cb}.	D.4.1.2
	$N_{sa} = n\,A_{se}\,f_{uta}$	Eq. (D-3)
	$f_{uta} \leq$ the smaller of $1.9f_{ya}$ and 125,000 psi	D.5.1.2
	$\phi n A_{se} f_{uta} \geq N_{ua}$ = 4200 lb	
	ϕ = 0.75 (for ductile steel)	D.4.4
	n = 1 (single anchor)	
	Substituting and solving for A_{se}:	
	$A_{se} \geq 4200/(\phi\, f_{uta}) = 4200/(0.75 \times 75{,}000)$	
	$A_{se} \geq 0.0747$ in.^{2} (minimum effective cross-sectional area of anchor)	
	$\therefore$ Bolt diameter $\geq$ 3/8 in. (A_{se} = 0.0775 in.^{2})	

4.

Concrete breakout strength requirement for tension loading: D.5.2

$\phi N_{cb} \geq N_{ua} \geq 4200$ lb Eq. (D-1)

$$N_{cb} = \frac{A_{Nc}}{A_{Nco}}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b \quad \text{Eq. (D-4)}$$

where

$$N_b = k_c\sqrt{f'_c}\,h_{ef}^{1.5} \quad \text{Eq. (D-7)}$$

Substituting:

$$\phi\frac{A_{Nc}}{A_{Nco}}\psi_{ed,N}\psi_{c,N}\psi_{cp,N} \times k_c\sqrt{f'_c}\,h_{ef}^{1.5} \geq 4200 \text{ lb}$$

where

$$\frac{A_{Nc}}{A_{Nco}} = 1 \text{ (for a single anchor)}$$

k_c = 17 (for a post-installed anchor where cracking is expected)
$\psi_{ed,N}$ = 1.0 (for no edge effects) D.5.2.5
$\psi_{c,N}$ = 1.0 (assuming cracking at service loads) D.5.2.6
$\psi_{cp,N}$ = 1.0 (for post-installed anchors intended for use in cracked concrete)
ϕ = 0.65 (for Category 1 and Condition B, no supplemental reinforcement)
ϕ = 0.55 (for Category 2 and Condition B, no supplemental reinforcement) D.4.4(c)ii

Note: An anchor category is established independently for each anchor diameter from the reliability tests of ACI 355.2. Smaller diameter anchors are, in general, more sensitive to the conditions established for reliability tests, including reduced torque, variations in hole diameter, and cracks in the concrete.

Assuming a Category 1 anchor, solve for the required embedment h_{ef} corresponding to the desired concrete breakout strength:

$$h_{ef} \geq \left[\frac{4200}{0.65 \times 1 \times 1 \times 1 \times 17 \times \sqrt{4000}}\right]^{2/3} = 3.3 \text{ in.}$$

If we instead assume a Category 2 anchor, the required embedment becomes:

$$h_{ef} \geq \left[\frac{4200}{0.55 \times 1 \times 1 \times 1 \times 17 \times \sqrt{4000}}\right]^{2/3} = 3.7 \text{ in.}$$

for a Category 2 anchor for breakout strength.

For this example, Table 4.5 provides combinations of acceptable anchors from Table A.3 that satisfies the minimum embedment criteria for concrete breakout strength in tension.

Table 4.5—Acceptable anchor diameters and embedments

Diameter, in.	Suitable embedments, in.	Category	Minimum h_{ef}, in.
3/8	4.5	2	3.7
1/2	5.5	2	3.7
5/8	4.5, 6.5	1	3.3
3/4	3.5, 5, 8	1	3.3

5. Pullout strength: D.5.3

For static loading, pullout strength is established by reference tests in cracks and by the crack movement reliability test of ACI 355.2. For post-installed anchors, data from the anchor prequalification testing should be used.

$\phi N_{pn} \geq N_{ua} = 4200$ lb Eq. (D-1)

$N_{pn} = \psi_{cp} N_p$ Eq. (D-14)

where

$\psi_{c,p} = 1.0$ D.5.3.6
$\phi = 0.65$ (for Category 1 and Condition B)
$\phi = 0.55$ (for Category 2 and Condition B) D.4.4(c)ii

From the evaluation report (Table A.3), the 5% fractile capacities shown in Table 4.6 were established for the combinations of anchor diameter, anchor categories, and embedment to check the required design concrete pullout strength using the above equation:

Table 4.6—Five percent fractile (pullout strength) for anchor diameter and embedment combinations

Diameter, in.	Embedment, in.	Anchor pullout capacity, lb	
3/8	4.5	$\phi N_{pn} = 0.55 \times 5583 = 3070 < 4200$	No good
1/2	5.5	$\phi N_{pn} = 0.55 \times 7544 = 4149 < 4200$	No good
5/8	4.5	$\phi N_{pn} = 0.65 \times 8211 = 5337 > 4200$	OK
5/8	6.5	$\phi N_{pn} = 0.65 \times 14{,}254 = 9265 > 4200$	OK
3/4	3.5	$\phi N_{pn} = 0.65 \times 5632 = 3661 < 4200$	No good
3/4	5	$\phi N_{pn} = 0.65 \times 9617 = 6251 > 4200$	OK
3/4	8	$\phi N_{pn} = 0.65 \times 19{,}463 = 12{,}651 > 4200$	OK

Try a 5/8 in. diameter anchor with $h_{ef} = 4.5$ in. from the selected anchor system.

6. For a post-installed torque-controlled expansion anchor with a 5/8 in. diameter and embedment depth of 4.5 in., check all failure modes:

a) For steel strength: D.5.1

$\phi N_{sa} = \phi n A_{se} f_{uta} \geq N_{ua} = 4200$ lb

$0.75 \times 1 \times 0.2260 \times 75{,}000 = 12{,}712$ lb > 4200 lb – OK

b) For concrete breakout strength: D.5.2

$$\phi N_{cb} = \phi \frac{A_{Nc}}{A_{Nco}} \psi_{ed,N} \psi_{cp,N} k_c \sqrt{f_c'}\, h_{ef}^{1.5} \geq N_{ua} = 4200 \text{ lb}$$

$\phi N_{cb} = 0.65 \times 1.0 \times 1.0 \times 1.0 \times 17 \times \sqrt{4000} \times 4.5^{1.5} = 6671$ lb > 4200 lb – OK

c) For pullout strength: D.5.3

$\phi N_{pn} \geq N_{ua} = 4200$ lb (value obtained from Table A.3, manufacturer's evaluation report)

$0.65 \times 8211 = 5337$ lb > 4200 lb – OK

7. Check for minimum concrete thickness: D.8.5
To prevent splitting, the thickness of the concrete in which the anchor is embedded should be at least $1.5h_{ef}$.

$1.5 \times 4.5 = 6\text{-}3/4$ in. < 8 in. – OK

8. *Summary:*

Table 4.7 summarizes the various design strengths with concrete pullout controlling.

Table 4.7—Summary of design strengths

Failure mode	Anchor design strength	Calculated design strength, lb	Controlling failure mode
Steel	ϕN_{sa}	12,712	—
Concrete breakout	ϕN_{cb}	6671	—
Concrete pullout	ϕN_{pn}	5337	← Controls

9. Final recommendation:

Use a post-installed torque-controlled expansion anchor with a 5/8 in. diameter and an embedment depth of 4-1/2 in. meeting the requirements of ASTM F1554 Grade 55.

4.4—Example 4: Group of headed studs in tension near an edge

Design a group of four AWS D1.1 Type B welded headed studs spaced 6 in. on center each way and concentrically loaded with a 10,000 lb service dead load. The anchor group is to be installed in the bottom of an 8 in. thick normalweight concrete slab made with the centerline of the connection 6 in. from a free edge of the slab, as shown in Fig. 4.4.

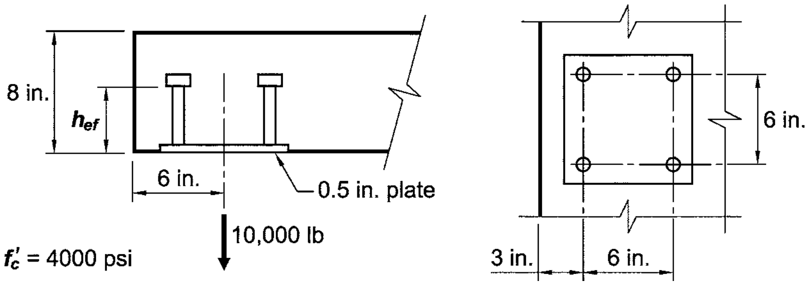
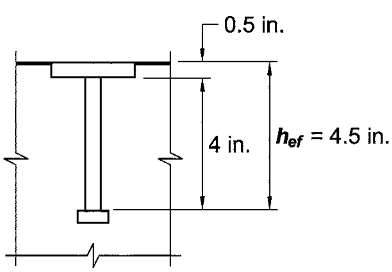


Fig. 4.4—Example 4: Group of headed studs in tension near an edge. (Note: reinforcement not shown for clarity.)

Step	Calculations and discussion	ACI 318-05 Section
1.	Determine factored design load: N_{ua} = 1.4(10,000) = 14,000 lb	9.2

2.	Determine anchor diameter:	D.5.1
	Using AWS D1.1 Type B welded headed studs.	
	Assume tension steel failure controls.	
	The basic requirement for the anchor steel is:	
	$\phi N_{sa} \geq N_{ua}$	Eq. (D-1) D.4.1.2
	where	
	$\phi = 0.75$	D.4.4(a)i
	Note: Per the ductile steel element definition in D.1, AWS D1.1, Type B studs qualify as a ductile steel element (20% minimum elongation in 2 in., which is greater than the 14% required and a minimum reduction in area of 50% that is greater than the 30% required [Table A.1]).	
	$N_{sa} = nA_{se} f_{uta}$	Eq. (D-3)
	For design purposes, Eq. (D-1) with Eq. (D-3) may be rearranged as:	
	$A_{se} \geq \dfrac{N_{ua}}{\phi n f_{uta}}$	
	where	
	$N_{ua} = 14{,}000$ lb $\phi = 0.75$ $n = 4$ $f_{uta} = 65{,}000$ psi	
	Note: Per Section D.5.1.2, f_{uta} *should not be taken greater than 1.9*f_{ya} *or 125,000 psi. For AWS D1.1 headed studs, 1.9*f_{ya} *= 1.9(51,000 psi) = 96,900 psi; therefore, use the specified minimum* f_{uta} *of 65,000 psi.*	D.5.1.2
	Substituting:	
	$A_{se} \geq \dfrac{14{,}000}{0.75(4)(65{,}000)} = 0.072 \text{ in.}^2$	
	Per Table A.2(b), 1/2 in. diameter welded headed studs will satisfy this requirement ($A_{se} = 0.196 \text{ in.}^2$).	
	Note: Per AWS D1.1 Table 7.1, Type B welded studs applies to studs in 1/2 in., 5/8 in., 3/4 in., 7/8 in., and 1 in. diameters. Although individual manufacturers may list smaller diameters, they are not explicitly covered by AWS D1.1.	
	The total design steel strength of four 1/2 in. headed studs:	
	$\phi N_{sa} = 0.75(4)(65{,}000)(0.196) = 38{,}220$ lb	

3.	Determine the required embedment length h_{ef} based on concrete breakout in tension:	D.5.2
	Two different equations are given for calculating concrete breakout strength; for single anchors, Eq. (D-4) applies, and for anchor groups, Eq. (D-5) applies. An anchor group is defined as:	
	"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."	D.1
	Since the spacing between anchors is 6 in., the anchors should be treated as a group if the embedment depth exceeds 2 in. Although the embedment depth is unknown, at this point it will be assumed that the provisions for an anchor group will apply.	
	The basic requirement for embedment of a group of anchors is:	
	$\phi N_{cbg} \geq N_{ua}$	Eq. (D-1) D.4.1.2
	where	
	$\phi = 0.70$	D.4.4(c)ii
	for anchors governed by concrete breakout and Condition B applies, since no supplementary reinforcement has been provided (for example, hairpin-type reinforcement that ties the failure prism into the structural member).	RD.4.4
	$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$	Eq. (D-5)
	Since this connection is likely to be affected by both group effects and edge effects, the embedment length h_{ef} is difficult to solve directly. In this case, an embedment length should be assumed at the outset and then proven to satisfy the requirement of Eq. (D-1).	
	Note: Welded studs are generally available in fixed lengths. Available lengths may be determined from manufacturers' catalogs. For this example, one manufacturer's product line indicates a length of 4 in. (after welding) for a standard 1/2 in. headed stud.	
	The effective embedment depth h_{ef} = 4 in. + 0.5 in. = 4.5 in.	
	Note: The effective embedment length h_{ef} *for the welded stud anchor is the shank length of the stud (4 in.) plus the thickness of the embedded plate (0.5 in.) as shown in Fig. 4.5.*	
		
	Fig. 4.5—Example 4: Effective embedment length h_{ef}.	
	Evaluate the terms in Eq. (D-5) with h_{ef} = 4.5 in. Determine A_{Nc} and A_{Nco} for the anchor group:	D.5.2.1
	A_{Nc} is the projected area of the failure surface as approximated by a rectangle with edges bounded by $1.5h_{ef}$ (in this case, 1.5 × 4.5 = 6.75 in.) and free edges of the concrete from the centerlines of the anchors as shown in Fig. 4.6.	
	A_{Nc} should not be taken greater than nA_{Nco}, where n = number of anchors in a group.	

3. (cont.)

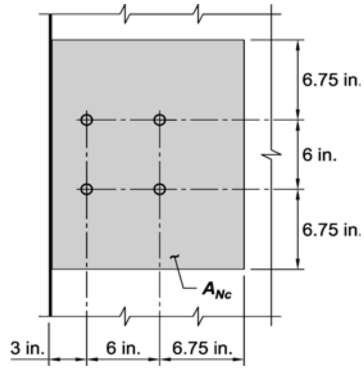


Fig. 4.6—Example 4: Projected concrete failure area A_{Nc}.

A_{Nc} = (3 + 6 + 6.75)(6.75 + 6 + 6.75) = 307 in.^{2}

$A_{Nco} = 9h_{ef}^2 = 9(4.5)^2$ = 182 in.^{2} (projected area of failure for one anchor without edge effects) — Eq. (D-6)

Check $A_{Nc} \leq nA_{Nco}$:

307 in.^{2} < 4(182 in.^{2}) = 728 in.^{2} – OK — D.5.2.1

Determine $\psi_{ec,N}$: — D.5.2.4

$\psi_{ec,N}$ = 1.0 (no eccentricity in the connection)

Determine $\psi_{ed,N}$: — D.5.2.5

Since $c_{a1} = c_{a,min} < 1.5h_{ef}$ (*Note:* c_{a1} = 3 in.)

$$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}}$$ — Eq. (D-11)

$$\psi_{ed,N} = 0.7 + 0.3\frac{3.0}{1.5(4.5)} = 0.83$$

Determine $\psi_{c,N}$: — D.5.2.6

$\psi_{c,N}$ = 1.0 (for locations where concrete cracking is likely to occur [bottom of the slab] and cracking is controlled by flexural or confining reinforcement)

Determine $\psi_{cp,N}$: — D.5.2.7

Note that this factor only applies to post-installed anchors. Therefore, for cast-in-place anchors, $\psi_{cp,N}$ = 1.0

Determine N_b: — D.5.2.2

$$N_b = k_c\sqrt{f_c'}\,h_{ef}^{1.5}$$

where

k_c = 24 (for cast-in anchors)

$$N_b = k_c\sqrt{f_c'}\,h_{ef}^{1.5} = 24\sqrt{4000}\,(4.5)^{1.5} \text{ lb} = 14{,}490 \text{ lb}$$ — Eq. (D-7)

Substituting into Eq. (D-5):

$$N_{cbg} = \left[\frac{307}{182}\right](1.0)(0.83)(1.0)(14{,}490) = 20{,}287 \text{ lb}$$

3. (cont.)	Check if: $\phi N_{cbg} \geq N_{ua}$ Substituting: ϕN_{cbg} = (0.70) (20,287) = 14,201 lb 14,201 lb > 14,000 lb – OK Specify a 4 in. length for the welded headed studs with the 1/2 in. thick base plate.	 Eq. (D-1)
4.	Determine if welded stud head size is adequate for pullout: $\phi N_{pn} \geq N_{ua}$ where $\phi = 0.70$ *Note: Condition B applies in all cases when pullout strength governs.* $N_{pn} = \psi_{c,P} N_p$ where $N_p = 8A_{brg} f_c'$ $\psi_{c,P}$ = 1.0 (for locations where concrete cracking is likely to occur [for example, the bottom of the slab] and cracking is controlled by flexural or confining reinforcement) For design purposes, Eq. (D-1) with Eq. (D-14) and Eq. (D-15) may be rearranged as: $A_{brg} \geq \dfrac{N_{ua}}{\phi \psi_{c,P} 8 f_c'}$ For the group of four studs, the individual factored tension load N_{ua} on each stud is: $N_{ua} = \dfrac{14{,}000}{4} = 3500$ lb Substituting: $A_{brg(required)} \geq \dfrac{3500}{0.70(1.0)(8)(4000)} = 0.156$ in.^{2} The bearing area of welded headed studs should be determined from manufacturers' catalogs. Figure 7.1 from AWS D1.1 lists the diameter of the head for a 1/2 in. diameter stud is 1 in. AWS states that alternate head configurations may be used with proof of full-strength development of design. Procedures in ACI 318 Appendix D are used to confirm alternate head configurations. $A_{brg(provided)} = \dfrac{\pi}{4}((1.0)^2 - (0.5)^2) = 0.589$ in.^{2} > 0.156 in.^{2} – OK The total design pullout strength of four 1/2 in. headed studs: $\phi n N_{pn}$ = 0.7(4)(8)(1.0)(0.589)(4000) = 52,774 lb	D.5.3 Eq. (D-1) D.4.1.2 D.4.4(c)ii Eq. (D-14) Eq. (D-15) D.5.3.6
5.	Evaluate side-face blowout: Consider need for side-face blowout when the edge distance from the centerline of an anchor to the nearest free edge is less than $0.4h_{ef}$. For this example: $0.4h_{ef}$ = 0.4(4.5) = 1.8 in. < 3 in. actual edge distance ∴ The side-face blowout failure mode is not applicable.	D.5.4

6. (cont.)	Required edge distances, spacing, and thickness to preclude splitting failure: Since a welded headed anchor is not torqued, the minimum cover requirements of ACI 318, Section 7.7 apply. Per Section 7.7, the minimum clear cover for a 1/2 in. bar not exposed to earth or weather is 3/4 in., which is less than the 2-3/4 in. cover provided – OK.	D.8 7.7.1(c)
7.	*Summary of connection strength*: Table 4.8 summarizes the various design strengths with concrete breakout controlling.	

Table 4.8—Summary of design strengths

Failure mode	Anchor design strength	Calculated design strength, lb	Controlling failure mode
Steel	ϕN_{sa}	38,220	—
Concrete breakout	ϕN_{cb}	14,201	← Controls
Concrete pullout	ϕN_{pn}	52,774	—
Concrete side-face blowout	ϕN_{sb}	Not applicable	
Concrete splitting	Not applicable	Not applicable	—

Use four 1/2 in. diameter welded studs meeting AWS D1.1 Type B with an effective embedment of 4-1/2 in.

4.5—Example 5: Single headed bolt in shear near an edge

Determine the reversible service wind load shear capacity for a single 1/2 in. diameter cast-in, hex-headed bolt meeting ASTM F1554 Grade 36. The headed bolt is installed in a normalweight continuous concrete foundation with a 7 in. embedment and a 2-3/4 in. edge distance as shown in Fig. 4.7. No supplemental reinforcement is present.

Note: This is the minimum anchorage requirement at the foundation required by International Code Council (IBC) 2006 Section 2308.6 for conventional light-frame wood construction. The 2-3/4 in. edge distance represents a typical connection at the base of framed walls using wood 2 x 6 sill members.

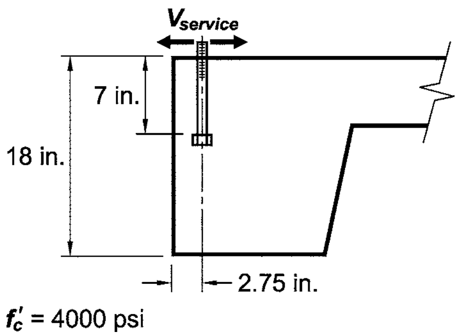


Fig. 4.7—Example 5: Single headed bolt in shear near an edge. (Note: Foundation reinforcement not shown for clarity.)

This problem provides the anchor diameter, embedment depth, and anchor material type. The designer is required to compute the maximum reversible service shear load due to wind.

Step	Calculations and discussion	ACI 318-05 Section
1.	Determine the controlling shear design strength ϕV_n based on the smaller of the steel strength, concrete, or both: Step 6 of this example provides the conversion of the controlling factored shear load V_{ua} to a service load due to wind using factored loads of ACI 318-05 Section 9.2.	

2.	Determine V_{ua}, as controlled by the steel strength of the anchor in shear: $\phi V_{sa} \geq V_{ua}$ where $\phi = 0.65$ Per the ductile steel element definition in D.1, ASTM F1554 Grade 36 steel qualifies as a ductile steel element (Table A.1). $V_{sa} = n0.6A_{se}f_{uta}$ To determine V_{ua} for the steel strength, Eq. (D-2) is combined with Eq. (D-20) to give: $V_{ua} = \phi V_{sa} = n0.6A_{se}f_{uta}$ where $\phi = 0.65$ $n = 1$ $A_{se} = 0.142$ in.^{2} (for a 1/2 in. threaded bolt, Table A.2(a)) $f_{uta} = 58{,}000$ psi. Per ASTM F1554, Grade 36 has a specified minimum yield strength f_{ya} of 36 ksi and a specified tensile strength f_{uta} of 58 ksi (Table A.1). For design purposes, the minimum tensile strength of 58 ksi should be used. *Note that* f_{uta} *should not be taken greater than 1.9*f_{ya} *or 125,000 psi. For ASTM F1554 Grade 36, 1.9*f_{ya} *= 1.9(36,000) = 68,400 psi; therefore, use the specified minimum* f_{uta} *of 58,000 psi.* Substituting, V_{ua} as controlled by steel strength is: $V_{ua} = \phi V_{sa} = 0.65(1)(0.6)(0.142)(58{,}000) = 3212$ lb	D.6.1 Eq. (D-2) D.4.1.1 D.4.4 Eq. (D-20)

3. Determine V_{ua} governed by concrete breakout strength with shear directed toward a free edge: D.6.2

$$\phi V_{cb} \geq V_{ua}$$ Eq. (D-2) D.4.1.2

where

$\phi = 0.70$ (for Condition B, no supplementary reinforcement has been provided) D.4.4(c)i

$$V_{cb} = \frac{A_{Vc}}{A_{Vco}} \psi_{ed,V} \psi_{c,V} V_b$$ Eq. (D-21)

where

$$\frac{A_{Vc}}{A_{Vco}} = \frac{3(2.75)(1.5)(2.75)}{4.5(2.75)^2} = 1.0$$ D.6.2.1

As shown in Fig. 4.8, the member thickness, 18 in., is greater than $1.5c_{a1}$ (4.125 in.) and the distance to an orthogonal edge, c_{a2}, is greater than $1.5c_{a1}$ because the foundation is continuous in both directions.

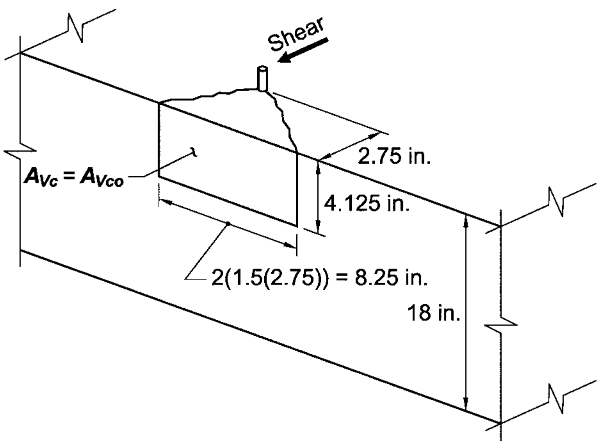


Fig. 4.8—Example 5: Projected concrete failure area A_{Vc}.

$\psi_{ed,V} = 1.0$ D6.2.6

$\psi_{c,V} = 1.0$ (for locations where concrete cracking is likely to occur [for example, at the foundation's edge, which is susceptible to cracks] and no supplemental reinforcement is provided or edge reinforcement is smaller than a No. 4 bar) D.6.2.7

$$V_b = 7\left(\frac{\ell_e}{d_o}\right)^{0.2} \sqrt{d_o}\sqrt{f_c'}(c_{a1})^{1.5}$$ Eq. (D-24)

where

ℓ_e = load-bearing length of the anchor for shear, not to exceed $8d_o$ D.6.2.2

$\ell_e = 8d_o = 8(0.5) = 4.0$ in. < 7.0 in.

For this problem, $8d_o$ will control since the embedment depth h_{ef} is 7 in.

To determine V_{ua} for the given embedment depth governed by concrete breakout strength, combine Eq. (D-2) with Eq. (D-20) and Eq. (D-23) to give:

$$V_{ua} = \phi V_{cb} = \phi \frac{A_{Vc}}{A_{Vco}} \psi_{ed,V} \psi_{c,V} 7\left(\frac{\ell_e}{d_o}\right)^{0.2} \sqrt{d_o}\sqrt{f_c'}(c_{a1})^{1.5}$$

Substituting, V_{ua} as controlled by concrete breakout strength is:

$$V_{ua} = \phi V_{cb} = 0.70(1.0)(1.0)1.0)7\left(\frac{4}{0.5}\right)^{0.2} \sqrt{0.5}\sqrt{4000}(2.75)^{1.5} = 1514 \text{ lb}$$

4. Determine V_{ua} governed by concrete pryout strength:

Note: The pryout failure mode typically controls only for stiff anchors installed at shallow embedments where the concrete breakout occurs behind the anchor in a direction opposite the shear force (Fig. RD.4.1(ii)). For this example, where the shear may be directed either toward the free edge or away from the free edge, the small edge distance may be the controlling value for pryout strength. — D.6.3

$\phi V_{cp} \geq V_{ua}$ — Eq. (D-2), D.4.1.2

where

$\phi = 0.70$ — D.4.4(c)i

Condition B applies in all cases when pryout strength governs

$V_{cp} = k_{cp}N_{cb}$ — Eq. (D-29)

where

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

$$N_{cb} = \frac{A_{Nc}}{A_{Nco}}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$$ — Eq. (D-4)

Evaluate the terms of Eq. (D-4) for this problem:

A_{Nc} is the projected concrete failure area on the surface as approximated by a rectangle with edges bounded by $1.5h_{ef}$ (in this case, 1.5[7] = 10.5) in a direction perpendicular to the shear force and the free edge of the concrete from the centerline of the anchor shown in Fig. 4.9.

$A_{Nc} = (2.75 + 10.5)(10.5 + 10.5) = 278$ in.^{2} — Eq. (D-6)

$A_{Nco} = 9h_{ef}^2 = 9(7.0)^2 = 441$

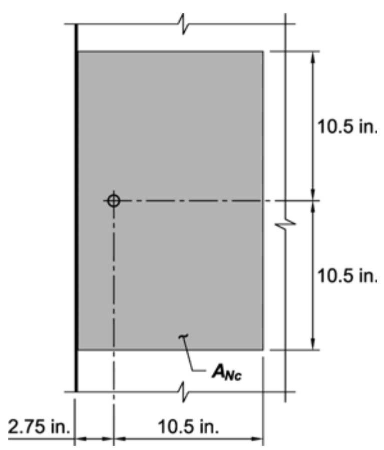


Fig. 4.9—Example 5: Projected concrete failure area A_{Nc}.

Determine $\psi_{ed,N}$: — D.5.2.5

$$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}}$$ — Eq. (D-11)

$$\psi_{ed,N} = 0.7 + 0.3\frac{2.75}{1.5(7.00)} = 0.78$$

Determine $\psi_{c,N}$: — D.5.2.6

$\psi_{c,N} = 1.0$ (for locations where concrete cracking is likely to occur [for example, the foundation's edge, which is susceptible to cracks] with cracking controlled by flexural reinforcement or confining reinforcement)

4. (cont.)	Determine $\psi_{cp,N}$: $\psi_{cp,N}$ = 1.0 (for cast-in-place anchors) Determine N_b for the anchor: $N_b = k_c\sqrt{f'_c}\,h_{ef}^{1.5}$ where k_c = 24 (for cast-in anchors) $N_b = 24\sqrt{4000}\,(7.0)^{1.5} = 28{,}112$ lb Substituting into Eq. (D-4): $N_{cb} = \left[\frac{278}{441}\right](0.78)(1.0)(1.0)(28{,}112) = 13{,}822$ lb To determine V_{ua} for the given embedment depth governed by pryout strength, Eq. (D-2) is combined with Eq. (D-29) to give: $V_{ua} = \phi V_{cp} = \phi k_{cp} N_{cb}$ Substituting V_{ua} for the embedment length governed by pryout strength is: $V_{ua} = \phi V_{cp} = 0.7(2.0)(13{,}822) = 19{,}350$ lb	D.5.2.7 D.5.2.2 Eq. (D-7)
5.	Required edge distances, spacings, and thickness to preclude splitting failure: Since a headed bolt used to attach wood frame construction is not likely to be torqued significantly, the minimum cover requirements of ACI 318-05 Section 7.7 apply. The minimum clear cover for a 1/2 in. bar is 1-1/2 in. when exposed to earth or weather. The clear cover provided for the bolt is 2-1/2 in. (2-3/4 in. to bolt centerline less one-half bolt diameter). Note that the bolt head will have less cover (2-3/16 in. for a hex head) – OK	D.8 7.7
6.	*Summary:* The factored shear load ($V_{ua} = \phi V_n$) based on the governing strength (steel, concrete breakout, and concrete pryout) is summarized as shown in Table 4.9:	

Table 4.9—Summary of design strengths

Failure mode	Anchor design strength	Calculated design strength, lb	Controlling failure mode
Steel	ϕV_{sa}	3212	—
Concrete breakout	ϕV_{cb}	1514	← Controls
Concrete pryout	ϕV_{cp}	19,350	—
Concrete splitting	Not applicable	Not applicable	—

The load factor for wind load is 1.6: (9.2.1)

$$V_{service} = \frac{\phi V_{cb}}{1.6} = \frac{1514}{1.6} = 946 \text{ lb}$$

The reversible service load shear strength from wind load of the International Code Council (ICC) 2003, Section 2308.6 minimum foundation connection for conventional wood-frame construction (1/2 in. diameter bolt embedded 7 in.) is 946 lb per bolt. The strength of the attached member (2 x 6 sill plate) should also be evaluated.

Note: This embedment strength is only related to the anchor being installed in concrete with a specified compressive strength of 4000 psi. In many cases, concrete used in foundations such as this is specified at 2500 psi, the minimum strength permitted by the code. Since the concrete breakout strength controlled the strength of the connection, a revised strength based on using 2500 psi concrete rather than the 4000 psi concrete used in the example, is determined as follows:

$$V_{service,\ f'_c = 2500\text{ psi}} = 946\frac{\sqrt{2500}}{\sqrt{4000}} = 747 \text{ lb}$$

4.6—Example 6: Single headed bolt in tension and shear near an edge

Determine if a single 1/2 in. diameter cast-in hex-headed bolt installed with a 7 in. embedment depth and a 2-3/4 in. edge distance in a normalweight, continuous concrete foundation is adequate for a service tension load from wind of 1000 lb and reversible service shear load from wind of 600 lb. No supplemental reinforcement is present, as shown in Fig. 4.10.

Note: This is an extension of Example 5, which includes a tension load on the anchor as well as a shear load.

ASTM F1554 Grade 36 hex-head anchor bolt material.

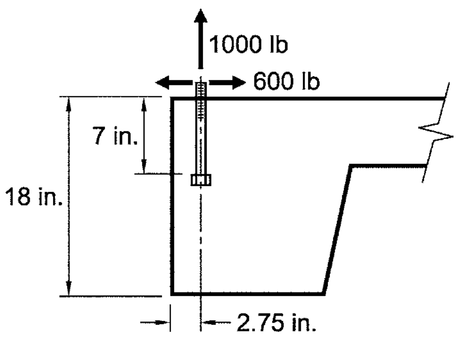


Fig. 4.10—Example 6: Single headed bolt in tension and shear near an edge. (Note: Foundation reinforcement not shown for clarity.)

Step	Calculations and discussion	ACI 318-05 Section
1.	Determine the factored design loads: N_{ua} = 1.6(1000) = 1600 lb V_{ua} = 1.6(600) = 960 lb	9.2
	This is a tension/shear interaction problem where values for both the design tensile strength (ϕN_n) and shear design strength (ϕV_n) will need to be determined. ϕN_n is the smallest of the design tensile strengths as controlled by steel (ϕN_{sa}), concrete breakout (ϕN_{cb}), pullout (ϕN_{pn}), and side-face blowout (ϕN_{sb}). ϕV_n is the smallest of the shear design strengths as controlled by steel (ϕV_{sa}), concrete breakout (ϕV_{cb}), and concrete pryout (ϕV_{cp}).	D.4.1.2

2.	Determine the design tensile strength ϕN_n: a) Steel strength ϕN_{sa}: $\phi N_{sa} = \phi n A_{se} f_{uta}$ where $\phi = 0.75$ $n = 1$ (for one anchor) Per the ductile steel element definition in D.1, ASTM F1554 Grade 36 steel qualifies as a ductile steel element. $A_{se} = 0.142$ in.^{2} (for 1/2 in. anchor bolt [Table A.2(a)]) $f_{uta} = 58{,}000$ psi (Table A.1) *Note:* f_{uta} *should not be taken greater than 1.9*f_{ya} *or 125,000 psi. For ASTM F1554 Grade 36, 1.9*f_{ya} *= 1.9(36,000) = 68,400 psi; therefore, use the specified minimum* f_{uta} *of 58,000 psi.* Substituting: $\phi N_{sa} = 0.75(1)(0.142)(58{,}000) = 6177$ lb b) Concrete breakout strength ϕN_{cb}: Since no supplementary reinforcement has been provided $\phi = 0.70$ In the process of calculating the pryout strength for this fastener in Example 5, Step 4, N_{cb}, for this anchor, was found to be 13,822 lb. Substituting: $\phi N_{cb} = 0.70(13{,}822) = 9675$ lb c) Pullout strength ϕN_{pn}: $\phi N_{pn} = \phi \psi_{c,P} N_p$ where $\phi = 0.70$ (Condition B applies in all cases when pullout strength governs) $\psi_{c,P} = 1.0$ (cracking may occur at the edges of the foundation) $N_p = 8 A_{brg} f_c'$ $A_{brg} = 0.291$ in.^{2} (for a 1/2 in. hex-head bolt, Table A.2(a)) Pullout strength ϕN_{pn}: $\phi N_{pn} = 0.7(1.0)(8)(0.291)(4000) = 6518$ lb d) Concrete side-face blowout strength ϕN_{sb}: The side-face blowout failure mode should be investigated when the edge distance c_{a1} is less than $0.4h_{ef}$. $0.4h_{ef} = 0.4(7) = 2.80$ in. > 2.75 in.; therefore, the side-face blowout strength should be determined $\phi N_{sb} = \phi(160 c_{a1} \sqrt{A_{brg}} \sqrt{f_c'})$ where $\phi = 0.70$ (no supplementary reinforcement has been provided) $c_{a1} = 2.75$ in. $A_{brg} = 0.291$ in.^{2} (for a 1/2 in. hex-head bolt, Table A.2(a))	D.5 D.5.1 Eq. (D-3) D.4.4(a)i D.5.1.2 D.5.2 D.4.4(c)ii D.5.3 Eq. (D-14) D.4.4(c)ii D.5.3.6 Eq. (D-15) D.5.4 D.5.4.1 Eq. (D-17) D.4.4(c)ii

2. (cont.)

Substituting:

$$\phi N_{sb} = 0.7(160(2.75)\sqrt{0.291}\sqrt{4000}\,) = 10{,}508 \text{ lb}$$

The summary of tension design strengths and the controlling strength is shown in Table 4.10.

Table 4.10—Summary of tension design strengths

Failure mode	Anchor design strength	Calculated design strength, lb	Controlling failure mode
Steel	ϕN_{sa}	6177	← Controls
Concrete breakout	ϕN_{cb}	9675	—
Concrete pullout	ϕN_{pn}	6518	—
Concrete side-face blowout	ϕN_{sb}	10,508	—

Check if $\phi N_n \geq N_{ua}$:

6177 lb > 1600 lb – OK — Eq. (D-1)

$\therefore\ \phi N_n = 6177$ lb

3.

Determine the shear design strength ϕV_n: — D.6

The summary of steel strength, concrete breakout strength, and pryout strength for anchor in shear from Example 5 is replicated for convenience here in Table 4.11:

Table 4.11—Summary of shear design strengths from Example 5

Failure mode	Anchor design strength	Calculated design strength, lb	Controlling failure mode
Steel	ϕV_{sa}	3212	—
Concrete breakout	ϕV_{cb}	1514	← Controls
Concrete pryout	ϕV_{cp}	19,350	—

Check $\phi V_n \geq V_{ua}$:

1514 lb > 960 lb – OK — Eq. (D-2)

$\therefore \phi V_n = 1514$ lb

4.

Check tension and shear interaction: — D.7

If $V_{ua} \leq 0.2\phi V_n$, then the full tension design strength is permitted — D.7.1

$V_{ua} = 960$ lb

$0.2\phi V_n = 0.2(1514) = 302$ lb < 960 lb

V_{ua} exceeds $0.2\phi V_n$, so the full tension design strength is not permitted.

If $N_{ua} \leq 0.2\phi N_n$ then the full shear design strength is permitted — D.7.2

$N_{ua} = 1600$ lb

$0.2\phi N_n = 0.2(6177) = 1235$ lb < 1600 lb

N_{ua} exceeds $0.2\phi N_n$, so the full shear design strength is not permitted.

The interaction equation should be used — D.7.3

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2$$ Eq. (D-31)

$$\frac{1600}{6177} + \frac{960}{1514} = 0.26 + 0.63 = 0.89 < 1.2 - \text{OK}$$

5.	Required edge distances, spacings, and thickness to preclude splitting failure: Since a headed bolt used to attach wood frame construction is not likely to be torqued significantly, the minimum cover requirements of ACI 318-05 Section 7.7 apply. The minimum clear cover for a 1/2 in. bar is 1-1/2 in. when exposed to earth or weather. The clear cover provided for the bolt is 2-1/2 in. (2-3/4 in. to bolt centerline less one-half bolt diameter). *Note: The bolt head will have less cover (2-3/16-in. for a hex head). Cover is considered adequate, that is, 2-3/4 in. to the centerline of the bolt results in the tip of the hex head close enough to the 1-1/2 in. clear clover to necessitate not moving the bolt axis farther away from the edge.*	D.8
6.	*Summary:* A single 1/2 in. diameter cast-in hex-headed bolt installed with a 7 in. embedment depth and a 2-3/4 in. edge distance in a concrete foundation is adequate for a service tension load from wind of 1000 lb and reversible service shear load from wind of 600 lb.	

4.7—Example 7: Single post-installed anchor in tension and shear near an edge

Determine if a single 1/2 in. diameter, post-installed, torque-controlled expansion anchor shown in Fig. 4.11, with a minimum 5-1/2 in. effective embedment, installed 3 in. from the edge of a continuous normalweight concrete footing (formed outer surface) is adequate for a service tension load of 1000 lb for wind and a reversible service shear load of 350 lb for wind. The anchor will be installed in the tension zone and the concrete is assumed to be cracked.

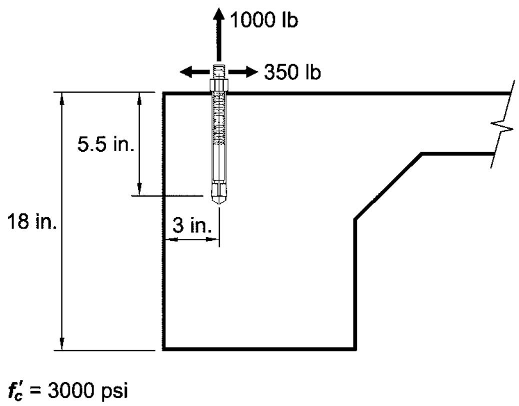


Fig. 4.11—Example 7: Single post-installed anchor in tension and shear near an edge. (Note: reinforcement not shown for clarity.)

Refer to Table A.3 for a sample table of post-installed anchor data from manufacturer (fictitious for example purposes only), as determined from testing in accordance with ACI 355.2.

Step	Calculations and discussion	ACI 318-05 Section
1.	Determine the factored tension and shear design loads: $N_{ua} = 1.6W = 1.6 \times 1000 = 1600$ lb $V_{ua} = 1.6W = 1.6 \times 350 = 560$ lb	9.2
2.	Design considerations: This is a tension/shear interaction problem where values for both ϕN_n and ϕV_n need to be determined. ϕN_n is the lesser of the tension design strength controlled by steel (ϕN_{sa}), concrete breakout (ϕN_{cb}), concrete side-face blowout (ϕN_{sb}), or concrete pullout (ϕN_{pn}). ϕV_n is the lesser of the shear design strength controlled by steel (ϕV_{sa}), concrete breakout (ϕV_{cb}), or pryout (ϕV_{cp}). Concrete side-face blowout requirements apply to cast-in and undercut anchors (RD.5.4) and are not considered here.	 D.4.1.2
3.	Evaluate anchor material: For this example, consider that the post-installed, torque-controlled expansion anchor is manufactured from carbon steel material conforming to the material requirements of ASTM F1554 Grade 55, which is a headed bolt ASTM specification. The data from anchor prequalification testing according to ACI 355.2 are shown in Table A.3. For ASTM F1554 Grade 55 material (Table A.1): $f_{uta} = 75{,}000$ psi $f_{ya} = 55{,}000$ psi Elongation at 2 in. = 21% minimum with a reduction of area = 30% minimum. Appendix D requires 14% minimum elongation and 30% minimum reduction of area for an anchor to be considered as a ductile steel element. $\therefore$ Anchor steel is ductile.	D.1

4.	Steel strength under tension loading: $\phi N_{sa} \geq N_{ua}$ $N_{sa} = nA_{se} f_{uta}$ $\phi nA_{se}f_{uta} \geq N_{ua} = 1600$ lb For ductile steel as controlling failure mode: $\phi = 0.75$ $n = 1$ (single anchor) Calculating for ϕN_{sa}: $\phi N_{sa} = 0.75 \times 1 \times 0.142 \times 75{,}000 = 7988$ lb > 1600 lb – OK ∴ 1/2 in. diameter anchor steel strength is adequate under tension loading.	D.5.1 D.4.1.1 Eq. (D-3) D.4.4(a)i
5.	Minimum edge distance requirements: The minimum edge distance for post-installed anchors should be based on the greater of the minimum cover requirements in ACI 318 Section 7.7 or minimum edge distance requirements for the products as determined by tests in accordance with ACI 355.2 (Table A.3), and should not be less than two times the maximum aggregate size. $c_{a,min}$ = (2 in. – maximum cover requirements for concrete exposed to earth; 2.5 in. – product requirement; or 2(0.75) = 1.5 in.) – assuming 3/4 in. maximum aggregate size) ∴ $c_{a,min}$ = 3 – 1/2(0.5 in.) = 2-3/4 in. – OK	 D.8.3

6.	Concrete breakout strength under tension loading:	D.5.2
	$\phi N_{cb} \geq N_{ua}$	D.4.1.1
	$N_{cb} = \frac{A_{Nc}}{A_{Nco}}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$	Eq. (D-4)
	where	
	$N_b = k_c\sqrt{f'_c}h_{ef}^{1.5}$	Eq. (D-7)
	Substituting:	
	$\phi N_{cb} = \phi\frac{A_{Nc}}{A_{Nco}}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}k_c\sqrt{f'_c}h_{ef}^{1.5} \geq N_{ua} = 1600$ lb	
	where	
	$k_c = k_{cr} = 17$ (Table A.3)	
	$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}}$ when $c_{a,min} < 1.5h_{ef}$	Eq. (D-11)
	by observation, $c_{a,min} < 1.5h_{ef}$	
	$\psi_{ed,N} = 0.7 + 0.3\frac{3}{1.5(5.5)} = 0.81$	
	$\psi_{c,N} = 1.0$ (assuming cracking at service loads, $f_t > f_r$)	D.5.2.6
	$\psi_{cp,N} = 1.0$ (assuming cracking at service loads, $f_t > f_r$)	D.5.2.7
	$\phi = 0.55$ (for a Category 2 anchor, no supplemental reinforcement provided)	D.4.4(c)ii
	$A_{Nco} = 9h_{ef}^2 = 9(5.5)^2$	Eq. (D-6)
	$\therefore A_{Nco} = 272.25$ in.^{2}	D.5.2.1
	$A_{Nc} = (c_{a1} + 1.5h_{ef})(2 \times 1.5h_{ef}) = (3.0 + 1.5(5.5))(2 \times (1.5)(5.5))$	Fig. RD. 5.2.1(b)
	$\therefore A_{Nc} = 185.63$ in.^{2}	
	$\frac{A_{Nc}}{A_{Nco}} = \frac{185.63}{272.25} = 0.68$	
	Calculating ϕN_{cb}:	
	$N_{cb} = 0.68 \times 0.81 \times 1.0 \times 17 \times \sqrt{3000} \times (5.5)^{1.5} = 6615$ lb	
	$\phi N_{cb} = 0.55(6615$ lb$) = 3638$ lb $> N_{ua} = 1600$ lb – OK	
7.	Pullout strength:	
	Pullout strength N_p for post-installed anchors is established by reference tests in cracked and uncracked concrete in accordance with ACI 355.2. Data from the anchor prequalification testing should be used.	D.5.3.2
	$N_p = N_{p,cr} = 7544$ lb (Table A.3).	
	$\phi N_{pn} \geq N_{ua}$	D.4.1.1
	$N_{pn} = \psi_{c,P}N_p$	Eq. (D-14)
	$\phi = 0.55$ (for a Category 2 anchor, Condition B applies in all cases when pullout strength governs)	D.4.4.(c)ii
	$\psi_{c,P} = 1.0$ (assuming cracking at service loads, $f_t > f_r$)	D.5.3.6
	$\phi N_{pn} = 0.5 \times 7544 = 4149 > 1600$ lb – OK	

8. Check all failure modes under tension loading: D.4.1.2

Summary: Table 4.12 shows that concrete breakout strength controls

$$\phi N_n = 3638 \text{ lb}$$

Table 4.12—Summary of tension design strengths

Failure mode	Anchor design strength	Calculated design strength, lb	Controlling failure mode
Steel	ϕN_{sa}	7988	
Concrete breakout	ϕN_{cb}	3638	← Controls
Concrete pullout	ϕN_{pn}	4149	—

9. Steel strength under shear loading: D.6.1

$$\phi V_{sa} \geq V_{ua}$$ D.4.1.1

$V_{sa} = n0.6A_{se}f_{uta}$ (anchor does not have sleeve to extend through shear plane) Eq. (D-20)

$$\phi n 0.6A_{se}f_{uta} \geq V_{ua} = 560 \text{ lb}$$

For ductile steel as controlling failure mode:

$$\phi = 0.65$$ D.4.4(a)ii

n = 1 (single anchor)

Calculating for ϕV_{sa}:

$$\phi V_{sa} = 0.65 \times 0.6 \times 0.142 \times 75{,}000 = 4153 \text{ lb} > 560 \text{ lb – OK}$$

∴ 1/2 in. diameter anchor steel strength is adequate under shear loading.

10.	Concrete breakout strength under shear loading:	D.6.2
	$\phi V_{cb} \geq V_{ua}$	D.4.1.1
	$V_{cb} = \frac{A_{Vc}}{A_{Vco}}\psi_{ed,V}\psi_{c,V}V_b$	Eq. (D-21)
	where	
	$V_{cb} = 7\left(\frac{\ell_e}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f_c'}(c_{a1})^{1.5}$	Eq. (D-24)
	Substituting:	
	$\phi V_{cb} = \phi\frac{A_{Vc}}{A_{Vco}}\psi_{ed,V}\psi_{c,V}7\left(\frac{\ell_e}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f_c'}(c_{a1})^{1.5} \geq V_{ua} \geq 560$ lb	
	where	
	ϕ = 0.7 (for anchors governed by concrete breakout due to shear; Condition B where no supplemental reinforcement is provided);	D.4.4(c)ii
	$\frac{A_{Vc}}{A_{Vco}}$ = 1.0; and c_{a1} = 3 in. (edge distance) c_{a2} is the distance from the center of the anchor to the edge of concrete in the direction orthogonal to c_{a1} (not specified in this example, but consider this distance greater than $1.5h_{ef}$)	
	$\psi_{ed,V}$ = 1.0 (since $c_{a2} \geq 1.5c_{a1}$)	Eq. (D-27)
	$\psi_{c,V}$ = 1.0 (assuming cracking at service loads, $f_t > f_r$)	D.6.2.7
	d_o = 0.5 in.	
	$\ell_e = h_{ef}$ = 5.5 in., but $\ell_e \leq 8d_o$; $8d_o = 8(0.5) = 4$ in.	
	$\therefore \ell_e$ = 4 in.	
	Substituting:	
	$\phi V_{cb} = 0.7 \times 1.0 \times 1.0 \times 1.0 \times 7\left(\frac{4}{0.5}\right)^{0.2} \times \sqrt{0.5} \times \sqrt{3000} \times (3)^{1.5} = 1495$ lb > 560 lb – OK	
11.	Concrete pryout strength:	D.6.3
	$\phi V_{cp} \geq V_{ua}$	D.4.1.1
	$V_{cp} = k_{cp}N_{cb}$	Eq. (D-29) D.6.3.1
	where	
	k_{cp} = 2.0 (since h_{ef} = 5.5 in. > 2.5 in.) N_{cb} = 6615 lb (refer to Step 6 of this design example) ϕ = 0.7 (Condition B applies)	D.4.4(c)i
	$\therefore \phi V_{cb} = 0.7 \times 2.0 \times 6615 = 9261$ lb > 560 lb – OK	

12. Check all failure modes under shear loading: D.4.1.2

Table 4.13 shows that concrete breakout strength controls.

Table 4.13—Summary of shear design strengths

Failure mode	Anchor design strength	Calculated design strength, lb	Controlling failure mode
Steel	ϕV_{sa}	4153	—
Concrete breakout	ϕV_{cb}	1495	← Controls
Concrete pryout	ϕV_{cp}	9261	—

13. Check interaction of tension and shear forces: D.7

If $V_{ua} \leq 0.2\phi V_n$, then the full strength in tension is permitted:

$\phi N_n \geq N_{ua}$ D.7.1

$0.2\phi V_n = 0.2(1495 \text{ lb}) = 299 \text{ lb} < 560 \text{ lb}$ – Requirement not met

If $N_{ua} \leq 0.2\phi N_n$, then the full strength in shear is permitted:

$\phi V_n \geq V_{ua}$ D.7.2

$0.2\phi N_n = 0.2(3638 \text{ lb}) = 728 \text{ lb} < 1600 \text{ lb}$ – Requirement not met

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

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2 \quad \text{Eq. (D-31)}$$

$$\frac{1600}{3638} + \frac{560}{1495} = 0.44 + 0.37 = 0.81 < 1.2 \text{ – OK}$$

14. *Summary:*

The post-installed, torque-controlled expansion anchor, 1/2-in. diameter at a 5-1/2 in. effective embedment depth, is adequate to resist the applied service tension and shear loads of 1000 lb and 350 lb, respectively.

4.8—Example 8: Group of cast-in anchors in tension and shear with two free edges and supplemental reinforcement

Check the capacity of a fastener group with four 3/4 in. diameter, ASTM F1554 Grade 55, cast-in anchor rods embedded 12 in. with hex nuts into the thickened slab as shown in Fig. 4.12(a) through (d). The concrete is 24 in. thick, f_c' = 3000 psi, and is normalweight. The anchor support is a combined factored load of 4000 lb shear and 12,000 lb tension. The plate is symmetrically placed at the corner. Seismic forces are not a consideration. Reinforcement is 60 ksi. Because both shear and tension are to be considered, the capacity of this detail will be based on the interaction equation given in D.7.

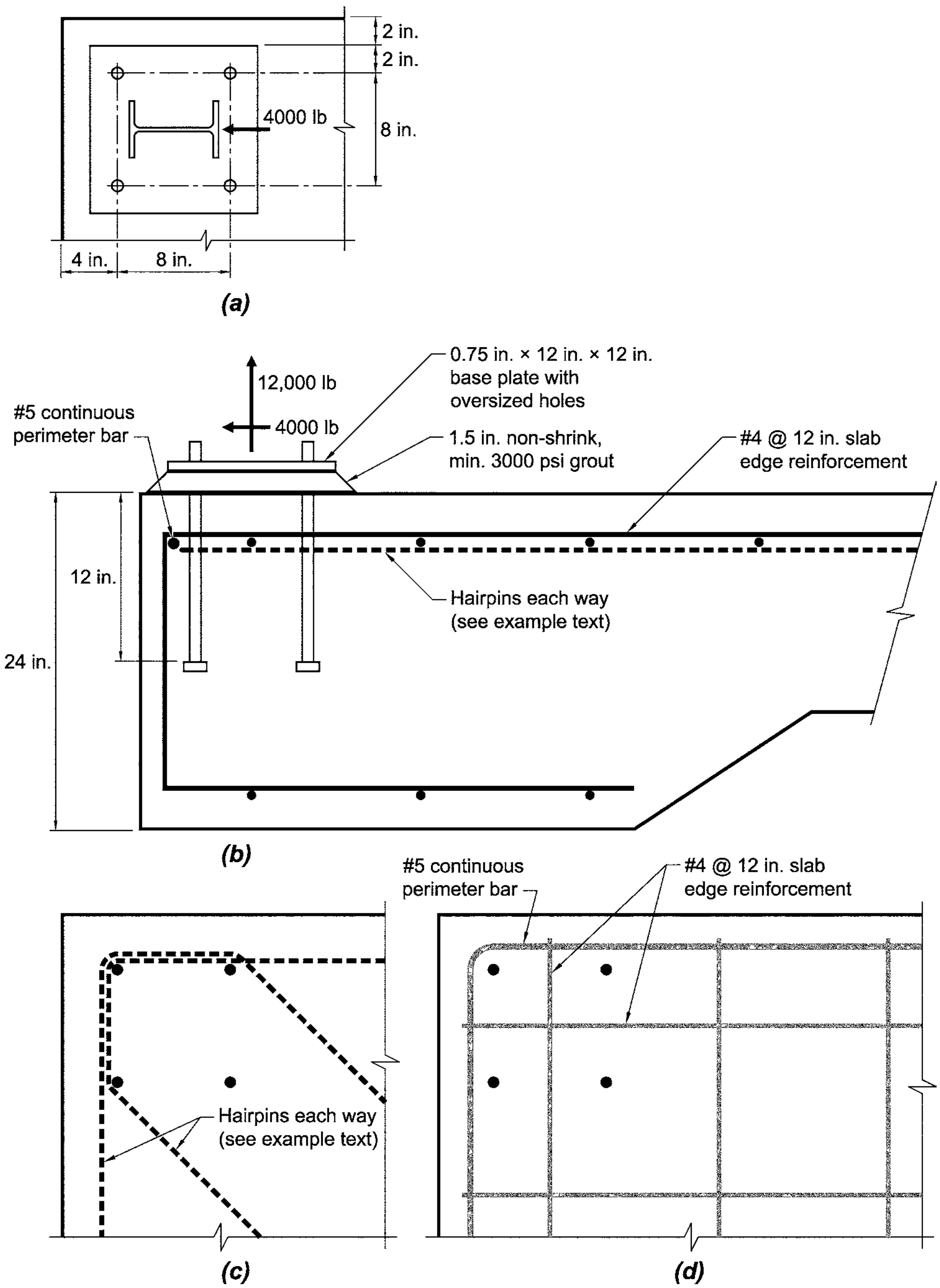
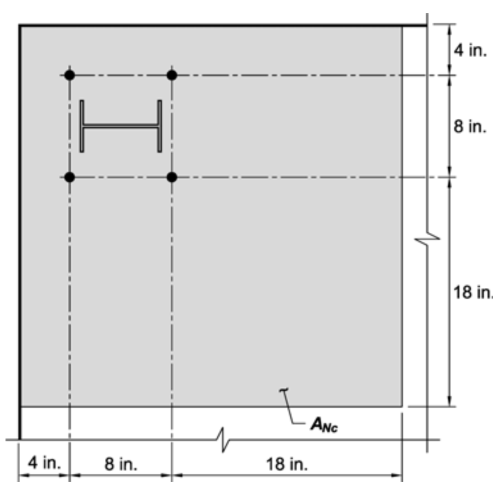


Fig. 4.12—(a) Example 8: Plan of column base plate and anchor rods; (b) section through foundation and anchors; (c) section (plan) looking down at hairpin supplemental reinforcement; and (d) section (plan) looking down at slab edge reinforcement.

Step	Calculations and discussion	ACI 318-05 Section
1.	Factored design load: N_{ua} = 12,000 lb V_{ua} = 4000 lb	
2.	Design considerations: Prior to ACI 318-08, there was no clear definition of the effects of reinforcement around the anchors. Previously, two conditions were defined: Condition A and Condition B. The differences between Condition A, Condition B, and the associated requirements for supplemental reinforcement were the subject of different applications. One application of the Condition A supplemental reinforcement was the reinforcement having to be proportioned, detailed, and developed to carry the full anchor design load, thereby nearly eliminating the breakout failure mode. A second application of Condition A is that the amount of reinforcement is intermediate between carrying the full anchor design load and that of Condition B, where no additional supplemental reinforcement is used. In the second application, Condition A supplemental reinforcement is not required to be proportioned, detailed, and developed to carry the full load. Condition A supplemental reinforcement is present and is oriented and detailed to restrain the breakout prism when cracking occurs (Refer to Fig. 4.12(c)), but is not specifically designed to carry the full load when the concrete breakout prism occurs. Experimental testing has shown that prescriptive reinforcement will increase the ductility of the system and a small increase in the phi factor is justified by the code. This second application has been clarified in ACI 318-08. For Condition B, reinforcement may be present near the anchor, but is not oriented to aid in restraining the breakout prism or no reinforcement is present. ACI 318-08 defines the fully developed condition as “anchor reinforcement,” which is proportioned, detailed, and developed to hold the breakout prism in place and to support a design load greater than the breakout strength. In this guide, the supplemental reinforcement is considered as Condition A for shear. Condition B has been considered for tension. This interpretation is confirmed in ACI 318-08. In the following calculations, the supplementary reinforcement (No. 4 hairpins and No. 5 perimeter bar) shown in Fig. 4.12(c)(d) is not proportioned to carry the anchor load at the shear breakout prism load. It is considered to enhance the ductility of the system in shear, as noted in the explanation of the second interpretation above for Condition A. Condition B is assumed for all tension capacity calculations. The anchor system should be designed as follows: $\phi N_n \geq N_{ua}$ where ϕN_n is the lowest design strength in tension from all appropriate failure modes. $\phi V_n \geq V_{ua}$ where ϕV_n is the lowest design strength in shear from all appropriate failure modes.	Eq. (D-1) Eq. (D-2)
3.	Anchor material: ASTM F1554 Grade 55 rod material meets the requirements of the ductile steel element definition in D.1, which is elongation of at least 14% and reduction in area of at least 30%. ASTM F1554 material has 21% elongation in 2 in. of length with 30% reduction in area (Table A.1).	D.1

	Tensile capacity calculations	
4.	Determine the steel strength of anchors in tension, ϕN_{sa}:	D.5.1.2
	$\phi N_{sa} = \phi n A_{se} f_{uta}$	Eq. (D-3)
	$\phi = 0.75$	D.4.4
	$A_{se} = 0.334$ (Table A.2(a))	
	$\phi n A_{se} f_{uta} = 0.75(4)(0.334)(75{,}000) = 75{,}150$ lb	
5.	Determine the breakout capacity of the anchor group in tension, ϕN_{cbg}:	D.5.2
	$\phi N_{cbg} = \phi \dfrac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$	Eq. (D-5)
	$\psi_{cp,N} = 1.0$ (for cast-in anchors)	D.5.2.7
	$\phi = 0.70$	D.4.4
	Condition B is selected. No supplementary reinforcement is provided to resist tension breakout. There is no reinforcement provided that is oriented to restrain the tension breakout prism. The reinforcement present is oriented in such a way that it will aid in enhancing the ductility of the system in shear—more on that in the shear section (Refer to Step 1 in shear calculations)—and not the tension breakout prism.	
	$1.5h_{ef} = 18$ in.	
	Check the basic concrete breakout strength:	RD.5.2.1
	$A_{Nc} = (4 + 8 + 18)(4 + 8 + 18) = 900$ in.^{2}	
		
	Fig. 4.13—Example 8: Sketch of projected breakout area in tension for group.	
	$A_{Nco} = 9h_{ef}^2 = 9(12)^2 = 1296$ in.^{2}	Eq. (D-6)
	$\psi_{ec,N} = 1.0$ (as no eccentricity exists)	D.5.2.4
	Check to see if the edge distance modifier $\psi_{ed,N}$ is other than 1.0:	
	$c_{a,min} = 4$ in.	
	$c_{a,min} < 1.5h_{ef} = 18$ in.	
	therefore, $\psi_{ed,N}$ will be less than 1.0.	D.5.2.5
	$\psi_{ed,N} = 0.7 + 0.3\dfrac{c_{a,min}}{1.5h_{ef}} = 0.7 + 0.3\dfrac{4.0}{1.5(12)} = 0.767$	Eq. (D-11)

5. (cont.)	Check if the cracked concrete $\psi_{c,N}$ modifier is other than 1.0:	
	Unless an analysis is done to show no cracking at service loads, cracking should be assumed; therefore, $\psi_{c,N}$ = 1.0.	D.5.2.6
	h_{ef} = 12 in. > 11 in.	
	therefore, Eq. (D-8) should be applied	D.5.2.2
	$N_b = 16\sqrt{f'_c}h_{ef}^{5/3} = 16\sqrt{3000}(12)^{5/3}$ = 55,120 lb	Eq. (D-8)
	$\phi N_{cbg} = 0.7\frac{900}{1296}(1.0)(0.767)(1.0)(55{,}120)$ = 20,551 lb	
6.	Determine the pullout strength in tension, ϕN_{pn}:	D.5.3
	$\phi N_{pn} = \phi\psi_{c,P}N_p$	Eq. (D-14)
	ϕ = 0.70	D.4.4
	Unless an analysis is done to show no cracking is present at service loads, cracking should be assumed, therefore,	
	$\psi_{c,P}$ = 1.0	D.5.3.6
	$N_p = 8A_{brg}f'_c$ = (8)(0.654)(3000) = 15,696 lb	Eq. (D-15)
	Refer to Table A.2(a) for bearing area of hex nut.	
	There are four rods, thus the total pullout capacity is	
	ϕN_{pn} = 0.7(4)(15,696) = 43,949 lb	
7.	Determine concrete side-face blowout capacity ϕN_{sbg}:	D.5.4
	Evaluate if side-face blowout is a consideration. The smallest edge distance, c_{a1}, should be less than $0.4h_{ef}$ = 0.4(12) = 4.8 in., and the spacing of the anchors should be less than $6c_{a1}$ = 6(4) = 24 in. Both requirements are met, so side-face blowout should be considered.	D.5.4.2
	$\phi N_{sbg} = \phi\left(1 + \frac{s}{6c_{a1}}\right)N_{sb}$	Eq. (D-18)
	ϕ = 0.70	D.4.4
	s = 8 in.	
	$N_{sb} = 160c_{a1}\sqrt{A_{brg}}\sqrt{f'_c} = 160(4)\sqrt{0.654}\sqrt{3000}$ = 28,348 lb	Eq. (D-17)
	$\phi N_{sbg} = 0.70\left(1 + \frac{8}{6(4)}\right)$ 28,348 = 26,458 lb	
8.	Summary in Table 4.14:	

Table 4.14—Summary of tension design strengths

Failure mode	Anchor design strength	Calculated design strength, lb	Controlling failure mode
Steel	ϕN_{sa}	75,150	—
Concrete breakout	ϕN_{cbg}	20,551	← Controls
Concrete pullout	ϕN_{pn}	43,949	—
Concrete side-face blowout	ϕN_{sb}	26,458	—

	Shear capacity calculation	
9.	Determine the steel strength of anchor in shear, ϕV_{sa}: $\phi V_{sa} = \phi n 0.6 A_{se} f_{uta}$ $\phi = 0.65$ $A_{se} = 0.334$ (Refer to Table A.2(a) of this design guide) The steel strength of each anchor is: $\phi n 0.6 A_{se} f_{uta} = 0.65(0.6)(0.334)(75{,}000) = 9770$ lb Where anchors are used with built-up grout pads, the design strength should be multiplied by 0.8. $\phi V_{sa} = 0.8(9.77) = 7816$ lb per anchor	D.6.1.2 Eq. (D-20) D.4.4 D.6.1.3

10. Determine the breakout strength in shear, ϕV_{cbg}: D.6.2

$$\phi V_{cbg} = \phi \frac{A_{Vc}}{A_{Vco}} \psi_{ec,V} \psi_{ed,V} \psi_{c,V} V_b$$ Eq. (D-22)

$\phi = 0.75$ D.4.4

Condition A. The combination of continuous No. 5 perimeter bars and No. 4 bent slab bars qualify as supplementary reinforcement for resisting a shear breakout, as do the hairpins. The hairpins could be designed to directly restrain the concrete breakout, thereby eliminating breakout as a failure mode.

However, at this time, the combination of the continuous No. 5 perimeter bars and No. 4 bent slab bars and hairpins are considered as supplementary reinforcement in terms of Condition A. Although the hairpin is not considered to directly restrain the breakout prism and does not eliminate breakout as a failure mode, there is reinforcement present that will help restrain the breakout prism and improve ductility. This allows an increase in the capacity by changing the phi factor to the Condition A value.

$$V_b = 7\left(\frac{\ell_e}{d_o}\right)^{0.2} \sqrt{d_o}\sqrt{f_c'}(c_{a1})^{1.5}$$ Eq. (D-24)

ℓ_e cannot exceed $8d_o = 8(3/4) = 6$ in. D.6.2.2

c_{a1} is either from the first or second line of fasteners. The base plate is fabricated with oversized holes, making it possible for the two fasteners closest to the edge to engage before those at the line farthest from the edge. Unless steps are taken to ensure that the fasteners farthest from the edge will be engaged—for example, welding plate washers with standard holes to the base plate in the field (which will be considered later in this example)—c_{a1} should conservatively be taken as the distance from the free edge to the first line of fasteners.

$c_{a1} = 4$ in. RD.6.2.1

$A_{Vc} = (c_{a2} + s + 1.5c_{a1})(1.5c_{a1}) = (4 + 8 + 6)(6) = 108$ in.^{2} Fig. RD.6.2.1(b)

$A_{Vco} = 4.5(c_{a1})^2 = 4.5(4)^2 = 72$ in.^{2} Fig. RD.6.2.1(a) D.6.2.1 Eq. (D-23)

Substituting:

$$V_b = 7\left(\frac{6}{0.75}\right)^{0.2} \sqrt{0.75}\sqrt{3000}(4)^{1.5} = 4026 \text{ lb}$$ Eq. (D-24)

$\psi_{ec,V} = 1.0$ (no eccentric shear is applied) D.6.2.5

$c_{a2} = 4$ in.

therefore, $c_{a2} < 1.5c_{a1}$, and $\psi_{ed,V}$ should be calculated D.6.2.6

$$\psi_{ed,V} = 0.7 + 0.3\frac{c_{a2}}{1.5c_{a1}} = 0.7 + 0.3\frac{4.0}{1.5(4.0)} = 0.9$$ Eq. (D-28)

A continuous No. 5 bar is present around the edge of the slab, therefore

$\psi_{c,V} = 1.2$ D.6.2.7

Substituting:

$$\phi V_{cbg} = 0.75\frac{108}{72}(1.0)(0.9)(1.2)(4026) = 4892 \text{ lb}$$ Eq. (D-22)

The breakout capacity of the detail is limited. The designer may design the hairpins in the slab to directly restrain concrete breakout per D.4.2.1, thereby eliminating breakout as a failure mode.

10. (cont.)	*Note: Consider the effect on the capacity if plate washers with standard holes were welded to the base plate at each anchor. In such a case, the breakout prism could be assumed to form from the back anchors per RD.6.2.2.* c_{a1} = 12 in. $1.5c_{a1}$ = 18 in. A_{Vc} = (4 + 8 + 18)(18) = 540 in.^{2} $A_{Vco} = 4.5(c_{a1})^2 = 4.5(12)^2$ = 648 in.^{2} $V_b = 7\left(\frac{6}{0.75}\right)^{0.2}\sqrt{0.75}\sqrt{3000}(12)^{1.5}$ = 20,921 lb c_{a2} = 4 in. therefore, $c_{a2} < 1.5c_{a1}$, and $\psi_{ed,V}$ should be calculated $\psi_{ed,V} = 0.767$ $\phi V_{cbg} = 0.75\frac{540}{648}(1)(0.767)(1.2)(20{,}921)$ = 12,035 lb The addition of plate washers with standard holes welded to the base plate would allow the designer to assume that the breakout would occur from the back row of fasteners and to realize a substantial increase in breakout capacity. If this assumption is made, the entire load is carried by the rear fasteners and the steel strength of the rear fasteners alone should be sufficient to support the full design load. Additionally, the bearing capacity of the thin edge of the steel washer against the bolt should be checked.	

11.	Determine the concrete pryout strength ϕV_{cp}: $\phi V_{cpg} = \phi k_{cp} N_{cbg}$ $\phi = 0.7$ $k_{cp} = 2.0$ (when $h_{ef} \geq 2.5$ in.) $N_{cbg} = \frac{A_{Nc}}{A_{Nco}}\psi_{ed,N}\psi_{c,N}N_b = \frac{900}{1296}(0.767)(1)(55{,}120) = 29{,}359$ lb $\phi V_{cp} = 0.7(2)(29{,}359) = 41{,}103$ lb	D.6.3 Eq. (D-30) D.4.4 D.6.3.1 Eq. (D-5)
12.	*Summary:* $\phi V_{sa} = 7816$ lb per anchor $\phi V_{cbg} = 4829$ lb (assuming no welded plate washers are provided) $\phi V_{cp} = 41{,}103$ lb *Note: In the case where plate washers were welded to the farthest line of fasteners, a substantial increase in concrete breakout capacity was realized. The steel strength of these two anchors should be compared with the concrete breakout capacity engaged by them. The concrete breakout capacity of 12,035 lb with two anchors effective is less than the steel capacity 2(7816) = 15,632 lb. Furthermore, the pryout capacity of the two rear fasteners alone should be checked:* $\phi V_{cpg} = \phi k_{cp} N_{cbg}$ $\phi = 0.7$ $k_{cp} = 2.0$ (when $h_{ef} \geq 2.5$ in.) $A_{Nc} = (4 + 8 + 18)(4 + 8 + 18) = 900$ in.^{2} (for the rear two anchors) $N_{cbg} = \frac{900}{1296}(0.767)(1)(55{,}120) = 29{,}359$ lb $\phi V_{cpg} = 0.7(2)(29{,}359) = 41{,}103$ lb *Concrete breakout in shear controls.* Shear breakout severely limits the capacity of this connection.	
	Interaction of tensile and shear forces	
13.	Check the provisions of D.7.1 and D.7.2: $V_{ua} = 4000$ lb $> 0.2\phi V_n = 0.2(4892) = 978$ lb (assumes no plate washers welded to base plate and no hairpins provided) $N_{ua} = 12{,}000$ lb $> 0.2\phi N_n = 0.2(20{,}551) = 4110$ lb therefore, the interaction of the two should be checked $\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} = \frac{12{,}000}{20{,}551} + \frac{4000}{4892} = 1.4 > 1.2$ – No good Add field-welded plate washers and thus increase ϕV_n to 12,035 lb (see above; concrete breakout failure of the two rear anchors governs). $V_{ua} = 4000$ lb $> 0.2\phi V_n = 0.2(12{,}035) = 2407$ lb therefore, the interaction should still be checked $\frac{N_u}{\phi N_{ua}} + \frac{V_u}{\phi V_{na}} = \frac{12{,}000}{20{,}551} + \frac{4000}{12{,}035} = 0.92 \leq 1.2$ – OK The four-anchor rod group is adequate to resist the applied loading assuming plate washers are welded to the base plate at all anchor rod locations.	D.7 D.7.1 D.7.2 Eq. (D-31)

4.9—Example 9: Group of headed studs in tension near an edge with eccentricity

Determine the allowable factored tension load N_{ua} (determined from ACI 318, Section 9.2.1) that can be supported by a group of four 1/2 in. x 4 in. AWS D1.1 Type B headed studs spaced 6 in. on center each way and welded to a 1/2 in. thick embedded plate. The centerline of the structural attachment to the plate is located 2 in. off the centerline of the plate, resulting in an eccentricity of the tension load of 2 in., as shown in Fig. 4.14. The anchor group is installed in the bottom of an 8 in. thick slab with the centerline of the group 6 in. from a free edge of the slab.

Note: This is the configuration chosen as a solution for Example 4 to support a 14,000 lb factored tension load centered on the connection. The only difference is the eccentricity of the tension load. In Step 3 of Example 4, the spacing between anchors dictates that they be designed as an anchor group.

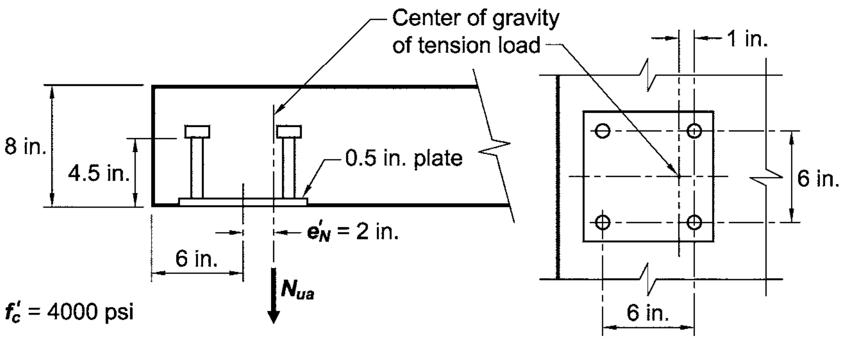
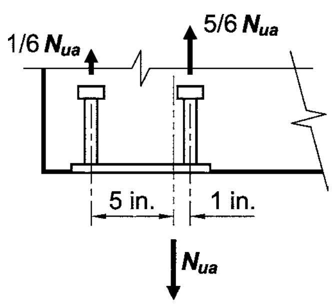


Fig. 4.14—Example 9: Group of headed studs in tension near an edge with eccentricity. (Note: Reinforcement not shown for clarity.)

Step	Calculations and discussion	ACI 318-05 Section
1.	Determine distribution of loads to the anchors: Assuming an elastic distribution of loads to the anchors, the eccentricity of the tension load will result in a higher force on the interior row of anchors. Although the studs are welded to the plate, their flexural stiffness at the joint with the plate is minimal compared with that of the plate. Therefore, assume a simple support condition for the embedded plate as shown in Fig. 4.15:  *Fig. 4.15—Example 9: Assumed distribution of load.* The two interior studs will control the capacity of the group when checking for strength controlled by steel (ϕN_{sa}), the concrete pullout strength(ϕN_{pn}), or both, since the two interior studs receive a higher percentage of load and these failure modes pertain to individual anchors in the group. 5/6N_{ua} should be less than or equal to ϕN_{sa} for two studs and ϕN_{pn} for two studs. Rearranged, $N_{ua} \leq 6/5\phi N_{sa,2\ studs}$ and $N_{ua} \leq 6/5\phi N_{pn,2\ studs}$.	D.3.1

2.	Determine the design steel strength ϕN_{sa}, as controlled by the two interior anchors with the highest tensile load: $\phi N_{sa} = \phi n A_{se} f_{uta}$ where $\phi = 0.75$ (assuming N_{ua} will be determined using load factors and combinations from ACI 318 Section 9.2). *Note: Per the ductile steel element definition in D.1, AWS D1.1 Type B studs qualify as a ductile steel element (20% minimum elongation in 2 in., which is greater than the 14% required, and a minimum reduction in area of 50%, which is greater than the 30% required). Refer to Table A.1 for this information.* $n = 2$ (for the two inner studs with the highest tension load) $A_{se} = 0.196$ in.^{2} (Table A.2(b)) $f_{uta} = 65{,}000$ psi (Table A.1) Substituting: $\phi N_{sa,\ 2\ studs} = 0.75(2)(0.196)(65{,}000) = 19{,}110$ lb Therefore, the allowable factored tensile load N_{ua}, as controlled by the anchor steel, is: $N_{ua} \leq 6/5\phi N_{sa,\ 2\ studs} \leq 6/5(19{,}110) \leq 22{,}932$ lb	D.5.1 Eq. (D-3) D.4.4(a)i
3.	Determine design concrete breakout strength ϕN_{cbg}: The only difference between concrete breakout strength in this example and Example 4 is the introduction of the eccentricity modification factor $\psi_{ec,N}$. From Example 4 with $\psi_{ec,N} = 1.0$ (no eccentricity) $\phi N_{cbg} = \frac{\phi A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b = 14{,}201$ lb Determine $\psi_{ec,N}$ for this example: $\psi_{ec,N} = \frac{1}{\left(1 + \frac{2e_N'}{3h_{ef}}\right)} \leq 1.0$ where $e_N' = 2$ in. (e_N' is the distance between centroid of anchor group and tension force) $h_{ef} = 4.5$ in. (1/2 in. plate plus 4 in. embedment of headed stud) Substituting: $\psi_{ec,N} = \frac{1}{\left(1 + \frac{2(2)}{3(4.5)}\right)} = 0.77$ therefore: $\phi N_{cbg} = (0.77)(14{,}201 \text{ lb}) = 10{,}935$ lb (ϕ of 0.7 was previously included in determining 14,201 lb result of Example 4)	D.5.2 Eq. (D-5) D.5.2.4 Eq. (D-9)

4.	Determine the design pullout strength as controlled by the two anchors with the highest tensile load (ϕN_{pn}): $\phi N_{pn,\,1\,stud} = \phi\psi_{c,P} N_p = \phi\psi_{c,P} 8 A_{brg} f_c'$ where $\phi = 0.70$ *Note: Condition B applies in all cases when pullout strength governs.* $\psi_{c,P} = 1.0$ (for locations where concrete cracking is likely to occur [for example, the bottom of the slab] and cracking is controlled by flexural or confining reinforcement) $A_{brg} = 0.589$ in.^{2} (refer to Step 4 of Example 4) Substituting: $\phi N_{pn,1\,stud} = (0.70)(1.0)(8.0)(0.589)(4000) = 13{,}194$ lb For the two equally loaded interior studs: $\phi N_{pn,2\,studs} = 2(13{,}194 \text{ lb}) = 26{,}387$ lb Therefore, the maximum N_{ua} as controlled by pullout is: $N_{ua} = 6/5\phi N_{pn,2\,studs} = 6/5(26{,}387 \text{ lb}) = 31{,}664$ lb	D.5.3 Eq. (D-14) Eq. (D-15) D.4.4(c)ii D.5.3.6
5.	Evaluate side-face blowout: Side-face blowout needs to be considered when the edge distance from the centerline of the anchor to the nearest free edge is less than $0.4h_{ef}$. For this example: $0.4h_{ef} = 0.4(4.5) = 1.8$ in. ≤ 3 in. actual edge distance – OK The side-face blowout failure mode is not applicable.	D.5.4
6.	Required edge distances, spacings, and thicknesses to preclude splitting failure: Since a welded headed anchor is not torqued, the minimum cover requirements in ACI 318 Section 7.7 apply. The minimum clear cover for a 1/2 in. bar not exposed to earth or weather is 3/4 in., which is less than the 2-1/2 in. cover provided to the stud head.	D.8 7.7 7.7.1(c)
7.	The summary of design strengths in tension for each failure mode is shown in Table 4.15:	

Table 4.15—Summary of tension design strengths

Failure mode	Anchor design strength	Calculated design strength, lb	Controlling failure mode
Steel	ϕN_{sa}	22,932	—
Concrete breakout	ϕN_{cbg}	10,935	← Controls
Concrete pullout	ϕN_{pn}	31,664	—
Concrete side-face blowout	ϕN_{sb}	No applicable	—

The maximum factored tension load N_{ua} that can be supported from this anchorage is 10,935 lb.

Note: Example 4 with the same anchors, but without an eccentricity, was also controlled by concrete breakout strength, but had a factored load capacity of 14,201 lb (Refer to Step 3 of Example 4).

4.10—Example 10: Multiple headed anchor connection subjected to moment and shear

Design an embedded plate to support the end reaction and end bending moment of a steel beam using a group of eight welded headed studs spaced as shown in Fig. 4.16. The factored design bending moment M_{ua} is 30,000 ft-lb (360 in.-kip) and the factored shear load coming from the beam end reaction, V_{ua}, is 20,000 lb (20 kips) (Section 9.2, ACI 318-05, load combinations used). The connection is located on the side face of a reinforced concrete spandrel girder (Fig. 4.16).

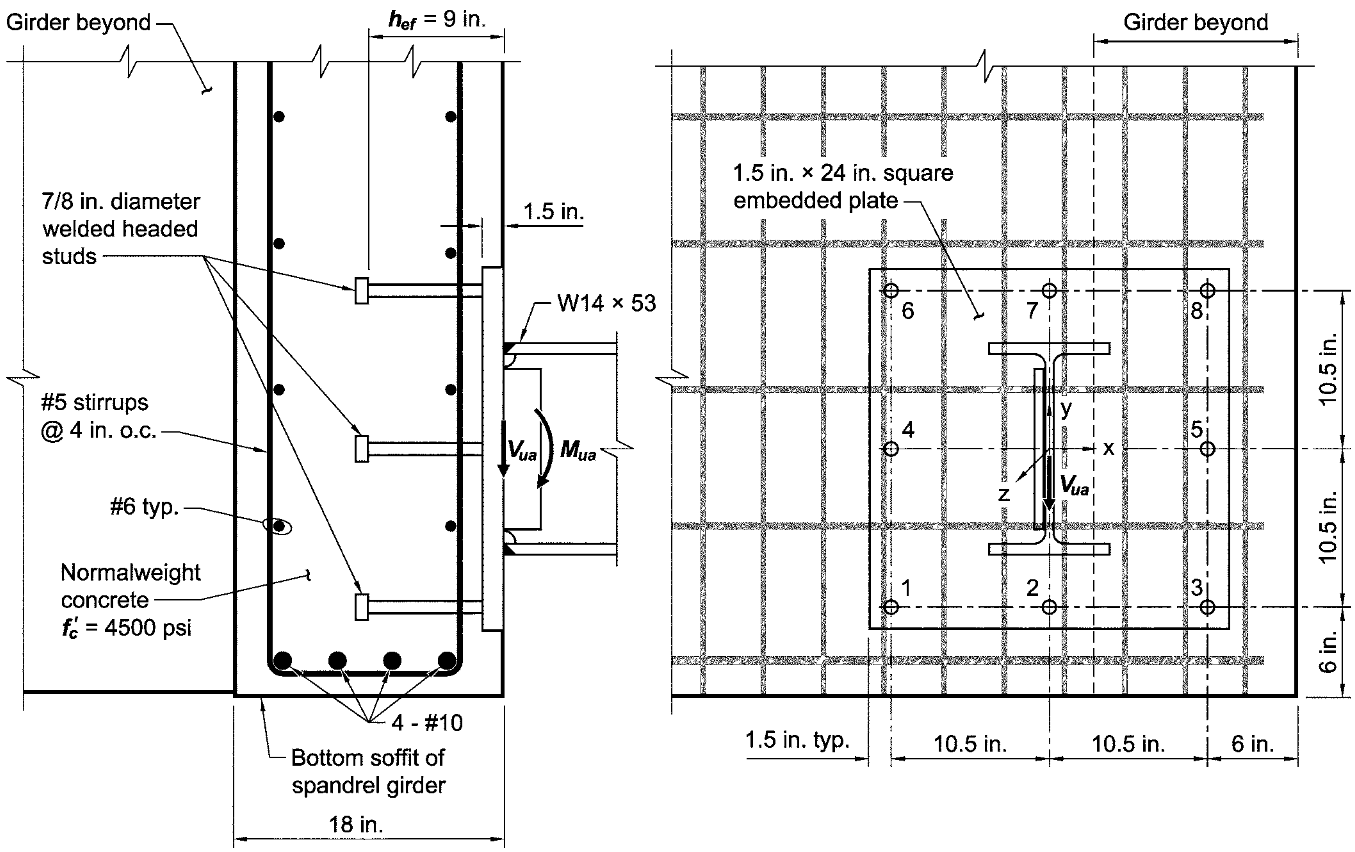
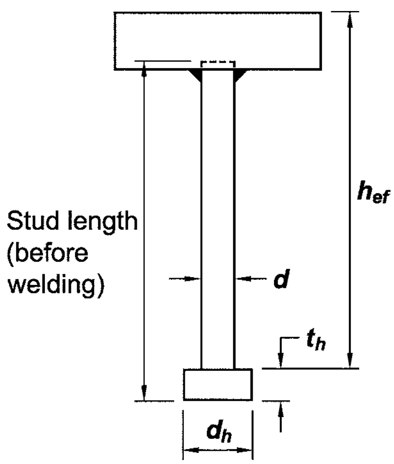


Fig. 4.16—Example 10: Beam support consisting of embedded plate with welded studs.

Step	Calculations and discussion	ACI 318-05 Section
1.	Check for conformance with code requirements in the connection detail shown in Fig. 4.16, as based on preliminary design: *Note: It is often difficult to simultaneously determine the required anchor diameter and embedment length due to the complexity of the load distribution to each anchor and the interaction of tensile and shear forces. The general approach, from a practical design perspective, is to assume some design values in advance and verify their acceptability in the design calculation.*	

2.	Design basis: a) Basic assumptions • Cracked concrete • Elastic design b) Materials • Embedded plate: ASTM A36/A36M 1-1/2 in. x 24 in. x 24 in. (t_p = 1-1/2 in.) • Anchors: ° 7/8 in. x 8-3/16 in. AWS D1.1 Type B mild steel welded headed stud (standard length shear stud) as shown in Fig. 4.16 ° Specified yield strength of anchor steel f_{ya} = 51 ksi ° Specified tensile strength of anchor steel f_{uta} = 65 ksi ° Stud diameter d = 0.875 in. ° Stud head diameter d_h = 1.375 in. ° Stud head thickness t_h = 3/8 in. ° Reduction in stud length due to welding ~ 3/16 in. ° Effective anchor embedment: h_{ef} = 8-3/16 in. – 3/8 in. – 3/16 in. + 1-1/2 in. = 9.125 in. ≈ 9 in.  *Fig. 4.17—Example 10: Welded headed stud.*	

3. Determine anchor reactions: D.3.1

a) Tension: The determination of the anchor reactions is not trivial. While the use of finite element modeling may provide the closest approximation, it is possible to use conservative assumptions and statics to derive an acceptable set of anchor forces.

Simplified statically determinant analysis: Assume that the compression reaction is located directly beneath the toe of the W14 beam (conservative) and that the embedded plate exhibits rigid-body rotation (Fig. 4.18).

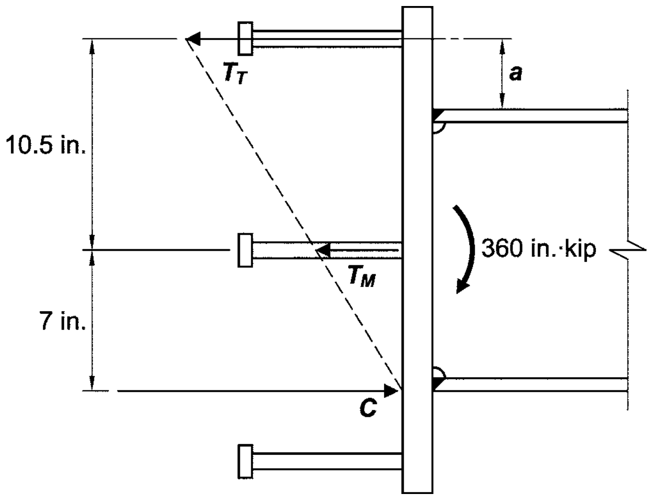


Fig. 4.18—Example 10: Tension load distribution.

T_T = tension reaction of top anchors (6, 7, and 8)
T_M = tension reaction of middle anchors (4 and 5)

$$\sum M = 0 \Rightarrow 17.5(T_T) + 7(T_M) - 360 = 0$$

Assume rigid-body rotation of embedded plate and determine ratio of anchor reactions (K = single anchor elastic stiffness, Δ = anchor elastic displacement):

$$\frac{\Delta_T}{\Delta_M} = \frac{17.5}{7} \Rightarrow \Delta_T = 2.5\Delta_M$$

Tension reaction of back three anchors:

$$T_T = 3\Delta_T K$$

Substituting Eq. (4-2) into Eq. (4-3):

$$T_T = 7.5\Delta_M K$$

Tension reaction of middle two anchors:

$$T_M = 2\Delta_M K$$

Substituting Eq. (4-5) into Eq. (4-4):

$$T_T = 3.75T_M$$

Substituting into Eq. (4-1):

T_M = 4.95 kips $\quad \therefore T_4 = T_5 = 4.95/2 = 2.48$ kips
T_T = 18.6 kips $\quad \therefore T_6 = T_7 = T_8 = 18.6/3 = 6.20$ kips

Total tension reaction:

3. N_{ua} = (3)(6.20) + (2)(2.48) = 23.6 kips

3. (cont.)

Check assumption of plate rigidity using results from analysis above:

The assumption that the compression reaction is located at the compression flange of the steel beam as shown in Fig. 4.18 is rational based on observations of tested assemblies. While the load distribution to the tension-loaded anchors will be affected by their proximity to the connected wide flange shape, such effects are likely to occur at larger rotations. Per D.3.1, the analysis may be based on elastic anchor response. On the tension side, check the moment at the beam face and compare it to the moment in the plate:

$$M_{y,pl} = S_x f_{y,pl} = \frac{bh^2}{6} \times f_{y,pl} = \frac{24(1.5)^2}{6} \times 36 \text{ ksi} = 324 \text{ in.-kip}$$

$$M_{face} = \Sigma T_T \times a = 3(6.20)(3.5) = 65.1 \text{ in.-kip} \leq 324 \text{ in.-kip} \quad \therefore \text{OK}$$

Note: The flexural stiffness of the plate in the transverse direction (x-axis, refer to Fig. 4.20) should be capable of distributing the forces to the outside anchors. This may be accomplished through the use of a thicker baseplate or stiffeners.

b) Shear: The anchors are welded to the attachment. Per RD.6.2.1, for anchors welded to a common plate, the shear is carried by the back anchor row as shown in Fig. 4.19.

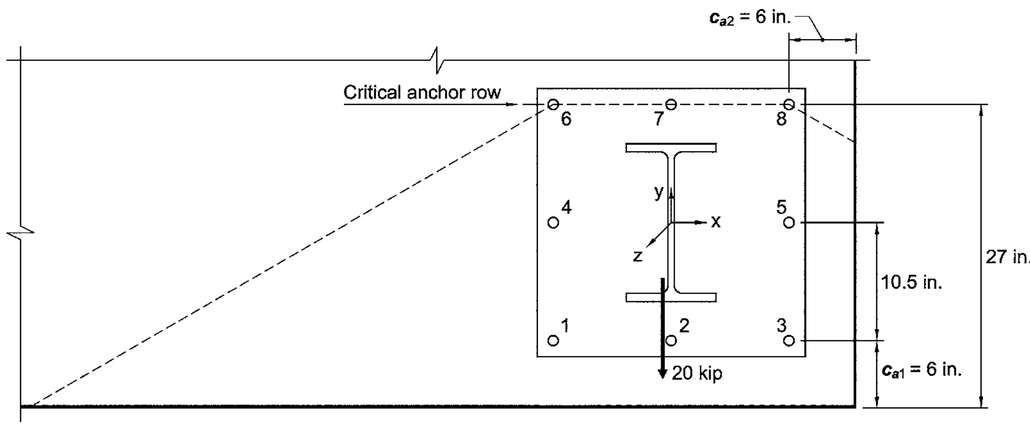


Fig. 4.19—Example 10: Assumption of critical anchor row for shear loading.

4. Determine design tensile strength ϕN_n as follows:

Nominal steel strength in tension N_{sa}: D.5.1.1
check:

f_{uta} = 65 ksi < $1.9f_{ya}$ = (1.9)(51) = 96.9 kip < 125 ksi ∴ OK D.5.1.2

Effective cross-sectional area of anchor:

$$A_{se} = \frac{\pi \times d^2}{4} = \frac{\pi \times 0.875^2}{4} = 601 \text{ in.}^2$$

Tension:

$N_{sa} = A_{se} \times f_{uta}$ = (0.601)(65) = 39.07 kips (for a single stud) Eq. (D-3)

Concrete breakout strength in tension, N_{cbg}: D.5.2

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

Determine A_{Nc} for the group of tensioned anchors, T_4, T_5, T_6, T_7, and T_8, and A_{Nco} for single anchor:

The projected concrete failure area A_{Nc} is shown shaded in Fig. 4.20 and is calculated as follows:

h_{ef} = 9 in.

$A_{Nc} = (1.5 \times h_{ef} + c_{a2} + 2 \times s_x)(2 \times 1.5 \times h_{ef} + s_y)$ D.5.2.1
= (1.5 × 9 + 6 + 2 × 10.5)(2 × 1.5 × 9 + 10.5)
= 1519 in.^{2}

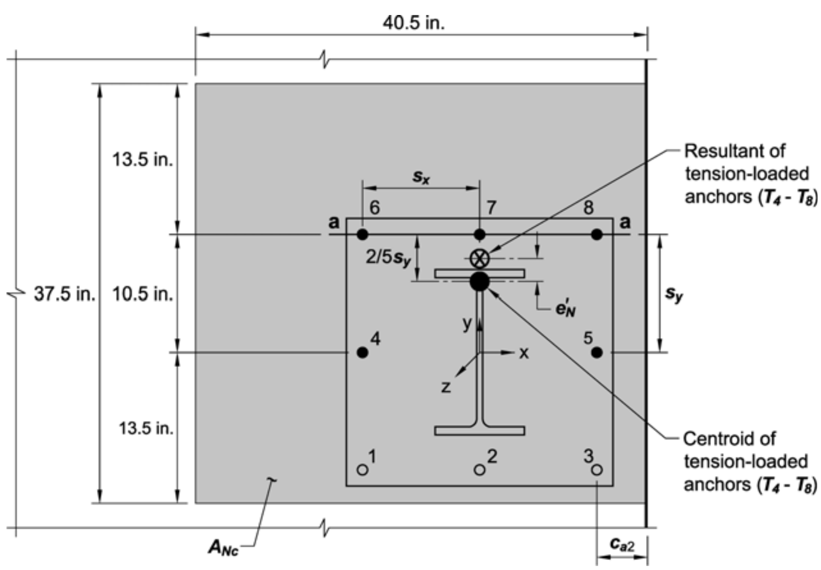


Fig. 4.20—Example 10: Determination of projected failure area and e'_N *for tension-loaded anchors.*

Determine A_{Nco} for the single anchor:

$A_{Nco} = 9 \times h_{ef}^2 = 9(9)^2$ = 729 in.^{2} Eq. (D-6)

Determine the eccentricity modification factor $\psi_{ec,N}$:

$$\psi_{ec,N} = \frac{1}{\left(1 + \frac{2 \times e_N'}{3 \times h_{ef}}\right)} \leq 1.0$$ Eq. (D-9)

4. (cont.)	Determine the eccentricity e_N' of the resultant tension load with respect to the centroid of the tension-loaded anchors.	Fig. RD.5.2.4(b)
	The location of the geometric centroid of tension-loaded anchors (T_4 to T_8) as measured from axis a-a (Fig. 4.20) is given by $(2/5)s_y$. Summing moments about a-a, the location of the resultant of the tension-loaded anchors is given by $[(T_4 + T_5)s_y]/N_{ua}$. The eccentricity e_N' between the centroid of the tension-loaded anchors and the tension resultant is thus given as:	
	$$e_N' = \frac{2 \times s_y}{5} - \frac{(T_4 + T_5)s_y}{N_{ua}} = \frac{(2)(10.5)}{5} - \frac{(2.48 + 2.48)(10.5)}{23.6} = 1.99 \text{ in.}$$	
	$$\psi_{ec,N} = \frac{1}{\left(1 + \frac{2(1.99)}{3(9)}\right)} = 0.87$$	
	Determine the near-edge modification factor $\psi_{ed,N}$:	
	$c_{a,min} = 6$ in. $< 1.5h_{ef} = 13.5$ in.	
	$$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} = 0.7 + 0.3\frac{6}{1.5(9)} = 0.83$$	Eq. (D-11)
	Determine the modification factor for cracked and uncracked concrete, $\psi_{c,n}$:	
	Per D.5.2.6, unless an analysis is performed to show no cracking at service loads, the concrete is assumed to be cracked for the purposes of determining the anchor design strength:	D.5.2.6
	$\psi_{c,N} = 1.0$ (for cracked concrete)	
	Determine the splitting modification factor $\psi_{cp,N}$:	
	$\psi_{cp,N} = 1.0$ (for cast-in anchors)	D.5.2.7
	Determine the basic concrete breakout tensile strength for a single anchor, N_b:	D.5.2.2
	$N_b = k_c\sqrt{f_c'}\,h_{ef}^{1.5}$	Eq. (D-7)
	Note: h_{ef} *= 9 in. < 11 in.* $\Rightarrow$ *Use of Eq. (D-7) required.*	
	$k_c = 24$	
	$N_b = (24)\sqrt{4500}\,(9)^{1.5} = 43{,}469$ lb $= 43.5$ kips	
	Substituting into Eq. (D-5):	
	$$N_{cbg} = \frac{1519}{729}(0.87)(0.83)(1.0)(1.0)(43.5) = 65.5 \text{ kips}$$	
	Determine the pullout strength of anchors in tension, N_{pn}:	D.5.3
	$N_{pn} = \psi_{c,P}N_p$	Eq. (D-14)
	where	
	$\psi_{c,P} = 1.0$ (cracking assumed)	D.5.3.6
	Pullout strength of a single anchor in tension, N_p:	D.5.3.4
	$N_p = 8A_{brg}f_c'$	Eq. (D-15)

4. (cont.)

Determine bearing area of the head of a single stud:

$$A_{brg} = \frac{\pi(d_h^2 - d^2)}{4} = \frac{\pi(1.375^2 - 0.875^2)}{4} = 0.884 \text{ in.}^2$$

$\therefore N_p = (8)(0.884)(4500) = 31{,}824 \text{ lb} = 31.8 \text{ kips}$

Substituting into Eq. (D-14):

$N_{pn} = (1.0)(31.8) = 31.8$ kips (for a single anchor)

Determine the side-face blowout tensile strength, N_{sb}: D.5.4

$c_{a,min} = 6 \text{ in.} \geq 0.4h_{ef} = 3.6 \text{ in.}$ D.5.4.1

$\therefore$ side-face blowout failure mode not applicable

The design tensile strengths are calculated with the calculated nominal strengths and strength reduction factors of D.4.4.

Steel strength, $\phi = 0.75$ D.4.4

Per AWS D1.1, Type B studs qualify as a ductile steel element (20% minimum elongation in 2 in., see Table A.1). D.4.1.2

$\phi N_{sa} = (0.75)(39.07) = 29.3$ kips (for a single anchor)

Strength reduction factor for concrete breakout, Condition B: $\phi = 0.70$ D.4.4

Note: Although the cantilever beam contains significant amounts of beam reinforcement located in the anchor vicinity, this reinforcement is not configured to provide the necessary restraint to the tension-induced concrete breakout surface to warrant the use of the strength reduction factor associated with Condition A. Therefore, Condition B is assumed (supplementary reinforcement not provided). D.4.4

$\phi N_{cbg} = (0.7)(65.5) = 45.9$ kips (for the anchor group comprised of anchors T_4 to T_8) D.4.1.2

Strength reduction factor for pullout, $\phi = 0.70$ D.4.4

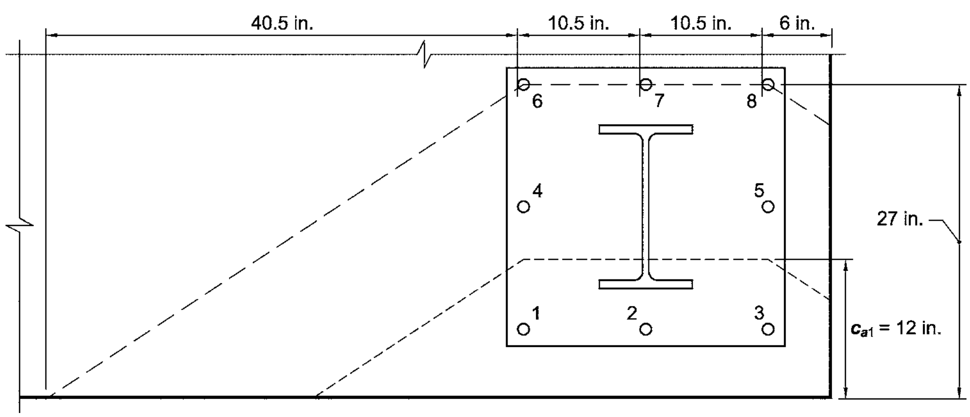
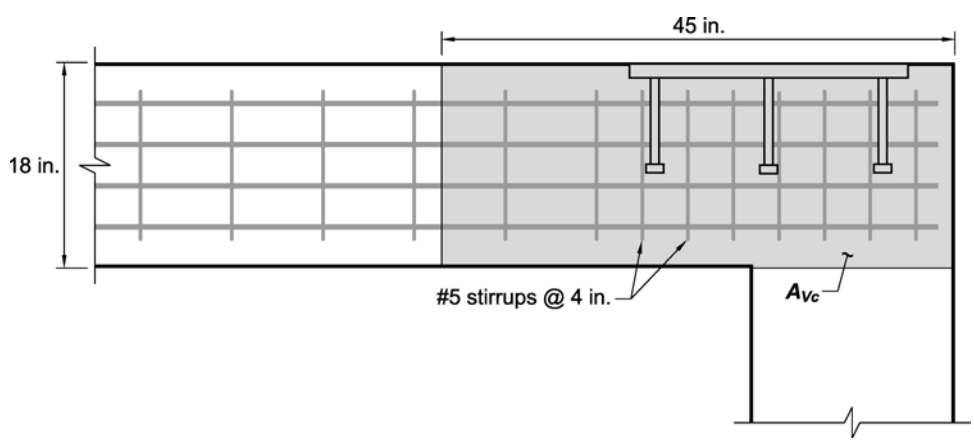
$\phi N_{pn} = (0.70)(31.8) = 22.3$ kip (for a single anchor) D.4.1.2

Note: Since this is an elastic analysis, the check for the controlling strength depends on the nature of the load distribution. In this case the anchor loads are not uniform, so the steel and pullout checks should be performed on the most critically-loaded anchor. Since the CCD method predicts the concrete breakout resistance of the group, the concrete breakout check is always performed for the group of tension-loaded anchors.

The summary of connection strength in tension is shown in Table 4.16. D.4.1.2

Table 4.16—Summary of tension design strengths and ratio of factored load to design strength

Failure mode	Critical anchor(s)	Anchor design strength ϕN_n	Calculated design strength, lb	Factored load N_{ua}, lb	Ratio $N_{ua}/\phi N_n$
Steel	6, 7, or 8	ϕN_{sa}	29,300	6200	0.21
Concrete breakout	Group 4 through 8	ϕN_{cbg}	45,900	23,600	0.5 (controls)
Concrete pullout	6, 7, or 8	ϕN_{pn}	22,300	6200	0.28
Concrete side-face blowout	8	ϕN_{sb}	Not applicable	Not applicable	—

5.	Determine shear design strength ϕV_n as follows:	
	Nominal steel strength in shear for cast-in headed studs, V_{sa}:	D.6.1.2
	$V_{sa} = nA_{se}f_{uta} = (1)(0.601)(65) = 39.1$ kip	Eq. (D-19)
	For a single anchor:	
	$V_{sa} = (1)(0.601)(65) = 39.1$ kip	
	Concrete breakout strength in shear, V_{cbg}:	D.6.2.1
	$V_{cbg} = \frac{A_{Vc}}{A_{Vco}}\psi_{ec,V}\psi_{ed,V}\psi_{c,V}V_b$	Eq. (D-22)
	Note: Per RD.6.2.1, for anchors welded to a common plate, the shear is carried by the back anchor row (Anchors 6, 7, and 8, Fig. 4.19). The shear load per anchor is therefore V_{ua} = 20 kip/3 = 6.7 kip. Note that the contribution to the shear capacity of the fastening by the concrete located in front of the embedded plate will be neglected.	RD.6.2.1
	The edge distance from the edge to the farthest anchor row = 27 in. (Fig. 4.21).	
	Determine the projected concrete failure areas A_{Vco} and A_{Vc}:	
	Note: The anchors are influenced by three edges, including the beam width (18 in.), as shown looking from underneath at the girder soffit (Fig. 4.22), and are located less than 1.5c_{a1}. Therefore, the value of c_{a1} used in Eq. (D-21) through (D-28) will be reduced.	D.6.2.4
		
	Fig. 4.21—Example 10: Determination of projected failure area for shear loading.	
		
	Fig. 4.22—Example 10: Determination of projected failure surface for shear loading—view of girder soffit.	

5. (cont.)

Check:

$$c_{a1} = max\left|\frac{c_{a2}}{1.5};\frac{h_a}{1.5};\frac{s_{x,max}}{3}\right| = max\left|\frac{6}{1.5};\frac{18}{1.5};\frac{21}{3}\right| = 12 \text{ in.}$$

$A_{Vco} = 4.5 \times c_{a1}^2 = 4.5(12)^2 = 648$ in.^{2} — Eq. (D-23)

$A_{Vc} = (1.5 \times c_{a1} + 2 \times s_x + c_{a2})(h) = (45)(18) = 810$ in.^{2} (for Anchors 6, 7, and 8) — D.6.2.1

Determine the eccentricity modification factor $\psi_{ec,V}$:

$\psi_{ec,V} = 1.0$ (for no eccentricity in the connection) — D.6.2.5

Determine the near-edge modification factor $\psi_{ed,V}$:

$$\psi_{ed,V} = 0.7 + 0.3\frac{c_{a2}}{1.5c_{a1}} = 0.7 + 0.3\left(\frac{6}{1.5 \times 12}\right) = 0.80$$ Eq. (D-28)

Determine the modification factor for cracked and uncracked concrete, $\psi_{c,V}$, per D.6.2.7: — D.6.2.7

$\psi_{c,V} = 1.4$ (for 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 no more than 4 in.)

The No. 10 girder flexural reinforcement substantially exceeds the requirement of a No. 4 bar, and the associated stirrups for this example will be considered effective in meeting the requirements of D.6.2.7. One issue arises from the edge reinforcement, which in this case is a No. 10 bar, which does not extend beyond the failure surface on the right-hand side. While the language of D.6.2.7 does not specifically require this, it is a reasonable expectation that the edge reinforcement is intended to act as a tension tie across the face of the failure prism. The assumption is made here that the beam reinforcement is adequate to justify the increase provided by D.6.2.7. A check for development of the No. 5 stirrups in the assumed breakout surface as shown should be made (Fig. 4.23).

Per Section 12.2.2, for No. 6 and smaller straight deformed bars, the development length is given by: — 12.2.2

$$\ell_d = \left(\frac{f_y \times \psi_t \times \psi_e \times \lambda}{25\sqrt{f'_c}}\right)d_b = \left(\frac{60{,}000 \times 1 \times 1 \times 1}{25\sqrt{4500}}\right)0.625 = 22.3 \text{ in.}$$

$\ell_{d,provided} \approx (2 \times 10.5) + 6 - 2 - 2 = 23$ in.

The No. 5 stirrups, in this case, are also closed-loop stirrups anchored by their interaction with the No. 10 longitudinal steel, so they are OK.

For this example, the No. 5 stirrups will be considered as supplemental reinforcement in meeting the requirements of tying the shear failure prism to the structure.

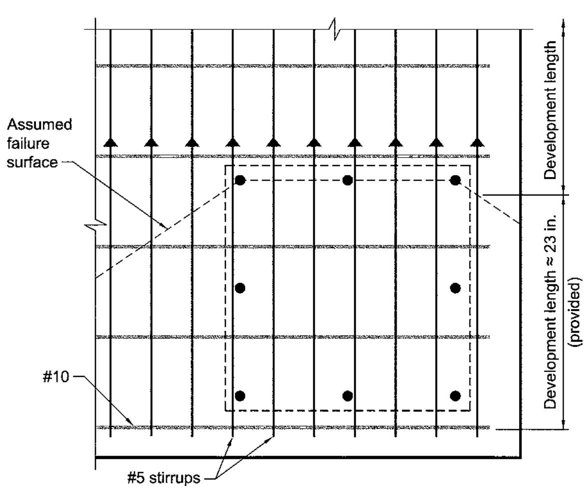


Fig. 4.23—Example 10: Elevation of girder reinforcement at end.

5. (cont.)

Determine basic concrete breakout shear strength V_b for cast-in anchors continuously welded to a steel attachment having a minimum thickness equal to the greater of 3/8 in. or half of the anchor diameter: D.6.2.3

$$V_b = 8\left(\frac{\ell_e}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f'_c}\,c_{a1}^{1.5}$$ Eq. (D-25)

where

$\ell_e = 8d_o = (8)(0.875) = 7$ in. ($8d_o$ controls) D.6.2.2

Substituting into Eq. (D-25):

$$V_b = (8)\left(\frac{7}{0.875}\right)^{0.2}\sqrt{0.875}\sqrt{4500}(12)^{1.5} = 31{,}629 \text{ lb} = 31.6 \text{ kip}$$

Substituting into Eq. (D-22):

$$V_{cbg} = \left(\frac{810}{648}\right)(1.0)(0.80)(1.4)(31.6) = 44.2 \text{ kip}$$

Determine the concrete pryout strength V_{cpg}: D.6.3

$V_{cpg} = k_{cp}N_{cbg}$ Eq. (D-30)

Check the pryout capacity associated with the back three anchors. This is conservative.

Determine the concrete breakout strength of rear three anchors (6, 7, and 8) shown in Fig. 4.24.

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

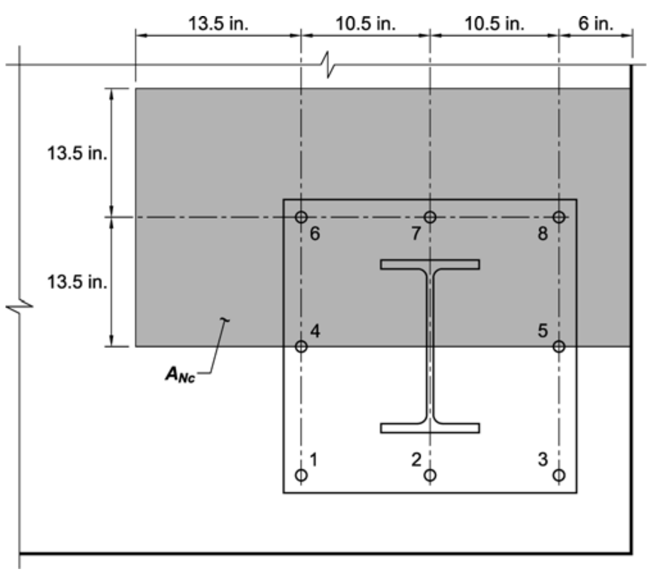


Fig. 4.24—Example 10: Determination of projected failure area for pryout.

$A_{Nc} = (3h_{ef})(1.5h_{ef} + 2 \times s_x + c_{a2}) = (27)(13.5 + 21 + 6) = 1094$ in.^{2}

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

$$N_{cbg} = \left(\frac{1094}{729}\right)(1.0)(0.83)(1.0)(1.0)(43.5) = 54.1 \text{ kips}$$

5. (cont.)

Note: For the calculation of the pryout capacity, the tension resistance of the anchor group is calculated assuming uniform load distribution to the anchors, therefore

$\psi_{ec,N} = 1.0$

$V_{cpg} = (2.0)(54.1) = 108.3$ kip

The shear design strengths are calculated with the calculated nominal strengths and strength reduction factors of D.4.4.

Determining the number of anchors contributing to the steel shear capacity depends on the assumption for concrete edge failure. The contributions of Anchors 1, 2, 3, 4, and 5 are neglected for the steel shear calculation because these anchors are contained within the assumed breakout surface (Fig. 4.25). This is reflected in the calculation of V_{ua} = 6.7 kips.

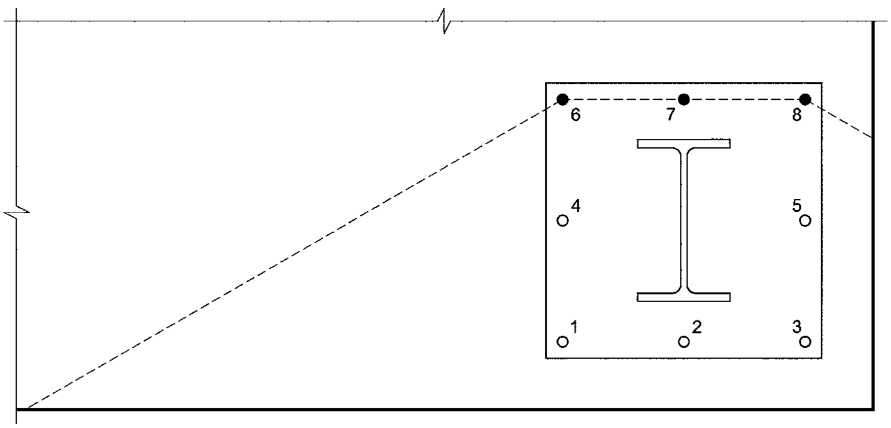


Fig. 4.25—Example 10: Determining the number of anchors contributing to V_{sa}.

Strength reduction factor for steel strength

$\phi = 0.65$ D.4.4

$\phi V_{sa} = (0.65)(39.1) = 25.4$ kip

Strength reduction factor for concrete breakout, Condition A:

$\phi = 0.75$ D.4.4

$\phi V_{cbg} = (0.75)(44.2) = 33.2$ kip D.4.1.2

Note: For the concrete breakout strength of the anchors in shear, the No. 5 stirrups in this example will be considered as supplemental reinforcement (Condition A), since the girder reinforcement will substantially engage the breakout surface.

Strength reduction factor for pryout, Condition B:

$\phi = 0.70$ D.4.4

$\phi V_{cpg} = (0.70)(108.3) = 75.8$ kip D.4.1.2

Note: As with the tension case, it is important to ensure that the most critically-loaded anchor is identified for the steel check. Since the CCD method predicts the concrete breakout resistance of the group, the concrete breakout and pryout checks are performed for the anchor group only.

The summary of connection design strength in shear is shown in Table 4.17: D.4.1.2

Table 4.17—Summary of shear design strengths and ratio of factored load to design strength

Failure mode	Critical anchor(s)	Anchor design strength ϕV_n	Calculated design strength, kip	Factored load V_{ua}, kip	Ratio $V_{ua}/\phi V_n$
Steel	6, 7, or 8	ϕV_{sa}	25.4	6.7	0.26
Concrete breakout	Group 6 through 8	ϕV_{cbg}	33.2	20	0.60 (controls)
Concrete pryout	Group 6 through 8	ϕV_{cpg}	75.8	20	0.26

6.	Check tension and shear interaction (concrete breakout): V_{ua} = 20 kip > $0.2\phi V_n$ = 6.64 kip N_{ua} = 23.6 kip > $0.2\phi N_n$ = 9.18 kip $\therefore \frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} = \frac{23.6}{45.9} + \frac{20}{33.2} = 1.12 < 1.2 \therefore$ – OK Interaction for steel failure system OK by inspection.	D.7 D.7.1 D.7.2 Eq. (D-31)

4.11—Example 11: Multiple post-installed anchor connection subjected to seismic moment and shear

Check the design of a steel plate beam support with a group of six post-installed undercut anchors spaced as shown (Fig. 4.26) to support a factored reversible bending moment M_{ua} of 5000 ft-lb (60 in.-kip) and a factored reversible shear load V_{ua} of 20,000 lb (20 kips) based on elastic analysis (refer to following note). The loading (Section 9.2, ACI 318-05) results from an earthquake in a region of moderate or high seismic risk. The connection is located far from edges (Fig. 4.26).

Refer to Table A.4 for a sample table of data for a fictitious post-installed, undercut anchor, determined from testing in accordance with ACI 355.2.

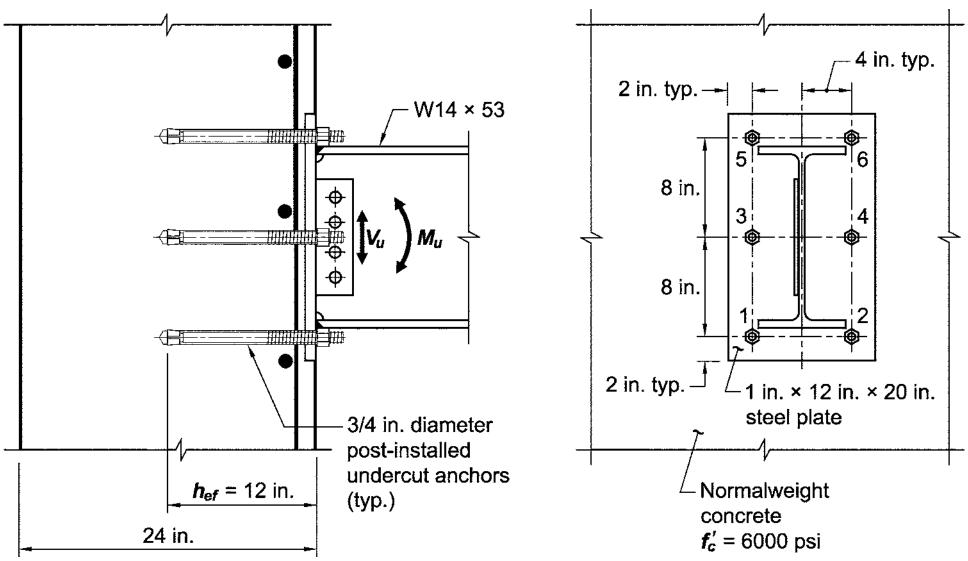


Fig. 4.26—Example 11: Beam support consisting of plate fastened with post-installed undercut anchors.

Note: As discussed in the Commentary R21.2.1, ACI 318-05, the design forces may be less than those corresponding to linear response at the anticipated earthquake intensity. The design loads for this connection should be established consistent with the requirements of a code, such as ASCE 7. Any reduction factor R_w *should consider the type of construction and the behavior required for the connection between the steel beam and the concrete.*

Step	Calculations and discussion	ACI 318-05 Section
1.	Check the connection shown in Fig. 4.26 for the given loading: In addition, the ductility requirements of Appendix D are verified for connections resisting seismic loads.	
2.	Design basis: a) Basic assumptions • Cracked concrete elastic design b) Materials • Steel plate: ASTM A36/A36M 1 in. x 12 in. x 20 in. • Steel beam: ASTM A992/A992M • Anchors: ° 3/4 in. diameter post-installed undercut anchor manufactured from carbon steel material conforming to the requirements of ASTM F1554 Grade 36, meeting the ductile steel requirements of ACI 318-05 Appendix D. ° Specified yield strength of anchor steel f_{ya} = 36 ksi ° Specified tensile strength of anchor steel f_{uta} = 58 ksi ° Nominal anchor diameter = 0.75 in. ° Effective anchor embedment h_{ef} = 12 in.	

3.	Requirements for design:	
	Proportion connection to satisfy D.3.3; either the design is governed by steel strength or the attachment will undergo ductile yielding before the anchors reach design strength.	D.3.3.5
	Checking the plastic capacity of the attached beam (F_y = 50 ksi):	
	Plastic section modulus for W14 x 53: Z_x = 87.1 in.^{3}	
	$\therefore\ M_{Pl}$ = (50)(87.1) = 4355 in.-kip	
	It is unpractical to develop the yield capacity of the beam. In this case, the anchor design should be controlled by the steel strength of the anchors.	D.3.3.4

4. Determine anchor forces: D.3.1

Tension: Anchor reactions are determined on the basis of elastic response.

Simplified statically determinate analysis: Assume that the compression reaction is located directly beneath the toe of the W14 beam (conservative) and that the steel plate exhibits rigid-body rotation (Fig. 4.27).

T_T = tension reaction on top anchors (5 and 6)
T_M = tension reaction of middle anchors (3 and 4)

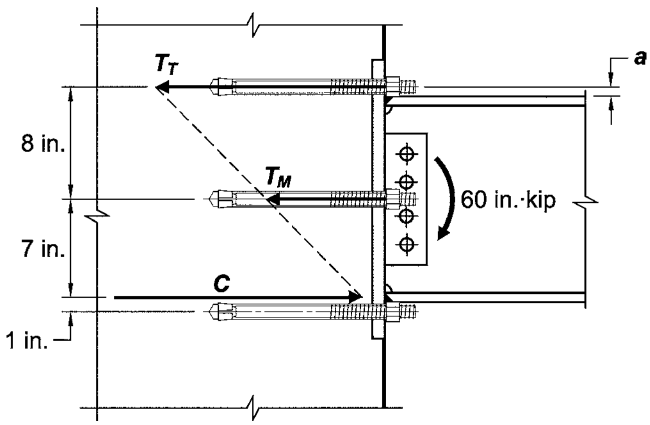


Fig. 4.27—Example 11: Tension load distribution.

$$\sum M = 0 \Rightarrow 15(2T_T) + 7(2T_M) = 60 \text{ kip-in.}$$

$$\frac{\Delta_T}{\Delta_M} = \frac{15}{7} \Rightarrow \Delta_T = \frac{15}{7}\Delta_M$$

$T_T = 2\Delta_T k = 2(2.14\Delta_M)k = 4.28\Delta_M k$ and $T_M = 2\Delta_M k$

$$\therefore T_T = 4.28\frac{T_M}{2} = 2.14T_M$$

Substituting:

$T_M = T_3 = T_4 = 0.77$ kips
$T_T = T_5 = T_6 = 1.65$ kips
$\therefore N_{ua} = 2(0.77) + 2(1.65) = 4.84$ kips

Check assumption of plate rigidity:

The 1 in. plate has a nominal yield moment capacity of:

$$M_y = S_x f_y = \frac{bh^2}{6} \times f_y = \frac{12(1)^2}{6} \times 36 \text{ ksi} = 72 \text{ in.-kip} > 60 \text{ in.-kip}$$

The assumption of the compression reaction at the toe is conservative. While the load distribution to the tension-loaded anchors will be affected by their proximity to the connected wide flange shape, such effects are likely to occur at larger rotations. Per D.3.1, the analysis is based on elastic anchor response. On the tension side, checking the moment at the beam face:

$$M_{face} = \sum T_T \times a = 2(1.65)(1) = 3.3 \text{ in.-kip} < 72 \text{ in.-kip} \text{ – OK}$$

5. Determine design tensile strength of the anchors, ϕN_n:

Calculate the nominal steel strength in tension, N_{sa}: — D.5.1.2

Check:

f_{uta} = 58 ksi < 1.9f_{ya} = 1.9 × 36 ksi = 68.4 ksi < 125 ksi – OK

Effective cross-sectional area of a single anchor:

A_{se} = 0.334 in.^{2} — Table A.4

Calculate nominal steel tensile strength:

$N_{sa} = A_{se} \times f_{uta}$ = (0.334)(58) = 19.37 kips (for a single anchor) — Eq. (D-3)

Calculate nominal concrete breakout strength in tension, N_{cbg}: — D.5.2

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

Determine the ratio of A_{Nc} for the group of tensioned anchors, T_3, T_4, T_5, and T_6, and A_{Nco} for a single anchor:

The projected concrete failure area A_{Nc}, shown as a shaded area on the figure, is calculated (Fig. 4.28).

h_{ef} = 12 in.

$A_{Nc} = (1.5h_{ef} + s_x + 1.5h_{ef})(1.5h_{ef} + s_y + 1.5h_{ef})$ — D.5.2.1
= (18 + 8 + 18)(18 + 8 + 18) = 1936 in.^{2}

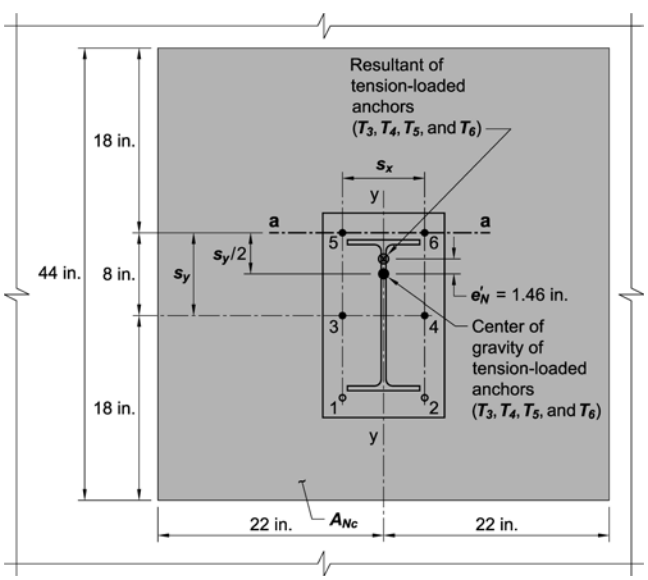


Fig. 4.28—Example 11: Determination of projected failure area and e'_N *for tension-loaded anchors.*

Determine A_{Nco} for a single anchor:

$A_{Nco} = 9h_{ef}^2 = 9(12)^2$ = 1296 in.^{2} (for a single anchor) — Eq. (D-6)

Determine the modification factors $\psi_{ec,N}$, $\psi_{ed,N}$, $\psi_{c,N}$, and $\psi_{cp,N}$:

$$\psi_{ec,N} = \frac{1}{\left(1 + \frac{2 \times e'_N}{3 \times h_{ef}}\right)} \leq 1.0$$ — Eq. (D-9)

5. (cont.)	Determine the eccentricity of the resultant anchor tension in y-direction with respect to the center of gravity of the anchors in tension by summing moments about *a-a* (Fig. 4.28)	
	$e_N' = \frac{s_y}{2} - \frac{(T_3 + T_4)s_y}{N_{ua}} = \frac{8}{2} - \frac{(0.77 + 0.77)8}{4.84} = 1.46 \text{ in.} < \frac{s_y}{2} = 4 \text{ in.} \therefore - \text{OK}$	Fig. RD.5.2.4(b)
	$\psi_{ec,N} = \frac{1}{\left(1 + \frac{2(1.46)}{3(12)}\right)} = 0.92$	
	Determine the edge modification factor $\psi_{ed,N}$:	
	Connection is remote from edges	
	$\psi_{ed,N} = 1.0$	
	Determine the modification factor for cracked concrete, $\psi_{c,N}$:	
	Per D.5.2.6, unless an analysis is performed to show no cracking at service loads, the concrete is assumed to be cracked for the purposes of determining the anchor design strength (this is a prudent assumption for most seismic design cases)	D.5.2.6
	$\psi_{c,N} = 1.0$	
	Determine the modification factor for post-installed anchor in uncracked concrete, $\psi_{cp,N}$: Per D.5.2.7, for post-installed anchor in cracked concrete	D.5.2.7
	$\psi_{cp,N} = 1.0$	
	Determine the basic concrete breakout strength, N_b, for a single anchor in tension:	D5.2.2
	$N_b = k_c\sqrt{f_c'}h_{ef}^{1.5}$	Eq. (D-7)
	$k_c = 24$	Table A.4
	$N_b = 24\sqrt{6000}(12)^{1.5} = 77{,}278 \text{ lb} = 77.28 \text{ kips}$	
	Substituting into Eq. (D-5):	
	$N_{cbg} = \frac{1936}{1296}(0.92)(1.00)(1.00)(1.00)(77.28) = 106.78 \text{ kips}$	
	Determine the pullout strength of anchors in tension, N_{pn}:	D.5.3
	$N_{pn} = \psi_{c,P} N_p$	Eq. (D-14)
	where	
	$\psi_{c,P} = 1.0$ (cracked concrete assumed)	D.5.3.6
	Determine the pullout strength of single anchor in tension, N_p:	D.5.3.4
	N_p for post-installed anchors is obtained from manufacturer's test data. From observing Table A.4, there are no reported values for pullout for this anchor at this embedment depth. However, for seismic loading, the pullout resistance is limited to N_{eq}. There are no reported pullout values for N_{eq}, therefore, pullout failure mode does not govern.	
	Determine side-face blowout tensile strength N_{sb}:	D.5.4
	Not applicable for post-installed anchors and there are no near edges.	
	Determine strength reduction factors applicable for the conditions:	
	The design tensile strengths are calculated with the calculated nominal strengths and strength reduction factors of D.4.4.	

5. (cont.)	Steel strength factor for ductile steel in tension:	
	$\phi = 0.75$	D.4.4(a)i
	$\phi N_{sa} = \phi n N_{sa} = (0.75)(4)(19.37) = 58.11$ kips	D.4.1.2
	Determine design concrete breakout strength in tension:	
	Strength reduction factor for concrete breakout for a Category 1 anchor, Condition B:	
	$\phi = 0.65$ (refer to Table A.4 for anchor category)	D.4.4(c)ii
	Note: The wall element in which the connection is embedded contains normal orthogonal reinforcement, which will not significantly interact with the tension failure surface of the tension-loaded anchors. Therefore, Condition B is assumed (supplementary reinforcement not provided).	D.4.4
	$\phi N_{cbg} = (0.65)(106.78) = 69.41$ kips	D.4.1.2
	The summary of tension design strengths and controlling mode of failure is shown in Table 4.18.	D.4.1.2
	Table 4.18—Summary of tension design strengths	D.3.3.4
6.	Design strength for tension anchors in a region of moderate or high seismic risk using the controlling value from Table 4.17:	D.3.3.3
	$0.75\phi N_n = 0.75(58.11) = 43.58$ kips	
	Note: If the provisions of ACI 318-08 are followed, the steel strength would be 58.11 kips.	

Table 4.18—Summary of tension design strengths

Failure mode	Anchor design strength	Calculated design strength, kip	Controlling failure mode
Steel	ϕN_{sa}	58.11	← Controls
Concrete breakout	ϕN_{cb}	69.41	—
Concrete pullout	ϕN_{pn}	Not applicable	—

7. Determine shear design strength ϕV_n:

Determine the nominal steel strength in shear, V_{sa}: D.6.1.2

$V_{sa} = n0.6A_{se}f_{uta} = 0.6(0.334)(58) = 11.62$ kips (for a single anchor) Eq. (D-20)

However, for seismic loading, the shear resistance, V_{eq} obtained from manufacturer's test data (Table A.4) is permitted.

$V_{eq} = 12$ kips for a single anchor (Table A.4)

$\therefore V_{sa} = nV_{eq} = (6)(12) = 72$ kips (for a group of six anchors)

Determine the anchor group nominal concrete breakout strength in shear, V_{cbg}:

Nominal concrete breakout strength in shear, V_{cbg}, not applicable since the anchors are not located close to a free edge. D.6.2.1 RD.6.2.1

Determine the anchor group nominal concrete pryout strength, V_{cpg}: D.6.3

$V_{cbg} = k_{cp}N_{cbg}$ Eq. (D-30)

Concrete breakout strength of four anchors (3, 4, 5, and 6):

Note: Since all six anchors are in shear, technically, N_{cbg} *of all six anchors should be determined. However, conservatively,* N_{cbg} *for the anchors in tension are taken for the calculation.*

$N_{cbg} = 106.78$ kips (refer to Step 5) Eq. (D-5)

$k_{cp} = 2.0$ for $h_{ef} > 2.5$ in.

$V_{cpg} = 2.0 \times 106.78 = 213.56$ kips

Determine the strength reduction factors applicable for the conditions:

The shear design strengths are calculated with the calculated nominal strengths and strength reduction factors of D.4.4.

Strength reduction factor for ductile steel in shear

$\phi = 0.65$ D.4.4(a)ii

$\phi V_{sa} = (0.65)(72) = 46.8$ kips D.4.1.2

Strength reduction factor for pryout

$\phi = 0.70$ D.4.4

$\phi V_{cpg} = (0.70)(213.56) = 149.49$ kips D.4.1.2

The summary of shear design strengths and controlling mode of failure is shown in Table 4.19. D.4.1.2

Table 4.19—Summary of shear design strengths D.3.3.4

Failure mode	Anchor design strength	Calculated design strength, kip	Controlling failure mode
Steel	ϕV_{sa}	46.8	← Controls
Concrete breakout	ϕV_{cbg}	149.49	—

8. Design strength for shear in a region of moderate or high seismic risk using controlling value in Table 4.19: D.3.3.3

$0.75\phi V_n = 0.75(46.8) = 35.1$ kips

Note: If the provisions of ACI 318-08 are followed, the steel strength would be 46.8 kips.

9.	Check tension and shear interaction: V_{ua} = 20 kips > 0.2(0.75ϕV_n) = 0.2(35.1 kips) = 7.02 kips N_{ua} = 4.84 kips < 0.2(0.75ϕN_n) = 0.2(43.58 kips) = 8.71 kips $\frac{N_{ua}}{0.75\phi N_n} + \frac{V_{ua}}{0.75\phi V_n} = \frac{4.84}{43.58} + \frac{20}{35.1} = 0.68 \leq 1.2$ – OK If the provisions of ACI 318-08 are followed: N_{ua} = 4.84 kips < 0.2(58.11) = 11.62 kips (full strength in shear is permitted) 46.8 kips shear design strength > 20 kips – OK	D.7 D.7.1 D.7.2 Eq. (D-31) D.7.2
10.	Minimum member thickness check: $h_{ef} \leq$ the greater of 2/3 member thickness or the member thickness less 4 in. 12 in. ≤ 16 in. or 20 in. – OK Thus, the connection using six post-installed undercut anchors (diameter = 0.75 in. and h_{ef} = 12 in.) is adequate to support the applied loads. The anchor design is governed by the steel failure mode of a ductile steel element and, therefore, meets the provisions of ACI 318-05 Section D.3.3.	D.8.5

4.12—Example 12: Multiple headed anchor connection subjected to seismic moment and shear

Check the design of an embedded plate with a group of eight welded headed studs spaced as shown in Fig. 4.29 to support a factored reversible bending moment M_{ua} of 30,000 ft-lb (360 in.-kip) and a factored reversible end reaction V_{ua} of 60,000 lb (60 kips) (based on elastic analysis referred to in the discussion following). The loading results from an earthquake in a region of moderate or high seismic risk. The connection is located far away from any edges of the concrete member; assume it to be in a shear wall (Fig. 4.29).

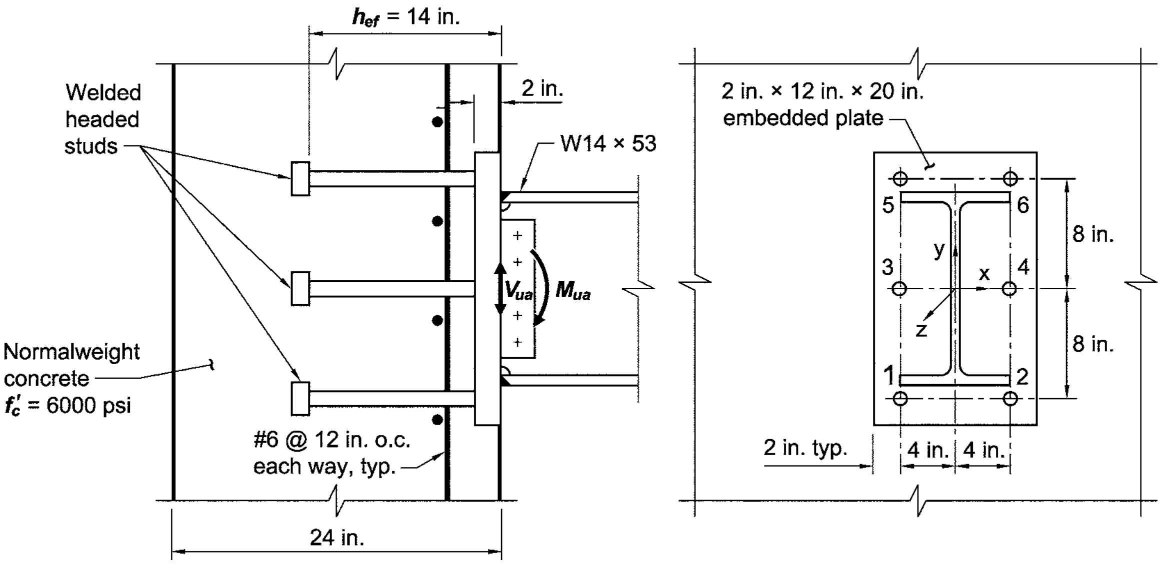


Fig. 4.29—Example 12: Beam support consisting of embedded plate with welded studs.

As discussed in the ACI 318-05 Commentary R21.2.1, the design forces may be less than those corresponding to linear response at the anticipated earthquake intensity. The design loads for this connection should be established consistent with the requirements of a code, such as ASCE 7. Any reduction factor R_w should consider the type of construction and the behavior required for the steel beam to concrete connection.

Step	Calculations and discussion	ACI 318-05 Section
1.	Check the connections shown in Fig. 4.29 for the given loading and ductility requirements of Appendix D for resisting seismic loads in a region of moderate or high seismic risk.	
2.	Design basis: a) Basic assumptions • Cracked concrete • Plastic design b) Materials • Embedded plate: ASTM A36/A36M, 2 in. x 12 in. x 20 in. (t_p = 2 in.) • Anchors: ° 3/4 in. diameter x 13-3/16 in. AWS D1.1 Type B mild steel welded headed stud ° Specified yield strength of anchor steel f_{ya} = 51 ksi ° Specified tensile strength of anchor steel f_{uta} = 65 ksi ° Stud head diameter d_h = 1.25 in. ° Stud head thickness t_h = 3/8 in. ° Reduction in stud length due to welding ~ 3/16 in. ° Effective anchor embedment: h_{ef} = 13-3/16 in. – 3/8 in. – 3/16 in. + 2 in. = 14.63 in. Assume h_{ef} = 14 in.	

3.	Requirements for design: Connection should be proportioned to satisfy D.3.3; either the anchor design is governed by steel strength or the attachment will undergo ductile yielding before the anchors reach design strength. Checking the plastic capacity of the attached beam (assume F_y = 50 ksi): Z = 87.1 in.^{3}, M_{pl} = 87.1 × 50 = 4355 in.-kip	D.3.3.5
	Clearly, it is not practical to develop the plastic strength of the beam. In this case, the anchor design should be controlled by steel strength of the anchors.	D.3.3.4

4. Determine anchor forces: D.3.1

Tension: Anchor forces are determined on the basis of plastic analysis.

Assume a simplified elastic statically determinate analysis, that the compression reaction is located directly beneath the toe of the W14 beam (conservative), and that the embedded plate is stiff enough to exhibit rigid-body rotation (Fig. 4.30).

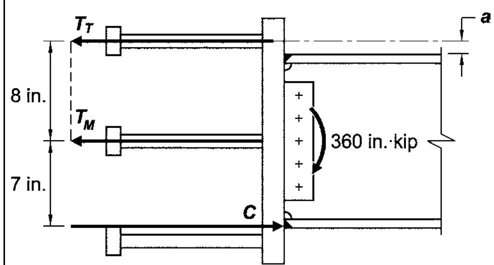


Fig. 4.30—Example 12: Tension load distribution.

T_T = tension force on top anchors (5 and 6)

T_M = tension force on middle anchors (3 and 4)

$$\sum M = 0 \Rightarrow 15(2T_T) + 7(2T_M) - 360 = 0$$

$30T_T + 14T_M$ = 360 in.-kip

Because plastic behavior is assumed, all tension anchors have equal force:

$T_T = T_M$

Substituting into the above equation:

$$T_T = T_M = \left(\frac{360 \text{ kip-in.}}{44}\right) = 8.18 \text{ kip}$$

Check assumption of plate rigidity:

$\therefore N_{ua}$ = 4(8.18) = 32.72 kip

The 2 in. embedded plate has a nominal yield moment capacity of:

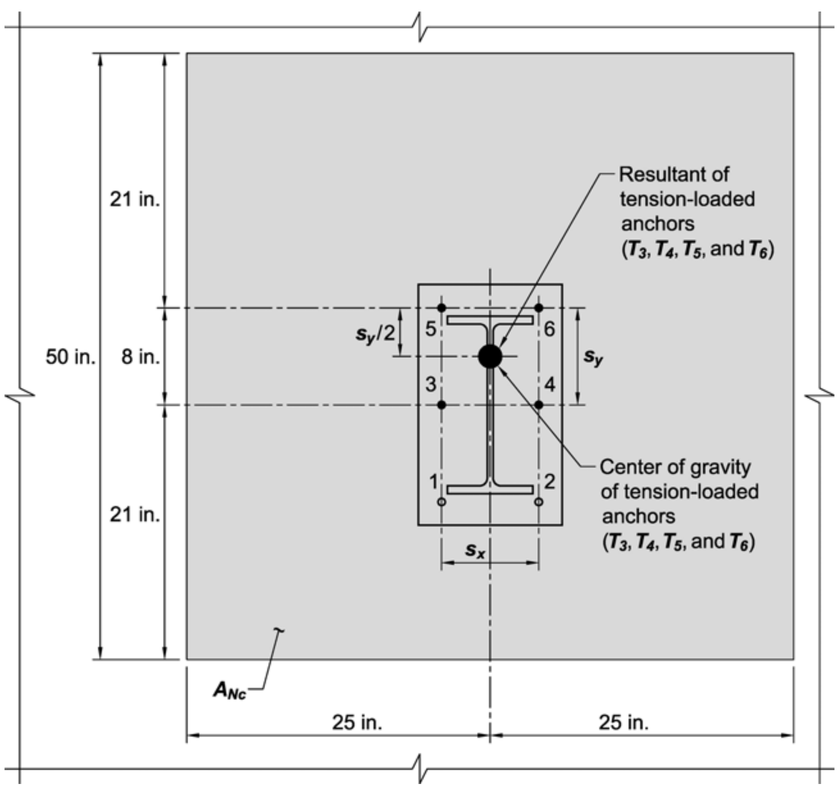
$$M_y = S_x f_y = \frac{bh^2}{6} \times f_y = \frac{12(2.0)^2}{6} \times 36 \text{ ksi} = 288 \text{ in.-kip}$$

However, the plastic moment capacity of the plate is given by:

$$M_{pl} = f_{pl} M_y = 1.5M_y = 1.5 \times 288 \text{ in.-kip} = 432 \text{ in.-kip}$$

Since the yield capacity of the embedded plate at the toe of the connected shape is close to the applied moment, the stiffness of the plate should minimize the prying forces in this connection. The assumption of the compression reaction at the toe is reasonable and conservative. While the load distribution to the tension-loaded anchors are also be affected by their proximity to the connected wide flange shape, such effects are likely to occur at larger rotations. Per D.3.1, plastic analysis is permitted. On the tension side, checking the moment in the plate at the top face of the beam flange:

$$M_{face} = \sum T_T \times a = 2(8.18 \text{ kip})(1 \text{ in.}) = 16.36 \text{ in.-kip} \leq 288 \text{ in.-kip} \therefore \text{ – OK}$$

5.	Determine design tensile strength of the anchors, ϕN_n:	
	Calculate the nominal steel strength in tension, N_{sa}:	D.5.1.2
	check:	
	f_{uta} = 65 ksi ≤ 1.9f_{ya} = 1.9 × 51 = 96.9 ksi < 125 ksi – OK	D.5.1.2
	Effective cross-sectional area of anchor:	
	$A_{se} = \frac{\pi \times d^2}{4} = \frac{\pi \times 0.75^2}{4} = 0.442 \text{ in.}^2$	
	Nominal steel tensile strength:	
	$N_{sa} = A_{se} \times f_{uta}$ = (0.442)(65) = 28.73 kips (for a single anchor)	Eq. (D-3)
	Calculate the nominal concrete breakout strength in tension, N_{cbg}:	D.5.2
	$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} = \psi_{ec,N}\psi_{c,N}\psi_{cp,N}N_b$	Eq. (D-5)
	Determine the ratio of A_{Nc} for the tension anchor group, T_3, T_4, T_5, and T_6, shown in Fig. 4.31, and A_{Nco} for a single anchor.	
	The projected concrete failure area A_{Nc}, shown as a shaded area on the following figure, is calculated as follows (Fig. 4.31):	
	h_{ef} = 14 in.	
	$A_{Nc} = (1.5h_{ef} + s_x + 1.5h_{ef})(1.5h_{ef} + s_y + 1.5h_{ef})$ = (21 + 8 + 21)(21 + 8 + 21) = 2500 in.^{2}	D.5.2.1
		
	Fig. 4.31—Example 12: Tension load distribution.	
	Determine A_{Nco} for single anchor:	D.5.2.1
	$A_{Nco} = 9(h_{ef})^2 = 9(14)^2 = 1764 \text{ in.}^2$ (for a single anchor)	Eq. (D-6)

5. (cont.)	Determine the modification factors $\psi_{ec,N}$, $\psi_{ed,N}$, $\psi_{c,N}$, and $\psi_{cp,N}$.	
	Determine the eccentricity modification factor $\psi_{ec,N}$:	
	$\psi_{ec,N} = \dfrac{1}{\left(1 + \dfrac{2e'_N}{3h_{ef}}\right)} \leq 1.0$	Eq. (D-9)
	Because plastic analysis is being used, all tension anchors have equal force, and the resultant of the four tension anchors is concentric with respect to the center of gravity of the anchors group, and the eccentricity e'_N is zero (Fig. 4.31).	
	$\psi_{ec,N} = 1.0$	Fig. RD.5.2.4(b)
	Determine the edge modification factor $\psi_{ed,N}$:	
	Connection is remote from edges:	
	$\psi_{ed,N} = 1.0$	
	Determine the modification factor for uncracked concrete, $\psi_{c,N}$:	
	$\psi_{c,N} = 1.0$	D.5.2.6
	Determine the modification factor for post-installed anchor in uncracked concrete, $\psi_{cp,N}$:	
	Since it is not a post-installed anchor	
	$\psi_{cp,N} = 1.0$	D.5.2.7
	Determine the concrete breakout strength N_b for a single anchor in tension:	D.5.2.2
	$N_b = 16\sqrt{f'_c}\,h_{ef}^{5/3}$ when 11 in. $\leq h_{ef} \leq$ 25 in.	Eq. (D-8)
	h_{ef} = 14 in.	
	$N_b = 16\sqrt{6000}(14)^{5/3}$ = 100.8 kips	
	Substituting into Eq. (D-5) for the group capacity:	
	$N_{cbg} = \dfrac{2500}{1764}(1.0)(1.0)(1.0)(1.0)(100.8)$ = 142.9 kips	
	Determine the pullout strength of anchors in tension, N_{pn}:	D.5.3
	$N_{pn} = \psi_{c,P}N_p$	Eq. (D-14)
	where	
	$\psi_{c,P}$ = 1.0 (cracked concrete assumed)	D.5.3.6
	Determine pullout capacity of single anchor in tension, N_p:	D.5.3.4
	$N_p = 8A_{brg}f'_c$	Eq. (D-15)
	Determine bearing area of the head of a single anchor:	
	$A_{brg} = \dfrac{\pi(D_h^2 - d^2)}{4} = \dfrac{\pi(1.25^2 - 0.75^2)}{4}$ = 0.785 in.^{2}	
	N_p = 8(0.785)(6000) = 37.6 kips	
	Substituting into Eq. (D-14):	
	N_{pn} = (1.0)(37.6) = 37.6 kips (for a single anchor)	

5. (cont.)

Determine side-face blowout tensile capacity N_{sb}:

Not applicable since there are no near edges. D.5.4

Determine strength reduction factors applicable for the conditions:

The design tensile strengths are determined with the calculated nominal strengths and strength reduction factors of D.4.4. Determine the design steel tensile strength:

Strength reduction factor for ductile steel in tension

$\phi = 0.75$ D.4.4(a)i

Per AWS D1.1, Type B studs qualify as a ductile steel element (20% minimum elongation in 2 in.).

$\phi N_{sa} = \phi n N_{sa} = 0.75(4)(28.73) = 86.2$ kips D.4.1.2

Determine design concrete breakout strength in tension:
Strength reduction factor for concrete breakout, Condition B:

$\phi = 0.70$ D.4.4(c)ii

Note: The wall element in which the connection is embedded contains normal orthogonal reinforcement, which will not significantly interact with the tension failure surface of the tension-loaded anchors. Therefore, Condition B is assumed (supplementary reinforcement not provided). D.4.4

$\phi N_{cbg} = 0.8(142.9) = 100.03$ kips D.4.1.2

Determine design concrete pullout strength in tension:

Strength reduction factor for pullout

$\phi = 0.70$ D.4.4(c)ii

$\phi N_{pn} = \phi n_t N_{pn} = 0.70(4)(37.6) = 105.3$ kips D.4.1.2

The summary of tension design strengths and controlling mode of failure is shown in Table 4.20. D.4.1.2

Table 4.20—Summary of tension design strengths D.3.3.4

Failure mode	Anchor design strength	Calculated design strength, kip	Controlling failure mode
Steel	ϕN_{sa}	86.2	← Controls
Concrete breakout	ϕN_{cbg}	100.03	—
Concrete pullout	ϕN_{pn}	105.3	—

Note: The design strength of steel shown in Table 4.20 is exceeded by the design strength of concrete breakout and concrete pullout. For plastic design, it may be prudent to require that the nominal strength of steel N_{sa} *(28.73 kips[4 anchors] = 114.92 kips) may be exceeded by the design strength of concrete breakout and concrete pullout. This would require an increase in embedment from 14 in. to about 15.5 in. for concrete breakout design strength equal to 114 kips. Concrete pullout design strength is unchanged by increasing embedment depth.*

6.

Design strength for tension in a region of moderate or high seismic risk: D.3.3.3

$0.75\phi N_n = 0.75(86.2) = 64.65$ kips

If the provisions of ACI 318-08 are followed, the steel strength is 86.2 kips.

7.	Determine shear design strength ϕV_n:	D.6.1.2
	The contribution to the shear capacity of the anchorage by the concrete located in front of the embedded plate is neglected.	
	Nominal steel strength in shear, V_{sa}, for six anchors:	
	$V_{sa} = nA_{se}f_{uta} = 6(0.442)(65) = 172.4$ kips	Eq. (D-19)
	Nominal concrete breakout strength in shear, V_{cbg}:	D.6.2.1
	Not applicable since there are no proximate edges.	
	Determine the anchor group nominal pryout strength V_{cpg}:	D.6.3
	Concrete pryout strength V_{cpg}:	
	$V_{cpg} = k_{cp} N_{cbg}$	Eq. (D-30)
	Concrete breakout strength of four anchors (3, 4, 5, and 6):	
	Note: As all six anchors are in shear, technically N_{cbg} *of all six anchors needs to be determined. However, conservatively,* N_{cbg} *for the anchors in tension are taken for the calculation.*	
	$N_{cbg} = 142.9$ kips	Eq. (D-5)
	$k_{cp} = 2.0$ for $h_{ef} \geq 2.5$ in.	
	$V_{cpg} = 2.0 \times 142.9 = 285.8$ kips	
	Determine shear design strength of the anchors:	
	The shear design strength is determined with the calculated nominal strengths and strength reduction factors of D.4.4	
	Per AWS D1.1, Type B studs qualify as a ductile steel element (20% minimum elongation in 2 in.).	D.1
	Strength reduction factor for ductile steel in shear:	
	$\phi = 0.65$	D.4.4(a)ii
	$\phi V_{sa} = 0.65(172.4) = 112.0$ kips	D.4.1.2
	Strength reduction factor for concrete pryout:	
	$\phi = 0.70$	D.4.4(c)i
	$\phi V_{cpg} = 0.70(285.8) = 200.0$ kips	
	The summary of shear design strengths and controlling mode of failure is shown in Table 4.21.	D.4.1.2
	Table 4.21—Summary of shear design strengths	D.3.3.4
	(see Table 4.21 below)	
	Steel strength should control for a region of moderate or high seismic risk and this is shown in Table 4.21.	
8.	Design strength for shear in a region of moderate or high seismic risk:	
	$0.75\phi V_n = 0.75(112.0) = 84.0$ kip	D.3.3.3
	If the provisions of ACI 318-08 are followed, the steel strength is 112 kips.	

Table 4.21—Summary of shear design strengths

Failure mode	Anchor design strength	Calculated design strength, kip	Controlling failure mode
Steel	ϕV_{sa}	112.0	← Controls
Concrete breakout	ϕV_{cbg}	200	—

9.	Check tension and shear interaction: V_{ua} = 60 kips > 0.2ϕV_n = 0.2(84) = 16.8 kips N_{ua} = 32.7 kips > 0.2ϕN_n = 0.2(64.65) = 12.93 kips $\therefore \frac{N_{ua}}{0.75\phi \times N_n} + \frac{V_{ua}}{0.75\phi \times V_n} = \frac{32.7}{64.65} + \frac{60}{84.0} = 1.22 > 1.2$ Interaction is not met and design strength should be increased. It can be achieved by using larger diameter anchors, deeper embedment, or in combination. If the provisions of ACI 318-08 are followed using the increased controlling design strengths: V_{ua} = 60 kips > 0.2(112) = 22.4 kips N_{ua} = 32.7 kips > 0.2(86.2) = 17.24 kips $\therefore \frac{N_{ua}}{\phi \times N_n} + \frac{V_{ua}}{\phi \times V_n} = \frac{32.7}{86.2} + \frac{60}{112} = 0.92 \le 1.2$ – OK Using ACI 318-08 (without the 0.75 reduction factor applied to the ductile steel design), the anchorage satisfies the interaction requirements as opposed to ACI 318-05.	D.7 D.7.1 D.7.2 Eq. (D-31)

4.13—Example 13: Group of tension anchors on a pier with shear lug

Design the anchors and shear lug for the steel column located atop the concrete pedestal shown in Fig. 4.32. Tension is resisted by the anchors, and all the shear is resisted by the shear lug. This is a common design situation encountered in industrial facilities. The pedestal, in this example, could just as easily have been a wall or pilaster. The challenge here is to design all of the elements to work properly together while making certain that the design is constructible. This example goes beyond the provisions of Appendix D and uses other provisions such as ACI 349 and the American Institute of Steel Construction (AISC) *Steel Design Guide Series 1: Column Base Plate*. Additional provisions are used to evaluate the shear lug. Be cautioned to review these other documents to fully understand their limitations.

Given the:

- Pedestal geometry shown in Fig. 4.32;
- Pedestal vertical reinforcement of 16 No. 8 bars (designed per ACI 318 for tension plus bending at the top of footing);
- Pedestal transverse tie sets of No. 4 bars are arranged as shown;
- Concrete strength is f_c' = 4000 psi and the reinforcement strength f_y = 60,000 psi;
- Grout strength exceeds the concrete strength;
- W12 x 58 column with 16 in. square base plate is 1-1/2 in. thick;
- All plate steel is ASTM A36/A36M grade;
- Four anchors used, as shown in Fig. 4.32, are ASTM F1554 Grade 55 material;
- Anchors used have a double heavy hex nut at the embedded end;
- Anchors are to be a ductile design;
- Service dead load (DL) = 200 kips; and
- Service wind load (WL) = 230 kips uplift and 40 kips shear (acting as shown in Fig. 4.32).

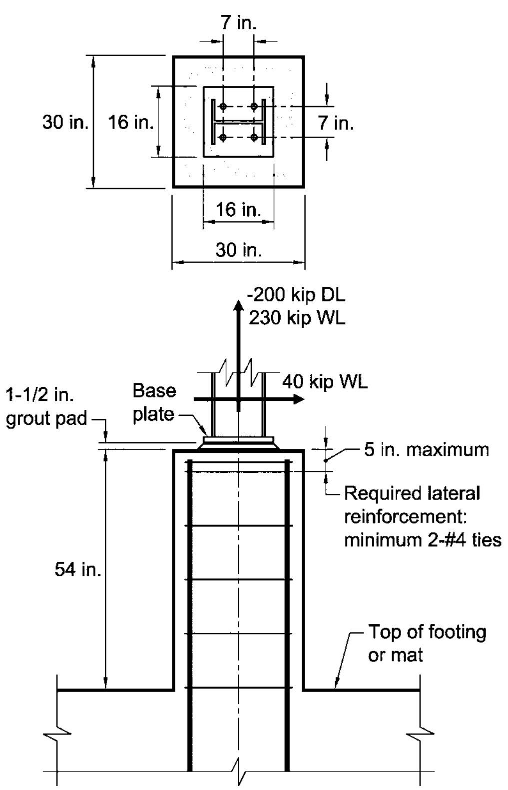


Fig. 4.32—Example 13: Group of tension anchors on a pier with shear lug.

Design ultimate loads

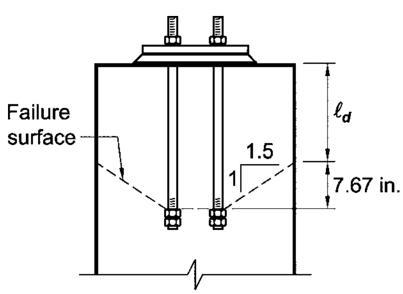
Load combinations are from Section 9.2 of ACI 318. Combination (9-6) controls for tension and shear on the anchors.

Tension

$$T_u = 0.9D + 1.6W + 1.6H = 0.9(200 \text{ kips}) + 1.6(–230 \text{ kips}) = –188 \text{ kips (upward)}$$

Shear

$$V_u = 0.9D + 1.6W + 1.6H = 1.6(40 \text{ kips}) = 64 \text{ kips}$$

Step	Calculations and discussion	ACI 318-05 Section
1.	Select the anchor diameter to resist the factored load using Section D.5.1 (ACI 318). Use 1-1/4 in. diameter anchor: ASTM F1554 Grade 55 is ductile steel. Check the assumed anchor diameter by checking the tensile capacity. $\phi N_{sa} = \phi n A_{se} f_{uta}$ $\phi = 0.75$ (for tension) $A_{se} = 0.969$ in.^{2} (from Table A.2(a) of this guide) $f_{uta} = 75$ ksi (from Table A.1 of this guide) $\phi N_{sa} = 0.75(4)(0.969 \text{ in.}^2)(75 \text{ ksi}) = 218 \text{ kips} > 188 \text{ kips}$ – OK	 Eq. (D-3) D.4.4
2.	Make initial estimate of embedment depth: Estimate the required embedded length h_{ef} at 12 diameters. Given the 7 in. anchor spacing and 30 in. pier, 12 diameters equals h_{ef} of ~15 in., which will include the entire area of the pier in the projected failure area A_{Nc}. *Note: In Section RD.5.2 (ACI 318), the failure surface projects outward 1.5 units per every one unit of depth (~35 degrees from the horizontal axis) from the centerline of the anchor. This failure surface intersects the outside surface of the pier 7.67 in. above the anchor head-bearing surface as shown in Fig. 4.33.* Since the tension force in this example places the entire concrete pier section in tension, it is doubtful that the concrete breakout capacity will be sufficient to resist the applied load (Step 3 to verify calculation). In this case, the load path should be from the anchors to the vertical pier reinforcement. The transfer of force from anchors to reinforcement requires development of the reinforcing bars into the concrete breakout cone. The No. 8 vertical bars have a tension development length of approximately 48 in. For now, provide h_{ef} such that a No. 6 bar with hook could be developed into the failure cone as shown in Fig. 4.33, within a distance of ℓ_d.  *Fig. 4.33—Example 13: Assumed failure cone in pier.* Allowing 10 in. to develop a No. 6 hooked bar (including the length reduction factor of 0.7 from ACI 318 Section 12.5.3(a)) in the breakout prism and 2 in. for cover at the top of pier: $h_{ef} = 10 + 2 + 7.67 = 19.67$ in. Use 20 in. as the embedment depth for the anchors.	
3.	Determine concrete breakout capacity of anchor group: Use Section D.5.2 (ACI 318) to calculate the concrete breakout capacity. *Note: This step is included for illustrative purposes only since it is apparent that the concrete breakout load should be resisted by supplemental reinforcement. According to Section D.4.4(a)i and D.4.4(c)i (ACI 318), the strength reduction factor ϕ will be 0.75 for shear and tension since Condition A applies.* The group tension capacity is: $\phi N_{cbg} = \phi \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$	

4.	Determine the maximum embedment depth allowed: Step 4 is required when the actual embedment depth is used, making the calculation overly conservative. $c_{a,max}$ = (30 – 7)/2 = 11.5 in.; 1.5h_{ef} = 1.5(20) = 30 in. $c_{a,max}$ = 11.5 in. < 1.5h_{ef} = 30 in. ∴ Per D.5.2.3 (ACI 318), limit the embedment depth used in Eq. (D-4) through (D-11). $h_{ef} > s_{max}/3$ = 7/3 =2.33 in. $h_{ef} = c_{a,max}/1.5$ = 11.5/1.5 = 7.67 in. (for use in Eq. (D-4) through (D-11)) h_{ef} maximum of $c_{a,max}/1.5$ or $s_{max}/3$ h_{ef} = 7.67 in.	
5.	Determine the concrete breakout areas to account for spacing: a) Determine concrete breakout area for anchor group: A_{Nc} = (30 in.)^2 = 900 in.^{2} Since 1.5h_{ef} = 30 in., by inspection the projected area includes the entire pier area. b) Determine the concrete breakout area for a single anchor: $A_{Nco} = 9h_{ef}$ A_{Nco} = 9(7.67 in.)^2 = 529 in.^{2}	 Eq. (D-6)
6.	Determine the single anchor tension breakout capacity: $N_b = k_c\sqrt{f'_c}\,h_{ef}^{1.5}$ *Note: Although the anchors have an embedment depth of about 20 in., D.5.2.3 (ACI 318) reduces the embedment depth because of the multiple side effects, requiring the use of Eq. (D-7) instead of Eq. (D-8).* k_c = 24 for cast-in anchors $N_b = 24\sqrt{4000}\,(7.67)^{1.5}$ = 32,243 lb	 Eq. (D-7)
7.	Determine the eccentricity modification factor $\psi_{ec,N}$: $\psi_{ec,N}$ = 1.0 Factor equals 1, since there is no eccentric load.	 Eq. (D-9)
8.	Determine the edge distance modification factor $\psi_{ed,N}$: $\psi_{ed,N} = 0.7 + 0.3\dfrac{c_{a,min}}{1.5h_{ef}}$ Factor applies due to close proximity of edges. $\psi_{ed,N} = 0.7 + 0.3\dfrac{11.5}{1.5(7.67)} = 1.0$	 Eq. (D-11)
9.	Determine the cracking modification factor $\psi_{c,N}$: $\psi_{c,N}$ = 1.25 By inspection of the magnitude and direction of the applied factored loads (axial tension and shear only), it is concluded that at service loads there will be no cracking along the axis of the anchors. Since there is no cracking that would affect the concrete breakout capacity, the cracking modification factor equals 1.25.	D.5.2.6

10. Calculate the anchor group concrete tension breakout capacity: D.5.2.1

$$\phi N_{cbg} = 0.75\frac{900}{529}(1.0)(1.0)(1.25)(32{,}243) = 51{,}427 \text{ lb} = 51.4 \text{ kips} \qquad \text{Eq. (D-5)}$$

Clearly, the concrete tension breakout capacity is not adequate to resist the factored tension load. Supplemental reinforcement is required.

11. Determine the anchor pullout capacity: D.5.3

$$\phi n N_{pn} = \phi n \psi_{c,p} N_p \qquad \text{Eq. (D-14)}$$

$$N_p = 8A_{brg}f_c' \qquad \text{Eq. (D-15)}$$

A_{bg} = 2.237 in.^{2} (Table A.2(a))

12. Determine the cracking modification factor $\psi_{c,P}$: D.5.3.6

$\psi_{c,p} = 1.4$

Factor equals 1.4 since there is no cracking that would affect bearing at the embedded head of the anchor.

13. Calculate the single anchor pullout capacity N_p:

$N_p = 8(2.237)(4000) = 71{,}584$ lb = 71.5 kips

Calculate the anchor group pullout capacity:

$\phi n N_{pn} = 0.75(4)(1.4)(71{,}584) = 300{,}652$ lb = 300.7 kips

14. Check side-face blowout: D.5.4

c_{a1} = 11.5 in. (edge distance)

$0.4h_{ef} = 0.4(20) = 8$ in. $< c_{a1} = 11.5$ in.

$\therefore$ Tension capacity is not affected by side-face blowout.

15. Summary of anchor capacity for tension load is shown in Table 4.22.

Table 4.22—Summary of tension design strengths

Failure mode	Anchor design strength	Calculated design strength, kip	Controlling failure mode
Steel	ϕN_{sa}	218	—
Concrete breakout	ϕN_{cbg}	51.4	← Controls
Concrete pullout	ϕN_{pn}	300.7	—
Concrete side-face blowout	ϕN_{sbg}	Not applicable	—

Since the concrete breakout capacity controls and is less than the factored tension of 188 kips, reinforcement is required to be anchored on both sides of the breakout planes. In addition, a ductile design is required as part of the problem statement. A ductile design requires that overall capacity be limited by the anchor's steel capacity, which is 218 kips. RD.4.2.1

Reinforcement design

16. Design the reinforcement from the many possible options:

The challenge is to make certain that the design is constructible. Often, the design will have to be sketched to scale to check for congestion and interferences. It is advisable to coordinate with a member of the construction team.

This example requires vertical reinforcement to constrain the concrete failure prism. Reinforcement should be developed on both sides of the failure plane. As previously discussed, the No. 8 vertical bars have an approximate tension development length of 48 in. (ACI 318 Section 12.2). Since the effective embedment h_{ef} is 20 in., a straight No. 8 bar cannot be developed. One option, which will not be examined in this example, would be to make the anchors long enough to develop the No. 8 bars; if there were an excess area of No. 8 bars, then the development length could be reduced in accordance with ACI 318 Section 12.2.5. Several other options will be evaluated.

The area of reinforcement required to develop the anchor capacity of 218 kips is: 9.3.2.1

$$A_s = \frac{218\text{ kips}}{0.9(60\text{ ksi})} = 4.04\text{ in.}^2$$

No. 8 bars equivalent to A_s = 4.04/0.79 = 5.11; use six bars

No. 6 bars equivalent to A_s = 4.04/0.44 = 9.18; use 10 bars

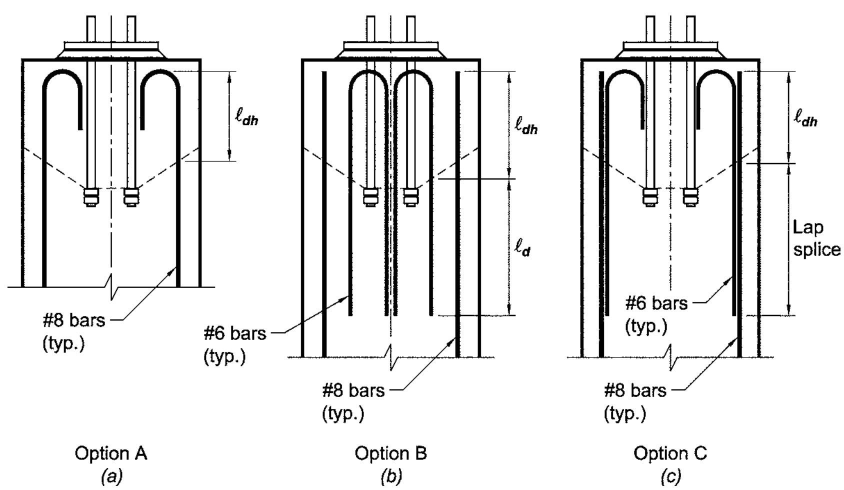


Fig. 4.34—Example 13: Placement of reinforcement.

Figure 4.34 shows three of several reinforcement options. One option not shown is the use of headed reinforcing bars for the No. 6 and No. 8 bar alternatives. A general discussion of the three options considered follows:

a) Option (A) requires a 180-degree hook at the top of each No. 8 bar. The development length for a standard hook on a No. 8 bar is approximately 14 in. (including the length reduction factor of 0.7 from ACI 318 Section 12.5.3(a)). Since there is an excess of No. 8 bars for developing the anchors (six bars required, 16 bars in the pier), the development length may be reduced according to ACI 318 Section 12.5.3(d):

$$\ell_{dh} = (14\text{ in.})\frac{A_s\text{ (required)}}{A_s\text{ (provided)}} = (14\text{ in.})\frac{4.04}{16(0.79)} = 4.47\text{ in.}$$ 12.5.3(d)

However, ℓ_{dh} should not be less than $8d_b$ or 6 in.

ℓ_{dh} = 8(1.0 in.) = 8 in.

Since 10 in. was previously allotted for bar development, the No. 8 bars with hooks will work.

The "J" dimension, or width of the hook, is 8 in., which puts the hooks in close proximity to the anchors. Also, the hooks should be rotated to avoid interfering with one another at the corners of the pier. As shown in Fig. 4.34, the No. 8 bars are continuous into the foundation below the pier; therefore, once the No. 8 bars are cast into the foundation, it will be difficult to adjust the hooks at the top to accommodate the anchors and the shear lug blockout. For this example, Option (A) is ruled out.

16. (cont.)	b) Option (B) requires No. 6 U-bars (or hairpins) to be installed adjacent to the anchors. Since 10 No. 6 bars are required to develop the anchors, five U-bars would have to be placed. This option allows the No. 6 U-bars bars to be placed at the same time as the anchors and shear lug blockout. Unfortunately, the U-bars will have to compete for space with the anchors, shear key blockout, and lateral tie sets. In addition, construction will have to take additional measures to secure the U-bars during the casting of concrete. For this example, Option (B) is ruled out. c) Option (C) is the solution chosen for this example. This option makes use of hooked No. 6 bars that are lap spliced with the No. 8 pier reinforcement. The required number of No. 6 bars is 10, but 12 bars will be provided for convenience because there are three No. 8 bars between corners in the pedestal (three on each side of the pier). The hooked bars may be placed at the same time as the anchors and shear key blockout, which will permit adjustment of the hooks to avoid congestion and interferences. Also, the hooked bars will not interfere with the interior legs of the lateral tie sets. The lap splice for the No. 6 bars should be Class B, and the required lap length is approximately 37 in., according to ACI 318 Section 12.15. Therefore, the overall length of the hooked bars is 10 in. (the development length) plus 37 in. (the lap splice length) plus 7.67 in. due to slope of shear cone (Fig. 4.33 and 4.34(c)), which equals 54 in. The 54 in. pier height is sufficient to accommodate the hooked bars. Use lateral reinforcement as required in Section 7.10.5.6 (ACI 318) to encompass the anchors in the top of a pier, column, or both: As shown in Fig. 4.32, two sets of No. 4 ties will be located within the top 5 in. of the pier.	

Shear lug design

A single cantilever type shear lug is proposed to transfer the entire shear load to the top of the concrete pier. For convenience, the lug will use the same 1-1/2 in. thick plate used for the column base plate. The methodology used in this example is based on ACI 349, Appendix D and AISC-*Load and Resistance Factor Design (LRFD)* *Specification Manual of Steel Construction, Design Guide 1.*

As a matter of good practice, the shear lug should be embedded a minimum of 2 in. into the concrete pier. The 1-1/2 in. thick grout pad between the base plate and pier top is considered to be ineffective for transfer of shear. The shear lug should, therefore, be a minimum of 3-1/2 in. deep. Use 2 in. of concrete embedment per 12 in. lug length (Fig. 4.35).

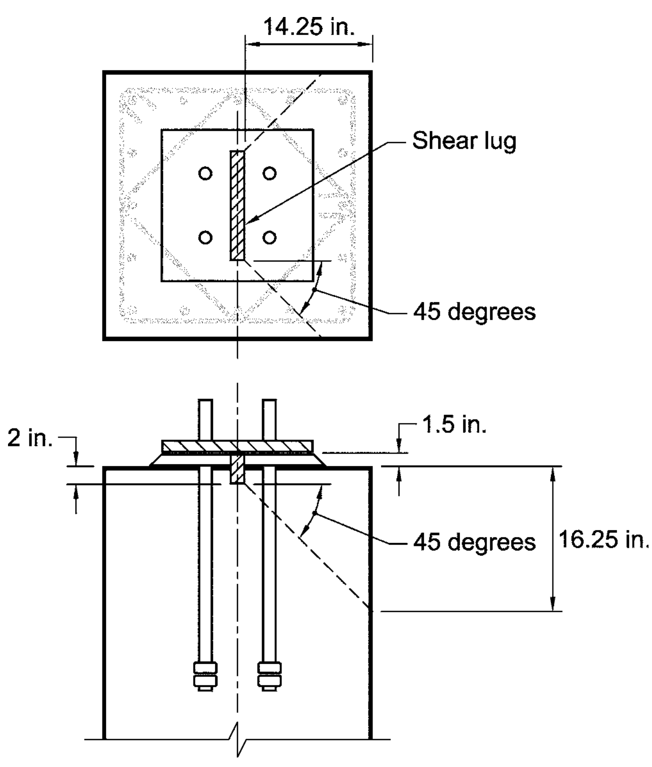
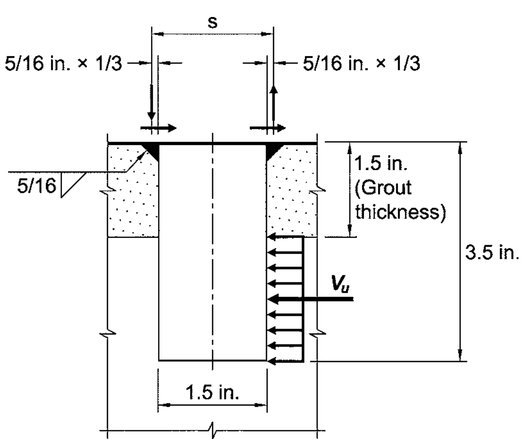


Fig. 4.35—Example 13: Shear lug.

Step	Calculations and discussion	ACI 349 Appendix D/ AISC, LRFD
1.	Check the lug for concrete bearing: *Note: The ϕ factors from ACI 349 are different than those used in ACI 318. ACI 349 ϕ factors will be used for this design.* $\phi P_n = \phi 1.3 f_c' A_{lug}$ (bearing capacity) $\phi P_n = (0.7)(1.3)(4000)(2 \times 12) = 87{,}360$ lb = 87.4 kips > V_u = 64 kips $\therefore$ – OK	ACI 349, D.4.6.2
2.	Check for shear acting toward the pier's edge: The effective stress area is found by projecting 45-degree planes from the bearing edges of the shear lug as shown in Fig. 4.35. $A_{eff} = (30 \times 16.25) - (12 \times 2) = 463.5$ in.^{2} *Note: The area of the lug must be deducted.* $\phi V_n = \phi 4 \sqrt{f_c'} A_{eff}$ $\phi = 0.8$ $\phi V_n = 0.8(4)\sqrt{4000}\,(436.5) = 93{,}803$ lb = 93.8 kips > V_u = 64 kips $\therefore$– OK	ACI 349, D.11.2

3.	Check the shear lug for bending and shear stresses: A uniform bearing pressure is assumed over the 2 in. of embedded depth into the top of the pedestal. The maximum moment will occur where the lug attaches to the base plate. The plate capacity is based on the load and resistance factor design (LRFD) capacity for a plate in bending and shear. Determine the bending capacity of shear lug: $M_u = (64)\left(1.5 + \frac{2}{2}\right) = 160$ in.-kip Applied factored moment at the weld of the lug to the base plate, based on the shear load applied at mid-depth of lug. $\phi M_n = \phi Z F_y$	
	Plastic moment capacity of the shear lug plate: $\phi M_n = (0.9)\frac{(12)(1.5)^2}{4}(36) = 218.7$ in.-kip $> M_u = 160$ in.-kip $\therefore$ – OK Determine the shear capacity of the shear lug: $\phi V_n = \phi 0.6 A_g F_y$	AISC-LRFD, F1.1
	Shear capacity of the shear lug plate: $\phi V_n = 0.9(0.6)(12 \times 1.5)(36) = 350$ kips $> V_u = 64$ kips $\therefore$ – OK  *Fig. 4.36—Example 13: Shear lug loads and welds.*	AISC-LRFD, J5.3

4.	Design the shear lug-to-base plate weld: Use E70XX electrodes referred to in AWS D1.1. The resultant forces on the welds are as shown in Fig. 4.36. The shear force is shared equally between the two fillets. The moment is resisted by a force couple at the two fillets. The vertical force couple with distance *s* is taken between the centroids of the two fillet welds. For 1-1/2 in. plate, the minimum allowable fillet size is 5/16 in. $f_v = (64)/(12 \times 2) = 2.67$ in.-kip (shear force per inch of weld) $f_t = \frac{160}{(12)\left[1.5 + \left(2 \times \frac{1}{3} \times \frac{5}{16}\right)\right]} = 7.8$ in.-kip (tension force per inch of weld) $R = \sqrt{f_v^2 + f_t^2} = 8.24$ in.-kip (total force per inch of weld) $\phi F_w = \phi 0.6 F_{EXX}(1.0 + 0.50\sin^{1.5}\theta)$ (weld capacity) $\theta = 90$ degrees (since the force is perpendicular to the axis of the weld) $\phi = 0.75$ Solve for the required weld leg size *a*: $a = \frac{8.24}{0.75(0.6)(70)(1 + 0.5\sin^{1.5}(90))(0.707)} = 0.25$ in. < 5/16 in. $\therefore$ Use the minimum allowable size of 5/16 in.	AISC-LRFD Appendix J2.4
5.	*Design summary:* a) Anchors: 1-1/4 in. diameter ASTM F1554 Grade 55 with double heavy hex nuts, 20 in. embedment depth (top of concrete to top of embedded head/nut) b) Supplementary reinforcement: 12 No. 6 bars with standard 180-degree hooks at the top and lap splice with No. 8 pier reinforcement. c) Shear lugs: 12 in. x 3-1/2 in. x 1-1/2 in. thick plate (ASTM A36/A36M) 5/16 in. fillet weld consistent with E70XX of the American Welding Society (AWS) D1.1 on each of shear lug side to base plate.	

CHAPTER 5—REFERENCES

5.1—Referenced standards and reports

The standards and reports listed below were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest version.

American Concrete Institute

318 Building Code Requirements for Structural Concrete and Commentary

349 Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary

355.2 Qualification of Post-Installed Mechanical Anchors in Concrete

American Institute of Steel Construction

AISC Steel Construction Manual: Steel Design Guide Series 1: Base Plate and Anchor Rod Design

AISC Steel Construction Manual: Steel Design Guide Series 1: Column Base Plates

American National Standards Institute

B1.1 Unified Inch Screw Threads (UN and UNR Thread Form)

B18.2.1 Square and Hex Bolts and Screws, Inch Series

B18.2.2 1987 Square and Hex Nuts

American Society of Civil Engineers

ASCE 7 Minimum Design Loads for Buildings and Other Structures

ASTM International

A29/A29M Specification for Steel Bars, Carbon and Alloy, Hot-Wrought, General Requirements for

A36/A36M Specification for Carbon Structural Steel

A108 Specification for Steel Bar, Carbon and Alloy, Cold-Finished

A193/A193M Specification for Alloy-Steel and Stainless Steel Bolting for High Temperature or High Pressure Service and Other Special Purpose Applications

A307 Specification for Carbon Steel Bolts and Studs, 60 000 PSI Tensile Strength

A449 Specification for Hex Cap Screws, Bolts and Studs, Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use

A992/A992M Specification for Structural Steel Shapes

F1554 Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength

American Welding Society (AWS)

D1.1 Structural Welding Code – Steel

These publications may be obtained from these organizations:

American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
www.concrete.org

American Institute of Steel Construction
One East Wacker Drive, Suite 700
Chicago, IL 60601-1802
www.aisc.org

American National Standards Institute
11 West 42nd Street
13th Floor
New York, NY 10036
www.ansi.org

American Society of Civil Engineers
1801 Alexander Bell Drive
Reston, VA 20191
(800) 548-2723
www.asce.org

ASTM International
100 Barr Harbor Drive
West Conshohocken, PA 19428
www.astm.org

American Welding Society
550 N.W. LeJeune Road
Miami, FL 33126
www.aws.org

5.2—Cited references

ACI Committee 318, 2002, "Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary," American Concrete Institute, Farmington Hills, MI, 443 pp.

ACI Committee 318, 2005, "Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary," American Concrete Institute, Farmington Hills, MI, 430 pp.

ACI Committee 355, 2004, "Qualification of Post-Installed Mechanical Anchors in Concrete (ACI 355.2-04) and Commentary," American Concrete Institute, Farmington Hills, MI, 31 pp.

Eligehausen, R., and Fuchs, W., 1988, "Loadbearing Behaviour of Anchor Fastenings under Shear, Combined Tension and Shear or Flexural Loading," *Betonwerk & Fertigteil-Technik (Wiesbaden)*, No. 2, pp. 48-56.

Eligehausen, R.; Fuchs, W.; and Mayber, B., 1987, "Load-bearing Behaviour of Anchor Fastenings in Tension," *Betonwerk & Fertigteil-Technik (Wiesbaden)*, No. 12, pp. 826-832.

Fuchs, W.; Eligehausen, R.; and Breen, J., 1995, "Concrete Capacity Design (CCD) Approach for Fastening to Concrete," *ACI Structural Journal*, V. 92, No. 1, Jan-Feb., pp. 73-93

International Code Council, Inc., 2003, "International Building Code," Falls Church, VA, 672 pp.

International Code Council, Inc., 2006, "International Building Code," Falls Church, VA, 666 pp.

Nelson Stud Welding Stud and Ferrule Catalog, 2004, Nelson Stud Welding, Elyria, OH.

Rehm, G.; Eligehausen, R.; and Malleé, R., 1992, "Befestigungstechnik (Fastening Technique)," *Betonkalender 1992*, V. II, Ernst & Sohn, Berlin, pp. 597-715. (in German)

APPENDIX A—TABLES

Table A.1—Materials for headed anchors and threaded rods

Material	Grade or type	Diameter, in.	Tensile strength, minimum ksi	Yield strength, minimum: ksi	Yield strength, minimum: Method	Elongation, minimum: %	Elongation, minimum: Length	Reduction of area, minimum %	Comments
Welded studs AWS D1.1:2006 ASTM A29/A29M-05/ A108-03	B 1010 1020	1/4 to 1	65	51	0.2%	20	2 in.	50	Structural Welding Code – Steel, Section 7, covers welded headed or welded bent studs. AWS D1.1 requires studs to be made from cold drawn bar stock conforming to the requirements of ASTM A108.
ASTM F1554-04 (H, HD, T)*	36	1/4 to 4	58	36	0.2%	23	2 in.	40	ASTM F1554, "Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength," is the preferred material specification for anchors.
	55	≤2†	75	55	0.2%	21	2 in.	30	
	105	1/4 to 3	125	105	0.2%	15	2 in.	45	
ASTM A193/ A193M-06a (H, T)	B7	≤2-1/2	125	105	0.2%	16	4D	50	ASTM A193/A193M, "Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for High-Temperature Service or High Pressure Service and Other Special Purpose Applications": Grade B7 is an alloy steel for use in high temperature service.
		2-1/2 to 4	115	95	0.2%	16	4D	50	
		Over 4 to 7	100	75	0.2%	18	4D	50	
ASTM A307-04 (Grade A: HD) (Grade C: H, T)	A	1/4 to 4	60	—	—	18	2 in.	—	ASTM A307, "Standard Specification for Carbon Steel Bolts and Studs, 60 000 PSI Tensile Strength": ACI 318 specifies that steel elements meeting the requirements of ASTM A307 shall be considered ductile. Note that Grade C conforms to tensile properties for ASTM A36/A36M.
	C	1/4 to 4	58	36	—	23	2 in.	—	
ASTM A36/A36M-05 (H, T)	—	To 8	58	36	—	23	2 in.	—	ASTM A36/A36M, "Standard Specification for Carbon Structural Steel": Since ACI 318 considers ASTM A307 to be ductile, ASTM A36/A36M will qualify since it is the basis for ASTM A307 Grade C.
ASTM A449-04b (H, HD, T)	1	1/4 to 1	120	92	0.2%	14	4D	35	ASTM A449, "Standard Specification for Hex Cap Screws, Bolts and Studs, Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use": This specification is for general high-strength applications.
		Over 1 to 1-1/2	105	81	0.2%	14	4D	35	
		Over 1-1/2 to 3	90	58	0.2%	14	4D	35	

*Anchor type availabilities are denoted as follows: H = hooked bolt, HD = headed bolt, and T = threaded rod.
†Diameters larger than 2 in. (up to 4 in.) are available, but the reduction of area will vary for Grade 55.

Table A.2(a)—Bearing area (A_{brg}) for cast-in anchors, threaded rods with nuts, and threaded rods with nuts and washers*

Nominal diameter of stud or bolt, d_o, in.	Number of threads per in., UNC, n	Effective cross-sectional area of bolt A_{se}, in.‡	Bearing area A_{brg}, in.†							
			Square head bolt	Hex head bolt	Heavy hex head bolt	Threaded rod with square nut	Threaded rod with heavy square nut	Threaded rod with hex nut	Threaded rod with heavy hex nut	Welded headed stud
0.250	20	0.032	0.092	0.117	—	0.142	0.201	0.117	0.167	0.147
0.375	16	0.078	0.206	0.164	—	0.280	0.362	0.164	0.299	0.331
0.500	13	0.142	0.366	0.291	0.467	0.464	0.569	0.291	0.467	0.589
0.625	11	0.226	0.572	0.454	0.671	0.693	0.822	0.454	0.671	0.920
0.750	10	0.334	0.824	0.654	0.911	0.824	1.121	0.654	0.911	0.785
0.875	9	0.462	1.121	0.891	1.188	1.121	1.465	0.891	1.188	0.884
1.000	8	0.606	1.465	1.163	1.501	1.465	1.855	1.163	1.501	—
1.125	7	0.763	1.854	1.472	1.851	1.854	2.291	1.472	1.851	—
1.250	7	0.969	2.288	1.817	2.237	2.288	2.773	1.817	2.237	—
1.375	6	1.16	2.769	2.199	2.659	2.769	3.300	2.199	2.659	—
1.500	6	1.41	3.295	2.617	3.118	3.295	3.873	2.617	3.118	—
1.750	5	1.90	—	3.562	4.144	—	—	—	4.144	—
2.000	4-1/2	2.50	—	4.653	5.316	—	—	—	5.316	—

*All washers need to meet the minimum thickness requirements of ACI 318-05, D.5.2.8 or the bolt/nut bearing area may conservatively be used to calculate A_{brg}.

†$A_{brg} = A_H - A_D$ (refer to Tables A.2(b) and (c) for A_H and A_D values).

‡$A_{se,N} = A_{se,V} = \pi/4(d_o - (0.9743/n))^2$, where n is the number of threads.

Notes:

1. Dimensions and data taken from ANSI 18.2.1 and 18.2.2.
2. $A_H = F^2$ (square head bolt/nut) or $A_H = 1.5F^2\tan 30$ (hex head bolt/nut, hex or heavy hex).

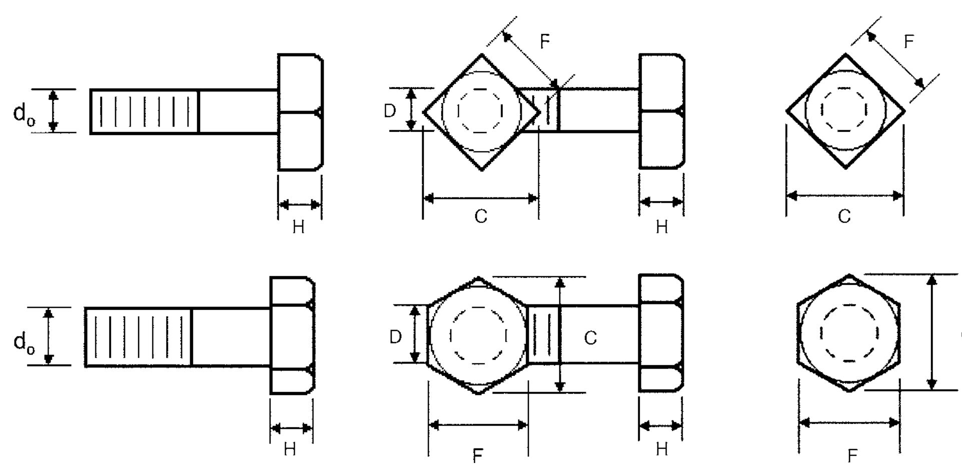


Fig. A.1—Bolt dimensions.

Table A.2(b)—Dimensional properties of bolts and studs for determining bearing area (A_{brg})

Bolt or stud diameter d_o, in.	Gross area of bolt or stud, in.^{2}	Square head bolt		Hex head bolt		Heavy hex head bolt		Welded headed stud	
		Width F, in.	Gross area of head A_H, in.^{2}	Width F, in.	Gross area of head A_H, in.^{2}	Width F, in.	Gross area of head A_H, in.^{2}	Head diameter d_H, in.	Gross area of head A_H, in.^{2}
0.250	0.049	0.375	0.141	0.438	0.166	NA	—	0.500	0.196
0.375	0.110	0.563	0.316	0.563	0.274	NA	—	0.750	0.442
0.500	0.196	0.750	0.563	0.750	0.487	0.875	0.663	1.000	0.785
0.625	0.307	0.938	0.879	0.938	0.761	1.063	0.978	1.250	1.227
0.750	0.442	1.125	1.266	1.125	1.096	1.250	1.353	1.250	1.227
0.875	0.601	1.313	1.723	1.313	1.492	1.438	1.790	1.375	1.485
1.000	0.785	1.500	2.250	1.500	1.949	1.625	2.287	—	—
1.125	0.994	1.688	2.848	1.688	2.466	1.813	2.845	—	—
1.250	1.227	1.875	3.516	1.875	3.045	2.000	3.464	—	—
1.375	1.485	2.063	4.254	2.063	3.684	2.188	4.144	—	—
1.500	1.767	2.250	5.063	2.250	4.384	2.375	4.885	—	—
1.750	2.405	NA	NA	2.625	5.967	2.750	6.549	—	—
2.000	3.142	NA	NA	3.000	7.794	3.125	8.457	—	—

Notes:
1. Dimensions and data taken from ANSI 18.2.1 and *Nelson Stud Welding Catalog* (2004).
2. $A_H = F^2$ (square head bolt/nut) or $A_H = 1.5F^2\tan 30$ (hex head bolt/nut, hex or heavy hex), or $A_H = \pi(dh)2/4$.
3. Refer to Fig. A.1, Table A.2(a).

Table A.2(c)—Dimensional properties of nuts for determining bearing area (A_{brg})

Bolt diameter d_o, in.	Gross area of bolt A_D, in.^{2}	Square nut		Heavy square nut		Hex nut		Heavy hex nut	
		Width F, in.	Gross area A_H, in.^{2}	Width F, in.	Gross area A_H, in.^{2}	Width F, in.	Gross area A_H, in.^{2}	Width F, in.	Gross area A_H, in.^{2}
0.250	0.049	0.438	0.191	0.500	0.250	0.438	0.166	0.500	0.217
0.375	0.110	0.625	0.391	0.688	0.473	0.563	0.274	0.688	0.409
0.500	0.196	0.813	0.660	0.875	0.766	0.750	0.487	0.875	0.663
0.625	0.307	1.000	1.000	1.063	1.129	0.938	0.761	1.063	0.978
0.750	0.442	1.125	1.266	1.250	1.563	1.125	1.096	1.250	1.353
0.875	0.601	1.313	1.723	1.438	2.066	1.313	1.492	1.438	1.790
1.000	0.785	1.500	2.250	1.625	2.641	1.500	1.949	1.625	2.287
1.125	0.994	1.688	2.848	1.813	3.285	1.688	2.466	1.813	2.845
1.250	1.227	1.875	3.516	2.000	4.000	1.875	3.045	2.000	3.464
1.375	1.485	2.063	4.254	2.188	4.785	2.063	3.684	2.188	4.144
1.500	1.767	2.250	5.063	2.375	5.641	2.250	4.384	2.375	4.885
1.750	2.405	—	—	—	—	—	—	2.750	6.549
2.000	3.142	—	—	—	—	—	—	3.125	8.457

Notes:
1. Dimensions taken from ANSI 18.2.2 and Table 7-20, *AISC Steel Construction Manual* (2004).
2. $A_H = F^2$ (square head bolt/nut) or $A_H = 1.5F^2\tan 30$ (hex head bolt/nut, hex/heavy hex).
3. Refer to Fig. A.1, Table A.2(a).

Table A.3—Sample data for a post-installed, torque-controlled mechanical expansion anchor

Characteristic	Symbol	Units	Nominal anchor diameter							
Installation information										
Outside diameter	d_o	in.	3/8		1/2		5/8		3/4	
Effective embedment depth	h_{ef}	in.	1.75		2.5		3		3.5	
			2.75		3.5		4.5		5	
			4.5		5.5		6.5		8	
Installation torque	T_{inst}	ft-lb	30		65		100		175	
Minimum edge distance	c_{min}	in.	1.75		2.5		3		3.5	
Minimum spacing	s_{min}	in.	1.75		2.5		3		3.5	
Minimum concrete thickness	h_{min}	in.	$1.5h_{ef}$		$1.5h_{ef}$		$1.5h_{ef}$		$1.5h_{ef}$	
Critical edge distance at h_{min}	c_{ac}	in.	2.1		3.0		3.6		4.0	
Anchor data										
Anchor material	ASTM F1554 Grade 55 (meets ductile steel element requirements)									
Category number	1, 2, or 3	—	2		2		1		1	
Yield strength of anchor steel	f_{ya}	psi	55,000		55,000		55,000		55,000	
Ultimate strength of anchor steel	f_{uta}	psi	75,000		75,000		75,000		75,000	
Effective tensile stress area	A_{se}	in.^{2}	0.0775		0.142		0.226		0.334	
Effective shear stress area	A_{se}	in.^{2}	0.0775		0.142		0.226		0.334	
Effectiveness factor for uncracked concrete	k_{uncr}	—	24		24		24		24	
Effectiveness factor for cracked concrete used for ACI 318 design	k_{cr}*	—	17		17		17		17	
$\psi_{c,N}$ for ACI 318 design in cracked concrete	$\psi_{c,N}$*	—	1.0		1.0		1.0		1.0	
$\psi_{c,N} = k_{uncr}/k_{cr}$ for ACI 318 design in uncracked concrete	$\psi_{c,N}$*	—	1.4		1.4		1.4		1.4	
Pullout or pull-through resistance from tests	N_p†	lb	h_{ef}	N_p	h_{ef}	N_p	h_{ef}	N_p	h_{ef}	N_p
			1.75	1354	2.5	2312	3	4469	3.5	5632
			2.75	2667	3.5	3830	4.5	8211	5	9617
			4.5	5583	5.5	7544	6.5	14,254	8	19,463
Tension resistance of single anchor for seismic loads	N_{eq}	lb	1.75	903	2.5	1541	3	2979	3.5	3755
			4.5	3722	5.5	5029	6.5	9503	8	12,975
Shear resistance of single anchor for seismic loads	V_{eq}	lb	2906		5321		8475		12,543	
Axial stiffness in service load range	β	lb/in.	55,000		57,600		59,200		62,000	
Coefficient of variation for axial stiffness in service load range	v	%	12		11		10		9	

*These are values used for k_c and $\psi_{c,N}$ in ACI 318 for anchors qualified for use only in both cracked and uncracked concrete.
†N_p is NA (not applicable). Pullout or pull-through type failures were not observed for any tests.

Note: This table was created for illustrating the use of test data as would be developed from a test program according to ACI 355.2 for use with the design procedures of ACI 318-05 Appendix D. These data are purely fictional and do not represent any specific anchor system. This data should not be used for actual design purposes. For actual anchor design data, obtain data that have been tested, developed, and certified to be in accordance with ACI 355.2.

Anchor system is qualified for use in both cracked and uncracked concrete in accordance with test program of Table 4.2 of ACI 355.2-04. The material, ASTM F1554 Grade 55, meets the ductile steel element requirements of ACI 318-05 Appendix D (tensile test elongation of at least 14% and reduction in area of at least 30%).

Table A.4—Sample data for a post-installed, undercut anchor

Characteristic	Symbol	Units	Anchor nominal diameter							
Installation information										
Stud diameter		in.	1/4		1/2		3/4		1-1/4	
Sleeve diameter	d_o	in.	0.450		0.900		1.280		2.20	
Effective embedment depth	h_{ef}	in.	1.50		3.0		6.0		12.0	
			2.00		5.0		9.0		18.0	
			2.50		7.0		12.0		24.0	
Minimum edge distance	c_{min}	in.	1.50		2.5		4.0		7.0	
Minimum spacing	s_{min}	in.	1.75		2.5		5.0		10.0	
Minimum concrete thickness	h_{min}	in.	$1.5h_{ef}$		$1.5h_{ef}$		$1.5h_{ef}$		$1.5h_{ef}$	
Critical edge distance at h_{min}	c_{ac}	in.	2.0		4.0		6.0		10.0	
Anchor data										
Category number	1, 2, or 3	—	1		1		1		1	
Yield strength of anchor steel	f_{ya}	psi	36,000		36,000		36,000		36,000	
Ultimate strength of anchor steel	f_{ut}	psi	58,000		58,000		58,000		58,000	
Effective tensile stress area	A_{se}	in.^{2}	0.032		0.142		0.334		1.000	
Effective shear stress area	A_{se}	in.^{2}	0.032		0.142		0.334		1.000	
Effective factor for uncracked concrete	k_{uncr}	—	24		24		24		24	
Effective factor for cracked concrete used for ACI 318 design	k_{cr}*	—	24		24		24		24	
$\psi_{c,N}$ for ACI 318 design in cracked concrete	$\psi_{c,N}$*	—	1.0		1.0		1.0		1.0	
$\psi_{c,N} = k_{uncr}/k_{cr}$ for ACI 318 design in uncracked concrete	$\psi_{c,N}$*	—	1.0		1.0		1.0		1.0	
Pullout or pull-through resistance from tests	N_p†	lb	h_{ef}	N_p	h_{ef}	N_p	h_{ef}	N_p	h_{ef}	N_p
			1.50	NA	3.0	NA	6.0	NA	12.0	NA
			2.00	NA	5.0	NA	9.0	NA	18.0	NA
			2.50	NA	7.0	NA	12.0	NA	24.0	NA
Tension resistance of single anchor for seismic loads	N_{eq}	lb	1.50	NA	3.0	NA	6.0	NA	12.0	NA
			2.50	NA	7.0	NA	12.0	NA	24.0	NA
Shear resistance of single anchor for seismic loads	V_{eq}	lb	2906		5321		12,000		20,000	
Axial stiffness in service load range	β	lb/in.	55,000		57,600		59,200		62,000	
Coefficient of variation for axial stiffness in service load range	v	%	12		11		10		9	

*These are values used for $-k_c$ and $\psi_{c,N}$ in ACI 318 for anchors qualified for use only in both cracked and uncracked concrete.
†N_p is NA (not applicable). Pullout or pull-through type failures were not observed for any tests.

Note: This table was created for illustrating the use of test data as would be developed from a test program according to ACI 355.2 for use with the design procedures of ACI 318-08 Appendix D. These data are purely fictional and do not represent any specific anchor system. These data should not be used for actual design purposes. For actual anchor design data, obtain data that have been tested, developed, and certified to be in accordance with ACI 355.2.

Anchor system is qualified for use in both cracked and uncracked concrete in accordance with test program of Table 4.2 of ACI 355.2-04. The material, ASTM F1554 Grade 36, meets the ductile steel element requirements of ACI 318-08 Appendix D (tensile test elongation of at least 14% and reduction in area of at least 30%).

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American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
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Phone: 248-848-3700
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Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D

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