

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

Reported by ACI Committee 355



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

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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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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.

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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

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	

soon, are under development for adhesive anchors and concrete screw anchors.

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 ϕ 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****CODE****COMMENTARY****D.1 — Definitions****RD.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.

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.

CODE

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.

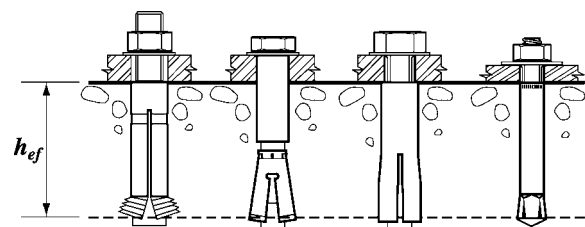
Post-installed anchor — An anchor installed in hardened concrete. Expansion anchors and undercut anchors are examples of post-installed anchors.

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.

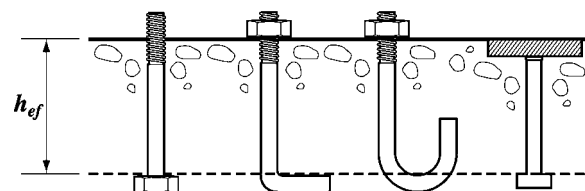
COMMENTARY

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.



CODE

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 $1.4N_p$ (where 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

CODE

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.

COMMENTARY

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.

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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 not 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,

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

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

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} , ϕN_{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

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.

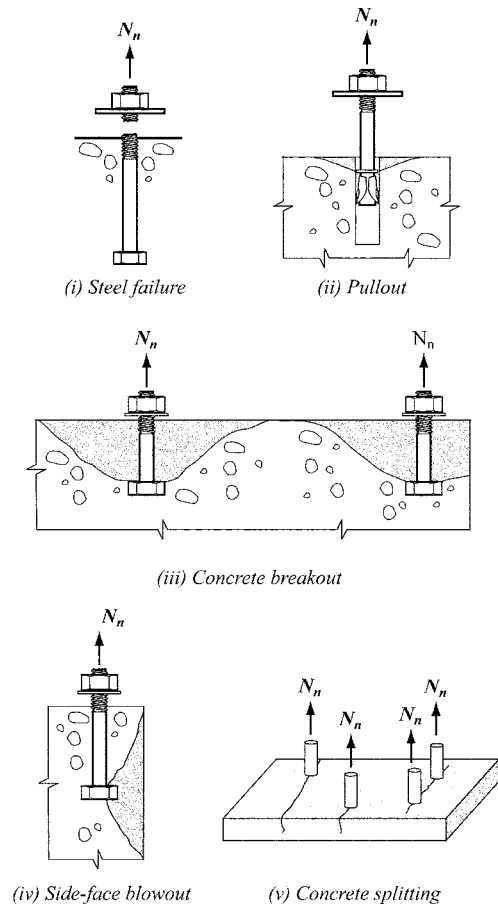
Test procedures can also be used to determine the single-anchor breakout strength in tension and in shear. The test results, however, are required to be evaluated on a basis statistically equivalent to that used to select the values for the concrete breakout method “considered to satisfy” provisions 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” (Fig. RD.4.2) as long as sufficient data are available to verify the model.

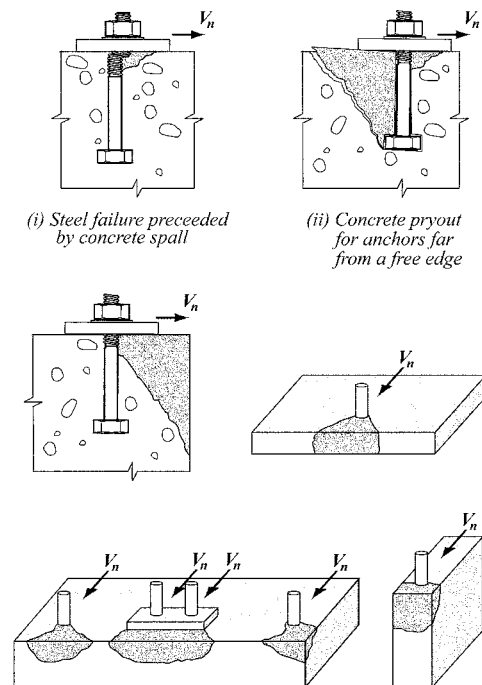
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the tests shall be compatible with the materials used in the structure. The nominal strength shall be based on the 5 percent fractile of the basic individual anchor strength. For nominal strengths related to concrete strength, modifications for size 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.

COMMENTARY



(a) tensile loading



(b) shear loading

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.

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

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

COMMENTARY

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)

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)].

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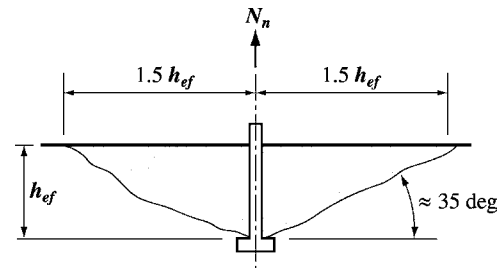


Fig. RD.4.2.2(a)—Breakout cone for tension

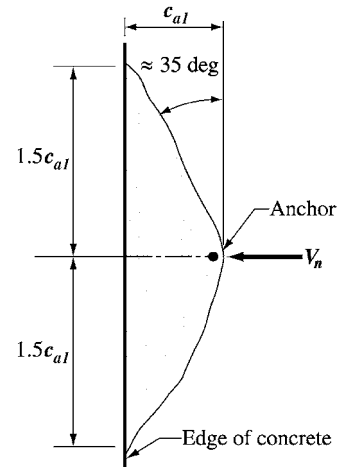


Fig. RD.4.2.2(b)—Breakout cone for shear

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

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			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)	0.75	0.65	Category 1 — low sensitivity to installation and high reliability;
Category 2 (Medium sensitivity to installation and medium reliability)	0.65	0.55	Category 2 — medium sensitivity to installation and medium reliability; and
Category 3 (High sensitivity to installation and lower reliability)	0.55	0.45	Category 3 — high sensitivity to installation and lower reliability.
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.			The capacities of anchors under shear loads are not as sensitive to installation errors and tolerances. Therefore, for shear calculations of all anchors, $\phi = 0.75$ for Condition A and $\phi = 0.70$ for Condition B.
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:			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.
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	Even though the ϕ factor for plain concrete in Appendix C uses a value of 0.65, the basic factor for brittle concrete failures ($\phi = 0.75$) was chosen based on results of probabilistic studies ^{D.16} that indicated the use of $\phi = 0.65$ with mean values of concrete-controlled failures produced adequate safety levels. Because the nominal resistance expressions used in this appendix and in the test requirements are based on the 5 percent fractiles, the $\phi = 0.65$ value would be overly conservative. Comparison with other design procedures and probabilistic studies ^{D.16} indicated that the choice of $\phi = 0.75$ 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:

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

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 1.35 (from $1.4D + 1.7L$) divided

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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:

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

(b) for a group of anchors:

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

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 $1.5h_{ef}$:

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

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.

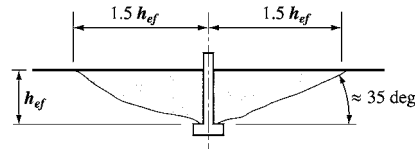
RD.5.2 — Concrete breakout strength of anchor in tension

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).

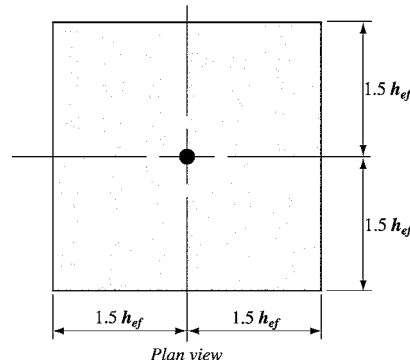
Figure RD.5.2.1(a) shows A_{Nco} and the development of Eq. (D-6). A_{Nco} is the maximum projected area for a single anchor. Figure RD.5.2.1(b) shows examples of the projected areas for various single-anchor and multiple-anchor arrangements. Because A_{Nc} is the total projected area for a group of 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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The critical edge distance for headed studs, headed bolts, expansion anchors, and undercut anchors is $1.5h_{ef}$



Section through failure cone

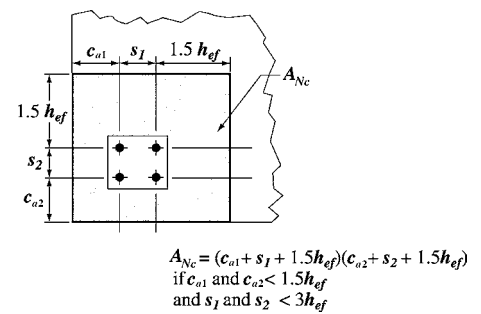
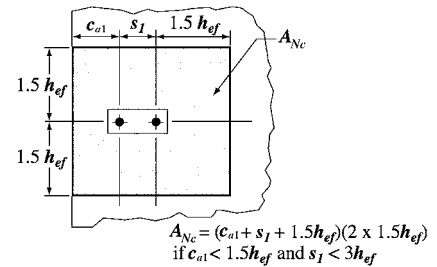
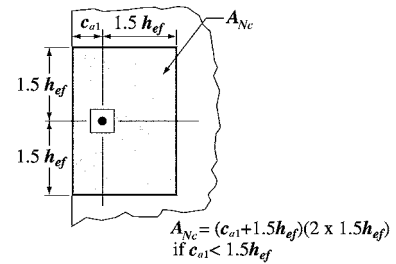


Plan view

$$A_{Nc0} = [2(1.5)h_{ef}] [2(1.5)h_{ef}] = 9h_{ef}^2$$

(a)

COMMENTARY



(b)

Fig. RD.5.2.1(a) — Calculation of A_{Nc0} ; and (b) projected areas for single anchors and groups of anchors and calculation of 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

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

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. $\leq h_{ef} \leq 25$ in., N_b shall not exceed

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

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. $\leq h_{ef} \leq 25$ in. The limit of 25 in. corresponds to the upper range of test data. This expression can also be applied to some

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D.5.2.3 — Where anchors are located less than $1.5h_{ef}$ from three or more edges, the value of h_{ef} used in Eq. (D-4) through (D-11) shall be the greater of $c_{a,max}/1.5$ and 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 $1.5h_{ef}$ from three or more edges, the tensile breakout strength computed by the CCD Method, which is the basis for Eq. (D-4) to (D-11), gives overly conservative results.^{D.20} This occurs because the ordinary definitions of A_{Nc}/A_{Nco} do not correctly reflect the edge effects. This problem is corrected by limiting the value of h_{ef} used in Eq. (D-4) through (D-11) to $c_{a,max}/1.5$, where $c_{a,max}$ is the largest of the influencing edge distances that are less than or equal to the actual $1.5h_{ef}$. In no case should $c_{a,max}$ be taken less than one-third of the maximum spacing between anchors within the group. The limit on 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 h_{ef} to be used in the computations where $h_{ef} = c_{a,max}/1.5$, results in $h_{ef} = h'_{ef} = 4$ in. For this example, this would be the proper value to be used for h_{ef} in computing the resistance even if the actual embedment depth is larger.

The requirement of D.5.2.3 may be visualized by moving the actual concrete breakout surface, which originates at the actual h_{ef} , toward the surface of the concrete parallel to the applied tension load. The value of 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 h_{ef} with the concrete surface.

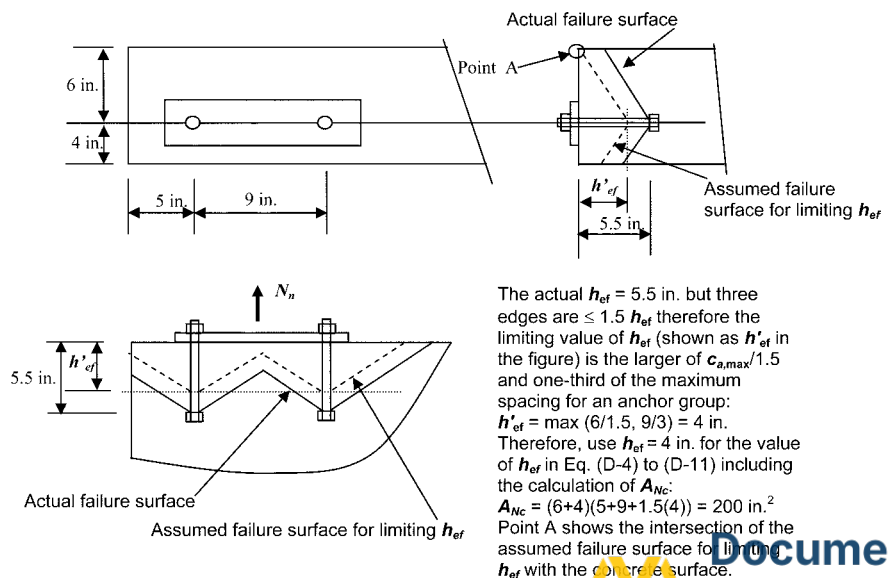
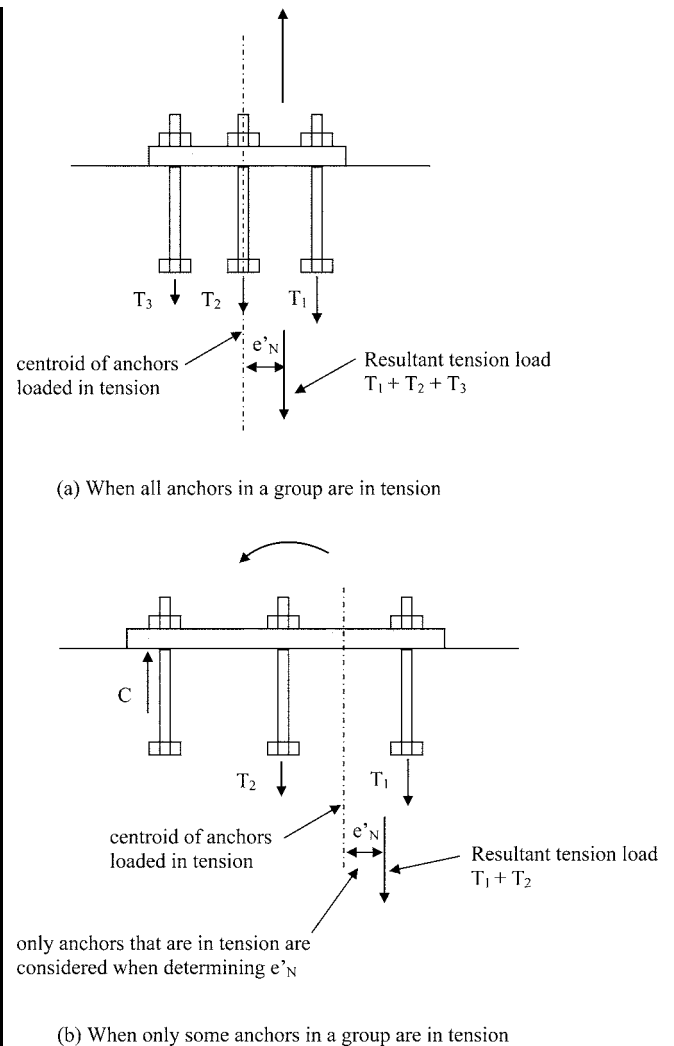


Fig. RD.5.2.3 — Tension in narrow members.

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COMMENTARY

Fig. RD.5.2.4 — Definition of e'_N for group anchors.

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

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

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

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

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:

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

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

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:

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

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

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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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

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

$$N_{pn} = \psi_{c,p} N_p \quad (D-14)$$

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.

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.

splitting mode of failure, the basic concrete breakout strength is reduced by a factor $\psi_{cp,N}$ if $c_{a,min}$ is less than the critical edge distance c_{ac} . If supplementary reinforcement to control splitting is present or if the anchors are located in a region where analysis indicates cracking of the concrete at service loads, then the reduction factor $\psi_{cp,N}$ is taken as 1.0. The presence of supplementary reinforcement to control splitting does not affect the selection of Condition A or B in D.4.4 or D.4.5.

RD.5.3 — Pullout strength of anchor in tension

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

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:

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

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:

$$N_p = 0.9f'_c e_h d_o \quad (D-16)$$

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:

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

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:

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

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

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

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

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

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:

$$V_{cb} = \frac{A_{vc}}{A_{vco}} \psi_{ed,v} \psi_{c,v} V_b \quad (D-21)$$

COMMENTARY

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} 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:

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

(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 $1.5c_{a1}$:

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

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

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

The critical edge distance for headed studs, headed bolts, expansion anchors, and undercut anchors is $1.5c_{a1}$

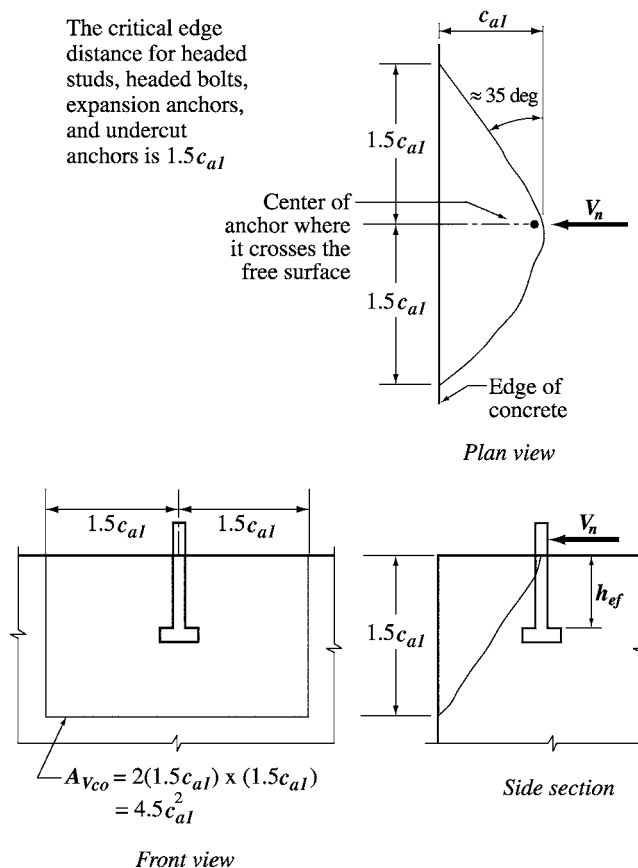


Fig. RD.6.2.1(a)—Calculation of A_{Vco}

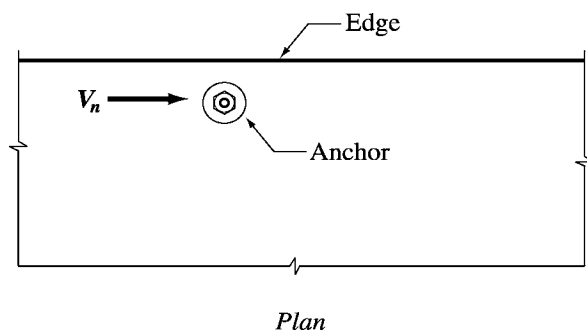


Fig. RD.6.2.1(c) — Shear force parallel to an edge

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.

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

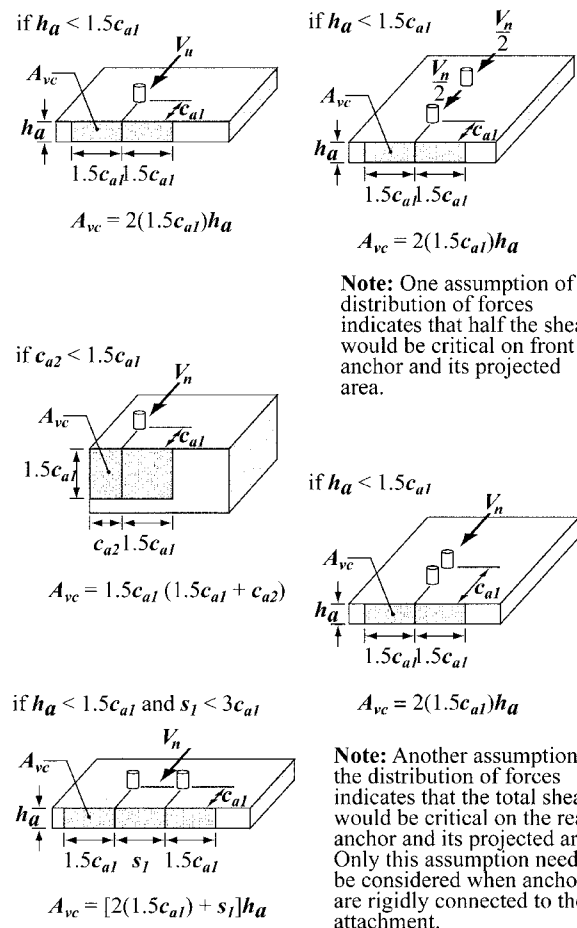


Fig. RD.6.2.1(b) — Projected area for single anchors and groups of anchors and calculation of A_{Vc}

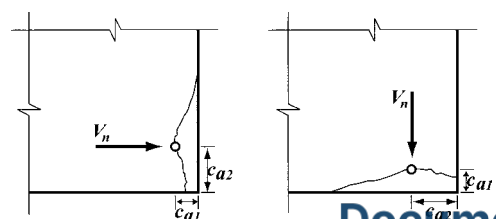


Fig. RD.6.2.1(d) — Shear force near a corner.

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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

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

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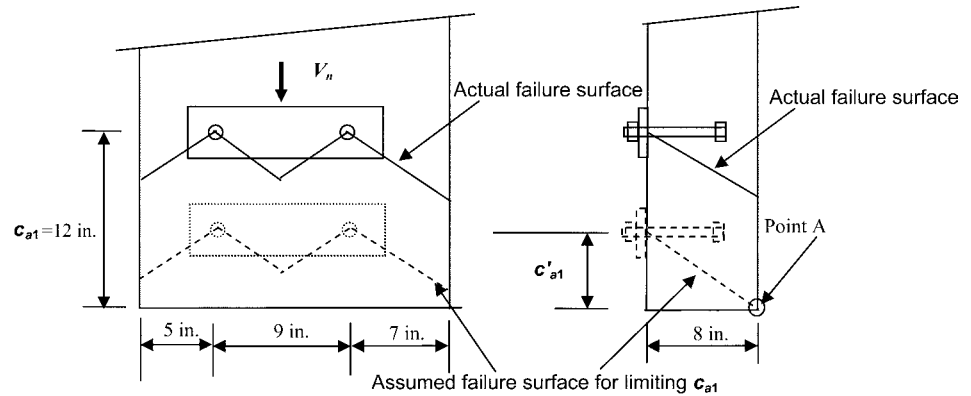
The constant, 7, in the shear strength equation was determined from test data reported in Reference D.9 at the 5 percent fractile adjusted for cracking.

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) through (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 $\leq 1.5 c_{a1}$ 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} :

$$A_{vc} = (5 + 9 + 7)(1.5(5.33)) = 168 \text{ in.}^2$$

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

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

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.

CODE

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

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

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

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

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

COMMENTARY

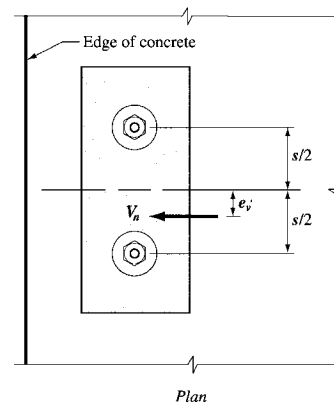


Fig. RD.6.2.5 — Definition of dimensions e'_v .

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:

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

(b) for a group of anchors:

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

where:

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.

CODE

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

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

D.7 — Interaction of tensile and shear forces

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

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

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

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

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

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.

COMMENTARY

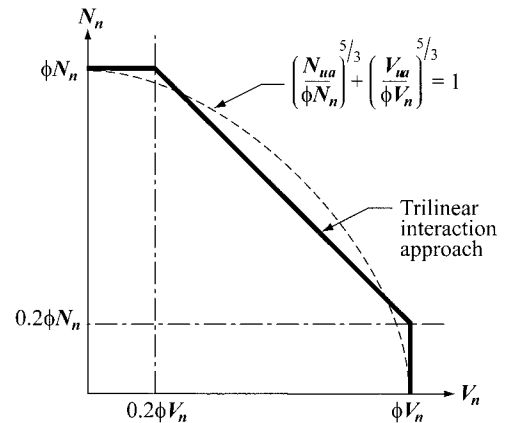


Fig. RD.7 — Shear and tensile load interaction equation.

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

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

The minimum spacings, edge distances, and thicknesses are 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.

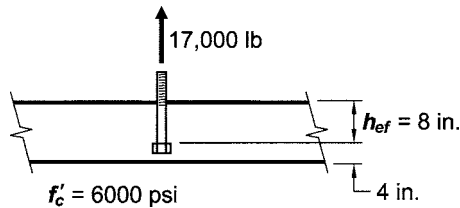


Fig. 4.1—Example 1: Single headed anchor in tension away from edges. (Note: Foundation reinforcement not shown for clarity.)

Step	Calculations and discussion	ACI 318-05 Section
1.	The factored design load is: $N_{ua} = 17,000 \text{ lb}$	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. 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 $\phi N_{sa} < \text{capacity of concrete breakout, pullout, side-face blowout, or splitting failure in tension to satisfy ductility requirements}$ where $\phi = 0.75$ and the steel bolt strength is: $N_{sa} = nA_{se}f_{uta}$ 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. $1.9f_{ya} = 1.9(36,000) = 68,400 \text{ psi} > 58,000 \text{ psi} - \text{OK}$ $n = 1 \text{ (single anchor)}$ ANSI/ASME B1.1 defines A_{se}: $A_{se} = \frac{\pi}{4} \left(d_o - \frac{0.9743}{n_t} \right)^2$ Substituting 1 in. for d_o and 8 (number of threads) for n_t, $A_{se} = 0.606 \text{ in.}^2$ $N_{sa} = (1)(0.606 \text{ in.}^2)(58,000 \text{ psi}) = 35,148 \text{ lb}$ $\phi N_{sa} = (0.75)35,148 \text{ lb} = 26,361 \text{ lb}$ Reduction for regions of moderate or high seismic risk $0.75(\phi N_{sa}) = 0.75(26,361 \text{ lb}) = 19,771 \text{ lb}$ $19,771 \text{ lb} > 17,000 \text{ lb (requirement of } 0.75\phi N_{sa} > N_{ua} \text{ is met)}$	D.3.3.4 D.3.3.5 D.4.4(a) Eq. (D-3) D.5.1.2 RD.5.1.2 D.3.3.3
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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: $0.75\phi N_{cb} \geq N_{ua} \text{ and } \phi N_{cb} > \phi N_{sa} \text{ (to ensure that the steel failure mode governs)}$ where $\phi = 0.70 \text{ (for concrete breakout)}$ $\phi = 0.75 \text{ (for ductile steel in tension)}$ Determine the concrete breakout strength assuming Condition B (no supplementary reinforcement has been provided): $N_{cb} = \frac{A_{Nc}}{A_{Nco}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$ where $A_{Nc}/A_{Nco} = 1.0 \text{ for single anchor remote from edges (a complete breakout prism is developed and no reduction is required)}$ $\psi_{ed,N} = 1.0 \text{ since anchor is remote from edges } (c_{a,min} \geq 1.5h_{ef})$ $\psi_{c,N} = 1.0 \text{ since for this example assumed cracking occurs in the concrete at service load levels}$ $\psi_{cp,N} = 1.0 \text{ (cast-in anchors)}$ The basic concrete breakout strength of a single anchor in tension is: $N_b = k_c \sqrt{f'_c} h_{ef}^{1.5}$ 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 $\phi N_{cb} = 0.7(42,065 \text{ lb}) = 29,445 \text{ lb}$ Reduction for regions of moderate or high seismic risk: $0.75\phi N_{cb} = 0.75(29,445 \text{ lb}) = 22,084 \text{ lb}$ $22,084 \text{ lb} \geq N_{ua} \text{ and } 22,084 \text{ lb} > 0.75\phi N_{sa} \text{ (or } 19,771 \text{ lb)}$ Therefore, the anchor strength remains governed by ductile steel failure.	D.4.4 Eq. (D-4) RD.5.2.1 D.5.2.5 D.5.2.6 D.5.2.7 Eq. (D-7) D.3.3.3
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