CHAPTER 17—ANCHORING TO CONCRETE

CODE

17.1—Scope

17.1.1 This chapter shall apply to the design of anchors in concrete used to transmit 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.

17.1.2 Provisions of this chapter shall apply to the following anchor types (a) through (g):

(a) Headed studs and headed bolts having a geometry that has been demonstrated to result in a pullout strength in uncracked concrete equal to or exceeding $1.4N_p$, where N_p is given in Eq. (17.6.3.2.2a).
(b) Hooked bolts having a geometry that has been demonstrated to result in a pullout strength without the benefit of friction in uncracked concrete equal to or exceeding $1.4N_p$, where N_p is given in Eq. (17.6.3.2.2b)
(c) Post-installed expansion (torque-controlled and displacement-controlled) anchors that meet the assessment criteria of ACI CODE-355.2.
(d) Post-installed undercut anchors that meet the assessment criteria of ACI CODE-355.2.
(e) Post-installed adhesive anchors that meet the assessment criteria of ACI CODE-355.4.
(f) Post-installed screw anchors that meet the assessment criteria of ACI CODE-355.2.
(g) Attachments with shear lugs.

17.1.3 If post-installed deformed reinforcing bars qualified in accordance with ACI CODE-355.5 are used as tension or compression reinforcement, development length shall be in accordance with 25.4.2 and 25.4.9.

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R17.1—Scope

R17.1.1 This chapter is restricted in scope to anchoring devices used to transmit loads into concrete and whose use is related to strength, stability, or life safety. Two main types of applications are envisioned. The first is connections between structural elements where the failure of an anchor or 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 factored load combinations and ϕ-factors are appropriate for structural applications. Other standards may require more stringent safety levels during temporary handling.

R17.1.2 Typical cast-in headed studs and headed bolts with head geometries consistent with ASME B1.1, B18.2.1, and B18.2.6 have been tested and proven to behave predictably; therefore, calculated pullout strengths are acceptable.

Post-installed expansion, screw, and undercut anchors do not have predictable pullout strengths, and therefore qualification tests to establish the pullout strengths according to ACI CODE-355.2 are required. For post-installed expansion, screw, and undercut anchors to be used in conjunction with the requirements of this chapter, the results of the ACI CODE-355.2 tests have to indicate that pullout failures exhibit acceptable load-displacement characteristics or that pullout failures are precluded by another failure mode.

In accordance with ACI CODE-355.4, steel elements for adhesive anchors may include threaded rods, deformed reinforcing bars, or internally threaded steel sleeves with external deformations. For adhesive anchors, the characteristic bond stress and suitability for structural applications are established by testing in accordance with ACI CODE-355.4. Adhesive anchors are particularly sensitive to a number of factors including installation direction and load type. If adhesive anchors are used to resist sustained tension, the provisions include testing requirements for horizontal or upwardly inclined installations in 17.2.3, design requirements in 17.5.2.2, certification requirements in 26.7, and inspection requirements in 26.13. Adhesive anchors qualified in accordance with ACI CODE-355.4 are tested in concrete with compressive strengths within two ranges: 2500 to 4000 psi and 6500 to 8500 psi. Bond strength is, in general, not highly sensitive to concrete compressive strength.

R17.1.3 Post-installed deformed reinforcing bar systems tested and assessed in accordance with ACI CODE-355.5 are suitable for use in accordance with the provisions for straight deformed reinforcing bars in the Code. Refer to R25.4.1.6. Post-installed reinforcing bar dowels are designed using the provisions for adhesive anchors in Chapter 17. In this case, qualification of the adhesive is performed in accordance with ACI CODE-355.4.

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17.1.4 The removal and resetting of post-installed mechanical anchors is prohibited.

17.1.5 This chapter does not apply for load applications that are predominantly high-cycle fatigue or due to impact.

17.1.6 This chapter does not apply to specialty inserts, through-bolts, multiple anchors connected to a single steel plate at the embedded end of the anchors, grouted anchors, or power driven anchors such as powder or pneumatic actuated fasteners.

17.1.7 Reinforcement used as part of an embedment shall have development length established in accordance with other parts of this Code. If reinforcement is used as anchorage, concrete breakout failure shall be considered. Alternatively, anchor reinforcement in accordance with 17.5.2.1 shall be provided.

17.1.8 To evaluate the anchorage of bar groups in tension, it shall be permitted to use the procedures of 25.4.11 to evaluate concrete breakout strength.

17.1.9 To evaluate the strength of anchor groups in tension, it shall be permitted to apply the procedure of 25.4.11 except that the values of N_{cbg} and N_{ag} shall be determined in accordance with 17.6.2 and 17.6.5.

17.2—General

17.2.1 Anchors and anchor groups shall be designed for critical effects of factored loads calculated by elastic analysis. If nominal strength is controlled by ductile steel elements, plastic analysis is permitted provided that deformation compatibility is taken into account.

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R17.1.4 ACI CODE-355.2 prohibits reuse of post-installed mechanical anchors.

R17.1.5 The exclusion of load applications producing high-cycle fatigue or extremely short duration impact (such as blast or shock wave) from the scope of this chapter is not meant to exclude earthquake loads. Section 17.10 presents additional requirements for design when earthquake loads are included.

R17.1.6 The wide variety of shapes and configurations of specialty inserts precludes prescription of generalized tests and design equations.

R17.1.7 Under certain conditions, multiple closely spaced bars can generate concrete breakout failure even if reinforcement meets the development length requirements of Chapter 25. For reinforcing bar groups, 25.4.11 provides a procedure for determining the concrete breakout strength of the group.

As an alternative to explicit determination of the concrete breakout strength of a group, anchor reinforcement provided in accordance with 17.5.2.1 may be used.

R17.1.8 The development of reinforcing bar groups in tension requires an evaluation that the breakout strength of the member in which the bars are terminated is adequate. Modifications to the breakout provisions of Chapter 17 are provided in 25.4.11 to enable this evaluation.

R17.1.9 The procedure of 25.4.11 can also be applied to the tension breakout strength of anchors in concrete as determined in accordance with Chapter 17. In this case, the breakout strength is based on the 5 percent fractile and cracked concrete is considered in accordance with 17.6.2 and 17.6.5. For specific cases, this may have advantages over the use of anchor reinforcement in accordance with 17.5.2.1(a).

R17.2—General

R17.2.1 If the strength of an anchor group is governed by concrete breakout, the behavior is brittle, and there is limited redistribution of 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. Cook and Klingner (1992a,b) and Lotze et al. (2001) discuss nonlinear analysis, using theory of plasticity, for the determination of the strengths of ductile anchor groups.

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17.2.1.1 Anchor group effects shall be considered if two or more anchors loaded by a common structural element are spaced closer than the spacing required for unreduced breakout strength. If adjacent anchors are not loaded by a common structural element, group effects shall consider simultaneous maximum loading of adjacent anchors.

17.2.2 Adhesive anchors shall be installed in concrete having a minimum age of 21 days at time of anchor installation.

17.2.3 Adhesive anchors installed horizontally or upwardly inclined shall be qualified in accordance with ACI CODE-355.4 requirements for sensitivity to installation direction.

17.2.4 *Lightweight concrete modification factor,* λ_a

17.2.4.1 Modification factor λ_a for lightweight concrete shall be in accordance with Table 17.2.4.1. It shall be permitted to use an alternate value of λ_a if tests are performed and evaluated in accordance with ACI CODE-355.2 or ACI CODE-355.4.

Table 17.2.4.1—Modification factor λ_a for lightweight concrete

Case	λ_a[1]
Cast-in and undercut anchor concrete failure	1.0λ
Expansion, screw, and adhesive anchor concrete failure	0.8λ
Adhesive anchor bond failure per Eq. (17.6.5.2.1)	0.6λ

[1]λ shall be in accordance with 19.2.4.

17.2.5 Anchors shall be installed and inspected in accordance with the requirements of 26.7 and 26.13.

17.3—Design Limits

17.3.1 The value of f_c' used for calculation purposes in this chapter shall not exceed 10,000 psi for cast-in anchors and 8000 psi for post-installed anchors. Post-installed anchors shall not be used in concrete with a strength greater than 8000 psi without testing to verify acceptable performance.

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R17.2.2 The design performance of adhesive anchors cannot be ensured by establishing a minimum concrete compressive strength at the time of installation in early-age concrete. Therefore, a concrete age of at least 21 days at the time of adhesive anchor installation was adopted.

R17.2.3 ACI CODE-355.4 includes optional tests to confirm the suitability of adhesive anchors for horizontal or upwardly inclined installations.

R17.2.4 *Lightweight concrete modification factor,* λ_a

R17.2.4.1 The number of tests available to establish the strength of anchors in lightweight concrete is limited. Tests of headed studs cast in lightweight concrete indicate that the present reduction factor λ adequately represents the influence of lightweight concrete (Shaikh and Yi 1985; Anderson and Meinheit 2005). Anchor manufacturer data developed for evaluation reports on post-installed expansion, screw, undercut, and adhesive anchors indicate that a reduced λ is needed to provide the necessary safety factor for the respective design strength. ACI CODE-355.2 and ACI CODE-355.4 provide procedures whereby a specific value of λ_a can be used based on testing, assuming the lightweight concrete is similar to the reference test material.

R17.3—Design Limits

R17.3.1 A limited number of tests of cast-in and post-installed anchors in high-strength concrete (Primavera et al. 1997) indicate that the design procedures contained in this chapter become unconservative with increasing concrete strength, 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 for the design of cast-in anchors. This limitation is consistent with those for shear strength, torsion strength, and reinforcement development length in the Code (22.5.3.1, 22.6.3.1, 22.7.2.1, 25.4.1.4). For some post-installed anchors, the capacity may be negatively affected by very high-strength concrete. These effects are associated with difficulty in fully expanding expansion anchors, cutting grooves in the sidewall of the predrilled hole by the screw anchor's threads, and reduced bond strength of adhesive anchors. The 8000 psi limit for post-installed anchors reflects

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17.3.2 For anchors with diameters $d_a \leq 4$ in., concrete breakout strength requirements shall be considered satisfied by the design procedures of 17.6.2 and 17.7.2.

17.3.3 For adhesive anchors with embedment depths $4d_a \leq h_{ef} \leq 20d_a$, bond strength requirements shall be considered satisfied by the design procedure of 17.6.5.

17.3.4 For screw anchors with embedment depths $5d_a \leq h_{ef} \leq 10d_a$, and $h_{ef} \geq 1.5$ in., concrete breakout strength requirements shall be considered satisfied by the design procedures of 17.6.2 and 17.7.2.

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the current concrete strength range for testing specified in ACI CODE-355.2 and ACI CODE-355.4. The 8000 psi limit may be exceeded if verified with tests.

R17.3.2 The limitation on anchor diameter is based on the current range of test data. In the 2002 through 2008 editions of the Code, there were limitations on the diameter and embedment of anchors to calculate the concrete breakout strength. These limitations were necessitated by the lack of test results on anchors with diameters larger than 2 in. and embedment lengths longer than 24 in. In 2011, limitations on anchor diameter and embedment length were revised to limit the diameter to 4 in. based on the results of tension and shear tests on large-diameter anchors with deep embedments (Lee et al. 2007, 2010). These tests included 4.25 in. diameter anchors, embedded 45 in., tested in tension and 3 in. diameter anchors tested in shear. The 4 in. diameter limit was selected to maintain consistency with the largest diameter anchor permitted in ASTM F1554. Other ASTM specifications permit up to 8 in. diameter anchors; however, they have not been tested to ensure applicability of the 17.6.2 and 17.7.2 concrete breakout provisions.

R17.3.3 ACI CODE-355.4 limits the embedment depth of adhesive anchors to $4d_a \leq h_{ef} \leq 20d_a$, which represents the theoretical limits of the bond model (Eligehausen et al. 2006a).

R17.3.4 Screw anchor research by Olsen et al. (2012) is based on the nominal screw anchor diameter corresponding to the nominal drill bit size (for example, a 5/8 in. screw anchor installs in a hole drilled by a 5/8 in. ANSI drill bit). This definition of screw anchor size is approximately the diameter of the core or shank of the screw rather than the size of the larger external diameter of the thread. This definition differs from the diameter of standard anchors with ASME B1.1 threads that have a reduced shaft area and smaller effective area. The effective area of the screw anchor, as with other post-installed mechanical anchors, is provided by the manufacturer.

The Olsen et al. (2012) empirical design model was derived from a database of tests in cracked and uncracked concrete on metric-sized screw anchors tested in Europe and inch-sized anchors tested by independent laboratories in accordance with ICC-ES AC193.

For concrete screw anchors, the effective embedment depth, h_{ef}, is determined as a reduction from the nominal embedment based on geometric characteristics of the screw. The effective embedment is verified during the qualification testing under ACI CODE-355.2 and provided by the manufacturer for use in design. Using the reduced, effective embedment depth with the concrete capacity design (CCD) method is shown to adequately represent the behavior of concrete screws in the current concrete screw database and also validates the effects and limitations of certain relevant parameters, such as the effective embedment depth and spacing of anchors (17.9).

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17.3.5 Anchors shall satisfy the edge distances, spacings, and thicknesses in 17.9 unless supplementary reinforcement is provided to control splitting failure.

17.4—Required strength

17.4.1 Required strength shall be calculated in accordance with the factored load combinations in Chapter 5.

17.4.2 For anchors in structures assigned to SDC C, D, E, and F, the additional requirements of 17.10 shall apply.

17.5—Design strength

17.5.1 For each applicable factored load combination, design strength of individual anchors and anchor groups shall satisfy $\phi\psi_a S_n \geq U$. Interaction between load effects shall be considered in accordance with 17.8.1.

17.5.1.1 Strength reduction factor, ϕ, shall be determined in accordance with 17.5.3.

17.5.1.2 ψ_a shall be determined in accordance with 17.5.4.1.

17.5.1.3 Nominal strength for an anchor or anchor groups shall be based on design models that result in predictions of strength in substantial agreement with results of comprehensive tests. The materials used in the tests shall be compatible with the materials used in the structure. The nominal strength shall be based on the 5 percent fractile of the basic individual anchor strength. For nominal strengths related to concrete strength, modifications for size effects, number of anchors, effects of close spacing of anchors, proximity to edges, depth of the concrete member, eccentric loadings of anchor groups, and influence of cracking shall be taken into account. Limits on edge distance and anchor spacing in the design models shall be consistent with the tests that verified the model. Strength of anchors shall be based on design models that satisfy 17.5.1.3 for the following:

(a) Steel strength of anchor in tension
(b) Concrete breakout strength of anchor in tension
(c) Pullout strength of a single cast-in anchor and single post-installed expansion, screw, and undercut anchor in tension
(d) Concrete side-face blowout strength of headed anchor in tension
(e) Bond strength of adhesive anchor in tension
(f) Steel strength of anchor in shear
(g) Concrete breakout strength of anchor in shear
(h) Concrete pryout strength of anchor in shear

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R17.4—Required strength

R17.5—Design strength

R17.5.1.3 This section provides requirements for establishing the strength of anchors in concrete. The various types of steel and concrete failure modes for anchors are shown in Fig. R17.5.1.3(a) and R17.5.1.3(b). Comprehensive discussions of anchor failure modes are included in CEB (1997), Fuchs et al. (1995), Eligehausen and Balogh (1995), and Cook et al. (1998). Tension failure modes related to concrete include concrete breakout failure (applicable to all anchor types), pullout failure (applicable to cast-in anchors, post-installed expansion, screw, and undercut anchors), side-face blowout failure (applicable to headed anchors), and bond failure (applicable to adhesive anchors). Shear failure modes related to concrete include concrete breakout failure and concrete pryout (applicable to all anchor types). These failure modes are described in the deemed-to-comply provisions of 17.6.2, 17.6.3, 17.6.4, 17.6.5, 17.7.2, and 17.7.3.

Any model that complies with the requirements of 17.5.1.3 and 17.5.2.3 can be used to establish the concrete-related strengths. Additionally, anchor tensile and shear strengths are limited by the minimum spacings and edge distances of 17.9 to preclude splitting. The design of post-installed anchors recognizes that the strength of anchors is sensitive to appropriate installation; installation requirements are included in Chapter 26. Some post-installed anchors are less sensitive to installation errors and tolerances. This is reflected in various ϕ-factors given in 17.5.3 and based on the assessment criteria of ACI CODE-355.2 and ACI CODE-355.4.

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 (refer to R17.7.2.1).

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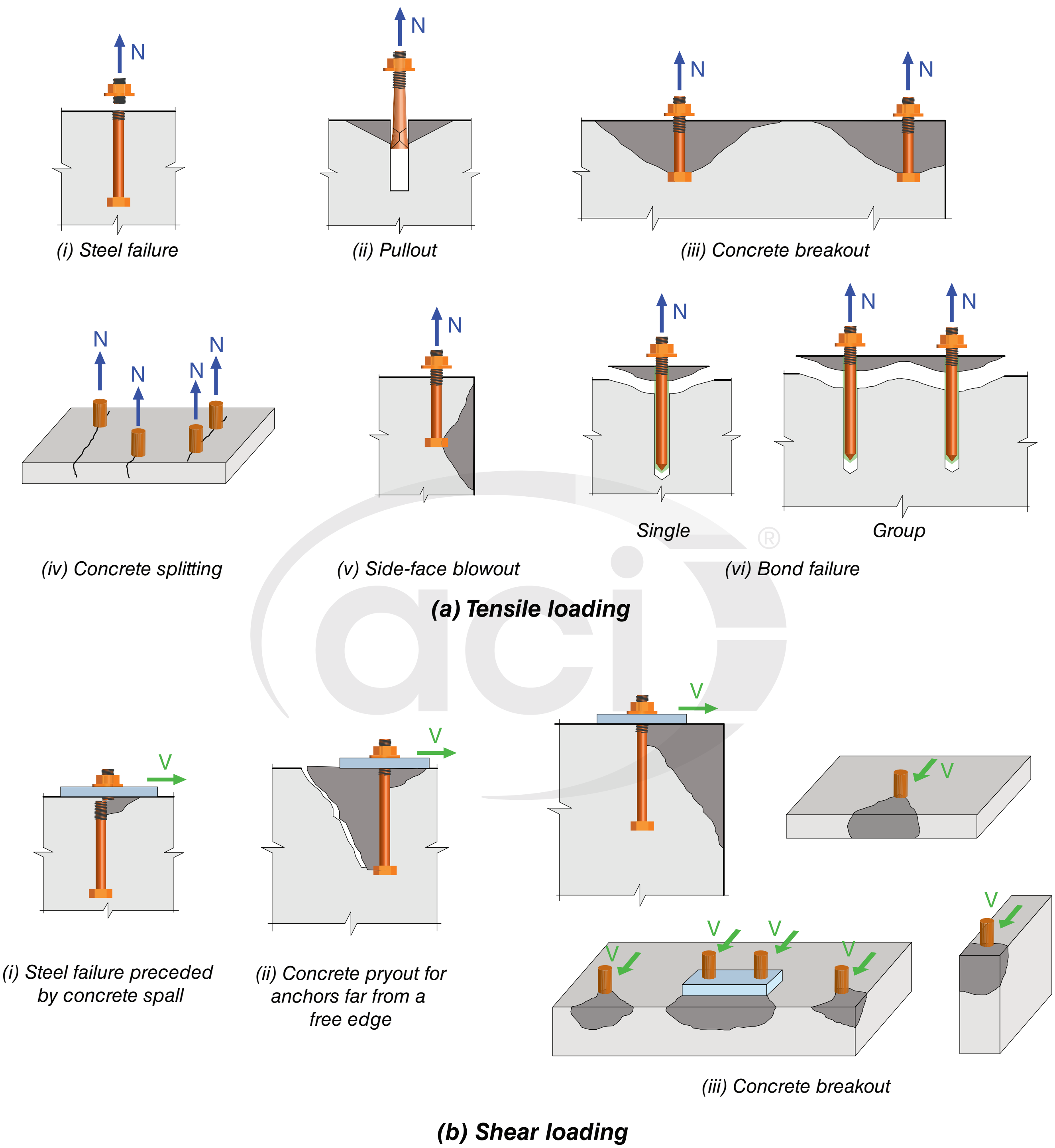

Fig. R17.5.1.3—Failure modes for anchors.

17.5.1.4 Strength of anchors shall be permitted to be determined in accordance with 17.6 for 17.5.1.3(a) through (e), and 17.7 for 17.5.1.3(f) through (h). For adhesive anchors that resist sustained tension, the requirements of 17.5.2.2 shall apply.

R17.5.1.4 The method for concrete breakout design deemed to comply with the requirements of 17.5.1.3 was developed from the concrete capacity design (CCD) Method (Fuchs et al. (1995); Eligehausen and Balogh (1995)), which was an adaptation of the Kappa Method (Eligehausen and Fuchs 1988; Eligehausen et al. 2006a) with a breakout failure

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17.5.1.4.1 Anchor group effects shall be considered wherever two or more anchors have spacing less than the critical spacing in Table 17.5.1.4.1, where only those anchors susceptible to the particular failure mode under investigation shall be included in the group.

Table 17.5.1.4.1—Critical spacing

Failure mode under investigation	Critical spacing
Concrete breakout in tension	$3h_{ef}$
Bond strength in tension	$2c_{Na}$
Concrete breakout in shear	$3c_{a1}$

17.5.1.5 Strength of anchors shall be permitted to be based on test evaluation using the 5% fractile of applicable test results for 17.5.1.3 (a) through (h).

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surface angle of approximately 35 degrees (Fig. 17.5.1.4a and b). It is considered to be sufficiently accurate, relatively easy to apply, and capable of extension to irregular layouts. The CCD Method predicts the strength of an anchor or anchor group by using a basic equation for tension in cracked concrete, which is multiplied by factors that account for the number of anchors, edge distance, spacing, eccentricity, and absence of cracking. For shear, a similar approach is used. Experimental and numerical investigations have demonstrated the applicability of the CCD Method to adhesive anchors as well (Eligehausen et al. 2006a).

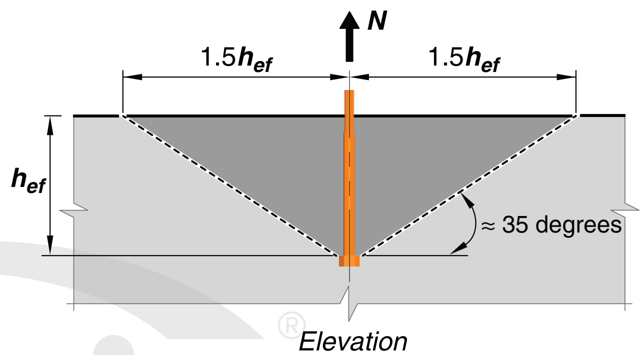

Fig. R17.5.1.4a—Breakout cone for tension.

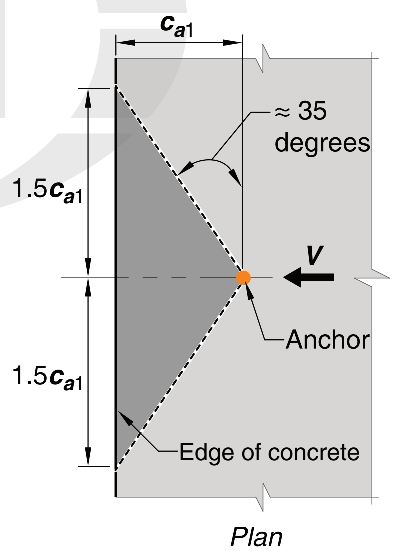

Fig. R17.5.1.4b—Breakout cone for shear.

R17.5.1.5 Sections 17.5.1.3 and 17.5.2.3 establish the performance factors for which anchor design models are required to be verified. Many possible design approaches exist, and the user is always permitted to "design by test" using 17.5.1.5 as long as sufficient data are available to verify the model. Test procedures can 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 provi-

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17.5.2 For each applicable factored load combination, design strength of anchors shall satisfy the criteria in Table 17.5.2. Design strength of anchors loaded in both tension and shear shall be in accordance with 17.8.

Table 17.5.2—Design strength requirements of anchors

Failure mode	Single anchor	Anchor group[1]: Individual anchor in a group	Anchor group[1]: Anchors as a group
Steel strength in tension (17.6.1)[2]	$\phi N_{sa} \geq N_{ua}$	$\phi N_{sa} \geq N_{ua,i}$	
Concrete breakout strength in tension[3] (17.6.2)	$\phi N_{cb} \geq N_{ua}$		$\phi N_{cbg} \geq N_{ua,g}$
Pullout strength in tension (17.6.3)	$\phi N_{pn} \geq N_{ua}$	$\phi N_{pn} \geq N_{ua,i}$	
Concrete side-face blowout strength in tension (17.6.4)	$\phi N_{sb} \geq N_{ua}$		$\phi N_{sbg} \geq N_{ua,g}$
Bond strength of adhesive anchor in tension (17.6.5)	$\phi N_{a} \geq N_{ua}$		$\phi N_{ag} \geq N_{ua,g}$
Steel strength in shear (17.7.1)	$\phi V_{sa} \geq V_{ua}$	$\phi V_{sa} \geq V_{ua,i}$	
Concrete breakout strength in shear[3] (17.7.2)	$\phi V_{cb} \geq V_{ua}$		$\phi V_{cbg} \geq V_{ua,g}$
Concrete pryout strength in shear (17.7.3)	$\phi V_{cp} \geq V_{ua}$		$\phi V_{cpg} \geq V_{ua,g}$

[1]Design strengths for steel and pullout failure modes shall be calculated for the most highly stressed anchor in the group.
[2]Sections referenced in parentheses are pointers to models that are permitted to be used to evaluate the nominal strengths.
[3]If anchor reinforcement is provided in accordance with 17.5.2.1, the design strength of the anchor reinforcement shall be permitted to be used instead of the concrete breakout strength.

17.5.2.1 *Anchor reinforcement*

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sions of 17.5.1.3. The basic strength cannot be taken greater than the 5% fractile. The number of tests has to be sufficient for statistical validity and should be considered in the determination of the 5% fractile.

R17.5.2 Under combined tension and bending, individual anchors in a group may be required to resist different magnitudes of tensile force. Similarly, under combined shear and torsion, individual anchors in a group may be required to resist different magnitudes of shear. Table 17.5.2 includes requirements to design single anchors and individual anchors in a group to safeguard against all potential failure modes. For steel and pullout failure modes, the most highly stressed anchor in the group should be checked to ensure it has sufficient strength to resist its required load. For concrete breakout, the anchors should be checked as a group. Elastic analysis or plastic analysis of ductile anchors as described in 17.2.1 may be used to determine the loads resisted by each anchor.

The addition of reinforcement in the direction of the load to restrain concrete breakout can enhance the strength and deformation capacity of the anchor connection. Such enhancement is practical with cast-in anchors such as those used in precast sections. Klingner et al. (1982), *fib* (2011), ACI CODE-349, and Eligehausen et al. (2006b) provide information regarding the effect of reinforcement on the behavior of anchors. The effect of reinforcement is not included in the ACI CODE-355.2 and ACI CODE-355.4 anchor acceptance tests or in the concrete breakout calculation method of 17.6.2 and 17.7.2. Anchor reinforcement may be provided in accordance with 17.5.2.1 and developed according to Chapter 25 instead of calculating breakout strength.

R17.5.2.1 *Anchor reinforcement*

For conditions where the factored tensile or shear force exceeds the concrete breakout strength of the anchor(s) or if the breakout strength is not evaluated, the nominal strength can be based on developed anchor reinforcement as illustrated in Fig. R17.5.2.1a for tension and Fig. R17.5.2.1b(i) and Fig. R17.5.2.1b(ii) for shear. If anchor reinforcement is provided in accordance with 17.5.2.1.1 or 17.5.2.1.2, the strength of the connection is assumed to be controlled by yielding of the anchor reinforcement as opposed to concrete breakout, and the strength reduction factor applied to these cases is given in Table 21.2.1. Because anchor rein-

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forcement is placed below where the shear is applied (refer to Fig. R17.5.2.1b), the force in the anchor reinforcement will be larger than the shear force. Anchor reinforcement is distinguished from supplementary reinforcement in that it is designed exclusively for the anchor loads and is intended to preclude concrete breakout. Strut-and-tie models may be used to design anchor reinforcement. For practical reasons, anchor reinforcement is only used for cast-in anchor applications.

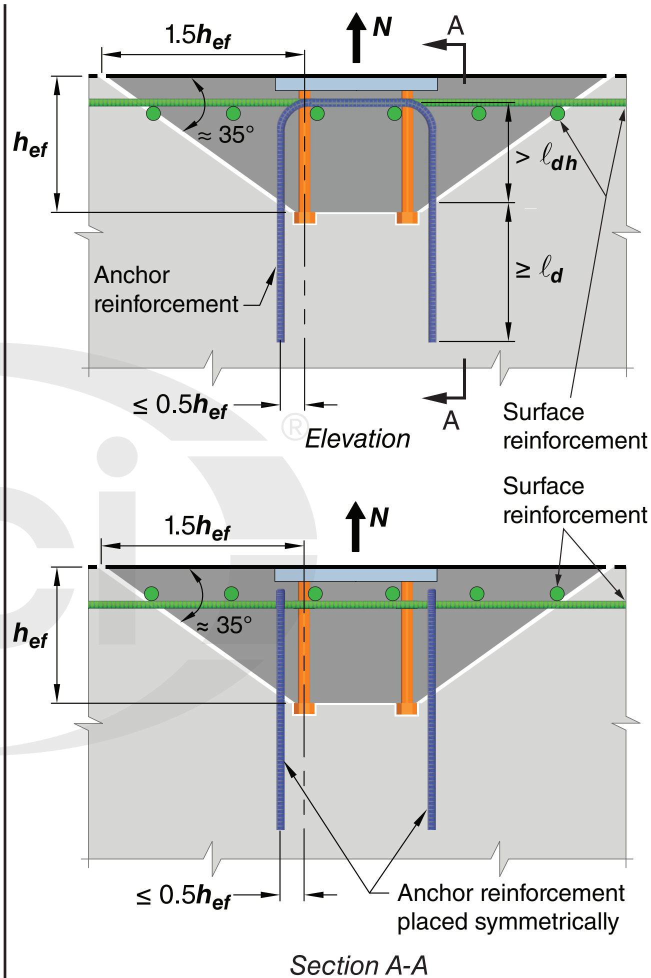

Fig. R17.5.2.1a—Anchor reinforcement for tension.

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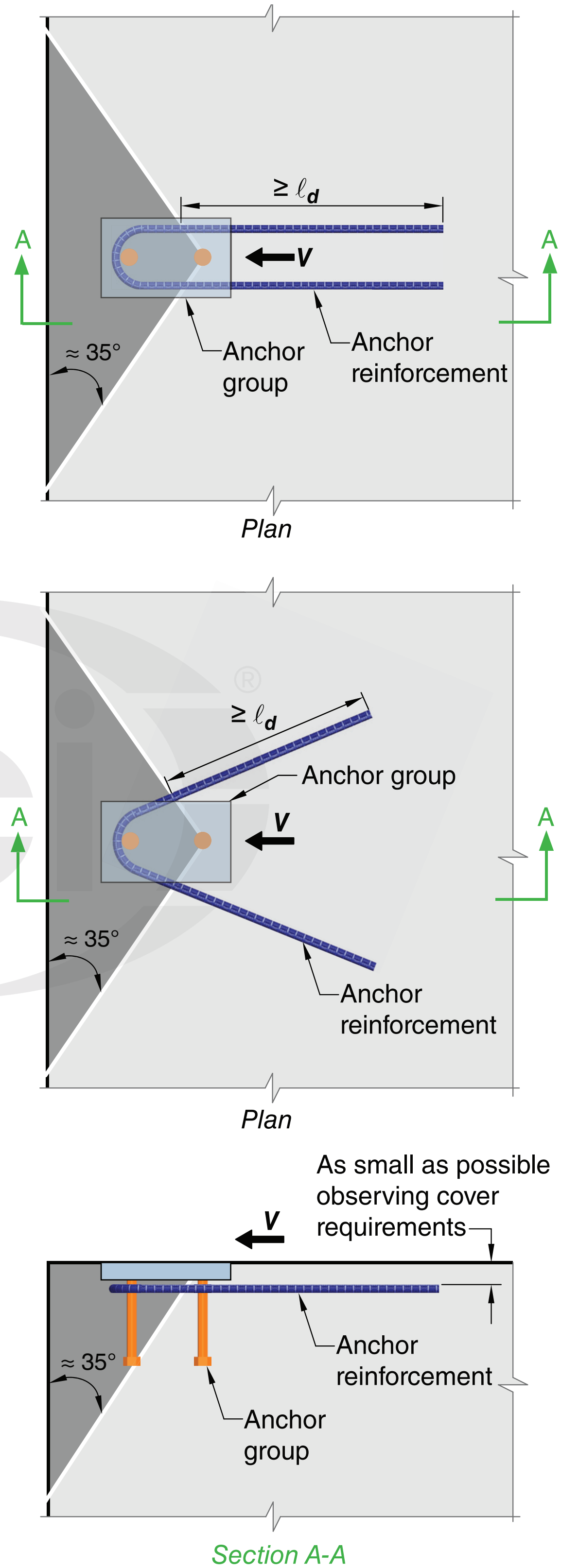

Fig. R17.5.2.1b(i)—Hairpin anchor reinforcement for shear.

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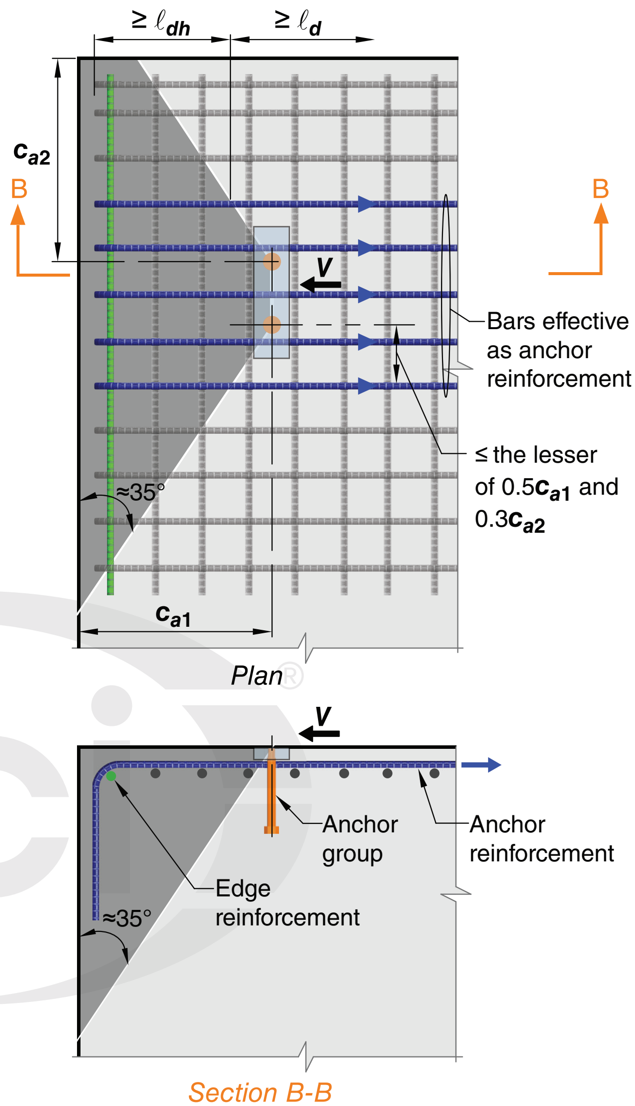

Fig. R17.5.2.1b(ii)—Edge reinforcement and anchor reinforcement for shear.

17.5.2.1.1 For anchors in tension, the design strength of anchor reinforcement shall be permitted to be used instead of the concrete tensile breakout strength of 17.6.2 if (a) and (b) are satisfied.

(a) Anchor reinforcement is developed in accordance with Chapter 25 on both sides of the concrete breakout surface

(b) Anchor reinforcement legs crossing the breakout failure plane are parallel to the applied tension force.

R17.5.2.1.1 Care needs to be taken in the selection and positioning of anchor reinforcement for tension. Ideally tension anchor reinforcement should consist of stirrups, ties, or hairpins placed as close as practicable to the anchor. It is beneficial for the anchor reinforcement to enclose the surface reinforcement where applicable. Anchor reinforcement spaced less than $0.5h_{ef}$ from the anchor centerline may be considered as effective. The research (Eligehausen et al. 2006b) on which these provisions are based was limited to anchor reinforcement with maximum diameter equivalent to a No. 5 bar.

17.5.2.1.2 For anchors in shear, the design strength of anchor reinforcement shall be permitted to be used instead of the concrete shear breakout strength of 17.7.2 if (a) and (b) are satisfied.

R17.5.2.1.2 To ensure development of anchor reinforcement for shear, the enclosing anchor reinforcement shown in Fig. R17.5.2.1(b)(i) should be in contact with the anchor and placed as close as practicable to the concrete surface. The research (Eligehausen et al. 2006b) on which the provi-

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(a) Anchor reinforcement is developed in accordance with Chapter 25 on both sides of the concrete breakout surface, or encloses and contacts the anchor or anchors and is developed beyond the breakout surface.
(b) Anchor reinforcement legs crossing the breakout failure plane are parallel to the applied shear force.

17.5.2.1.3 The resistance of anchor reinforcement not parallel to the force applied shall be resolved into parallel and perpendicular components. Only the parallel component shall be permitted to contribute to the resistance in tension.

17.5.2.1.4 Shear friction provisions of 22.9 shall not be used for the design of anchor reinforcement.

17.5.2.1.5 ϕ for anchor reinforcement shall be in accordance with Table 21.2.1.

17.5.2.2 Design of adhesive anchors to resist sustained tension shall satisfy Eq. (17.5.2.2)

$$0.55\psi_a\phi N_{ba} \geq N_{ua,s} \tag{17.5.2.2}$$

where N_{ba} is basic bond strength in tension of a single adhesive anchor and $N_{ua,s}$ is the factored sustained tensile load.

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sions for enclosing reinforcement are based was limited to anchor reinforcement with maximum diameter equivalent to a No. 5 bar. The larger bend radii associated with larger bar diameters may significantly reduce the effectiveness of the anchor reinforcement for shear; therefore, anchor reinforcement larger than a No. 6 bar is not recommended. Because development for full f_y is required, the use of excess reinforcement to reduce development length is not permitted for anchor reinforcement.

The anchor reinforcement for shear may also consist of stirrups, ties, hoops, or hairpins enclosing the edge reinforcement embedded in the breakout volume and placed as close to the anchors as practicable (refer to Fig. R17.5.2.1b(ii)). Generally, reinforcement spaced less than the smaller of $0.5c_{a1}$ and $0.3c_{a2}$ from the anchor centerline should be included as anchor reinforcement. In this case, the anchor reinforcement must be developed on both sides of the breakout surface. For equilibrium, edge reinforcement is required. The research on which these provisions are based was limited to anchor reinforcement with maximum diameter equivalent to a No. 6 bar.

R17.5.2.1.4 Surface reinforcement, as illustrated in Fig. R17.5.2.1a, is ineffective as anchor reinforcement for force components perpendicular to the surface and does not contribute to breakout resistance by the shear friction model described in 22.9. Anchor reinforcement prevents concrete breakout by establishing a tensile load path directly from the anchor to the anchor reinforcement and to the supporting member (Eligehausen et al. 2006b).

R17.5.2.2 For adhesive anchors that resist sustained tensile load, an additional calculation for the sustained portion of the factored load for a reduced bond resistance is required to account for possible bond strength reductions under sustained tension. The resistance of adhesive anchors to sustained tension is particularly dependent on correct installation, including hole cleaning, adhesive metering and mixing, and prevention of voids in the adhesive bond line (annular gap). In addition, care should be taken in the selection of the correct adhesive and bond strength for the expected on-site conditions such as the concrete condition during installation (dry or saturated, cold or hot), the drilling method used (rotary impact drill, rock drill, or core drill), and anticipated in-service temperature variations in the concrete.

The 0.55 factor used for the additional calculation for sustained tension is correlated with ACI CODE-355.4 test

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requirements and provides satisfactory performance of adhesive anchors under sustained tensile loads in accordance with ACI CODE-355.4. Product evaluation according to ACI CODE-355.4 is based on sustained tensile loads being present for 50 years at a standard temperature of 70°F and 10 years at a temperature of 110°F. For longer life spans (for example, greater than 50 years) or higher temperatures, lower factors should be considered. Additional information on use of adhesive anchors for such conditions can be found by consulting with the adhesive manufacturer.

Adhesive anchors are particularly sensitive to installation direction and load type. Adhesive anchors installed overhead that resist sustained tension are of concern because previous applications of this type have led to failures (National Transportation Safety Board 2007). Other anchor types may be more appropriate for such cases. For adhesive anchors that resist sustained tension in horizontal or upwardly inclined orientations, it is essential to meet test requirements of ACI CODE-355.4 for sensitivity to installation direction, use certified installers, and require special inspection. Inspection and installation requirements are provided in Chapter 26.

17.5.2.2.1 For groups of adhesive anchors subject to sustained tension, Eq. (17.5.2.2) shall be satisfied for the anchor that resists the highest sustained tension.

R17.5.2.2.1 The check for anchor groups is limited to the highest loaded anchor in the group, analogous to the design for pullout.

17.5.2.3 If both N_{ua} and V_{ua} are present, interaction effects shall be considered using interaction expressions resulting in calculated strengths in substantial agreement with results of comprehensive tests. This requirement shall be considered satisfied by 17.8 for interaction effects associated with concrete strength and steel strength.

R17.5.2.3 Section 17.8 considers interaction effects for concrete and steel strength separately.

17.5.2.4 Anchors shall satisfy the edge distances, spacings, and thicknesses in 17.9 to preclude splitting failure.

17.5.2.5 Anchors in structures assigned to Seismic Design Category C, D, E, or F shall satisfy the additional requirements of 17.10.

17.5.2.6 Attachments with shear lugs used to transfer structural loads shall satisfy the requirements of 17.11.

17.5.3 ϕ for anchors in concrete and anchor reinforcement shall be in accordance with Table 21.2.1.

R17.5.3 The ϕ-factors for anchors in concrete have been simplified and consolidated in Table 21.2.1 in the 2025 Code. The following adjustments have been implemented: 1) Separate strength reduction factors for anchors have been replaced with a new ψ_a factor (refer to 17.5.4) corresponding to evaluation results in accordance with the relevant standard and whether supplementary reinforcement is present, and 2) the Code distinguishes between redundant and nonredundant anchorages. Redundancy may be assumed where it can be shown that failure of a single anchor or anchorage point does not result in loss of position retention or progressive collapse. The reliability index (target reliability) associated with the

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provisions in Chapter 17 has traditionally been interpreted as corresponding to a condition where: a) warning of failure is minimal (concrete failure); b) failure of the anchorage will result in collapse of the supported element (nonredundant); and c) the consequences of failure are significant. For anchorages in tension where redundancy is inherent in the connection, an increased ϕ as provided in Table 21.2.1 is acceptable.

17.5.4 *Anchor strength modification factor* ψ_a

17.5.4.1 In addition to the strength reduction factor ϕ, the strength of anchors associated with concrete failure modes shall be modified by the factor ψ_a in accordance with Table 17.5.4.1.

Table 17.5.4.1—Anchor strength modification factor ψ_a

Anchor type	Anchor Category[1] from ACI 355.2 or ACI 355.4	ψ_a concrete failure: supplementary reinforcement not present — Tension	supplementary reinforcement not present — Shear	supplementary reinforcement present — Tension	supplementary reinforcement present — Shear
Cast-in	not applicable	0.95	0.95	1.00	1.00
Post-installed	1	0.85		1.00	
	2	0.75		0.85	
	3	0.60		0.75	

[1]Anchor Category 1 indicates low installation sensitivity and high reliability; Anchor Category 2 indicates medium installation sensitivity and medium reliability; Anchor Category 3 indicates high installation sensitivity and lower reliability.

R17.5.4 *Anchor strength modification factor* ψ_a

R17.5.4.1 Tests described in ACI CODE-355.2 and ACI CODE-355.4 to assess sensitivity to installation procedures are used to establish the Anchor Categories listed in Table 17.5.4.1 for proprietary post-installed expansion, screw, undercut, and adhesive anchors. ACI CODE-355.2 tests for installation sensitivity measure effects of variability in anchor torque during installation, tolerance on drilled hole size, and energy level used in setting anchors. For expansion, screw, and undercut anchors intended for use in cracked concrete, the effects of cracks passing through or near the anchor location are measured. ACI CODE-355.4 tests for installation sensitivity assess the influence of adhesive mixing and hole cleaning in dry, saturated, and water-filled/underwater bore holes on the performance of adhesive anchors. In addition, for adhesive anchors intended for use in cracked concrete, the effect of concrete cracking on anchor behavior is assessed.

Applications with supplementary reinforcement provide more deformation capacity, permitting the ψ_a-factors to be increased. An explicit design of supplementary reinforcement for anchor-related forces is not required; however, the arrangement of supplementary reinforcement should generally conform to that of the anchor reinforcement shown in Fig. R17.5.2.1(a) and R17.5.2.1(b)(i) and (ii). Unlike anchor reinforcement, development of supplementary reinforcement beyond the assumed breakout failure plane is not required. Refer also to R17.5.2.1.

17.6—Tensile strength

17.6.1 *Steel strength of anchors in tension,* N_{sa}

17.6.1.1 Nominal steel strength of anchors in tension as governed by the steel, N_{sa}, shall be evaluated based on the properties of the anchor material and the physical dimensions of the anchors.

17.6.1.2 Nominal steel strength of an anchor in tension, N_{sa}, shall be calculated by:

$$N_{sa} = A_{se,N} f_{uta} \qquad (17.6.1.2)$$

where $A_{se,N}$ is the effective cross-sectional area of an anchor in tension, in.^{2}, and f_{uta} used for calculations shall not exceed either $1.9f_{ya}$ or 125,000 psi.

R17.6—Tensile strength

R17.6.1 *Steel strength of anchors in tension,* N_{sa}

R17.6.1.2 The nominal strength of anchors in tension is best represented as a function of f_{uta} rather than f_{ya} because the large majority of anchor materials do not exhibit a well-defined yield point. AISC has based tension strength of anchors on $A_{se,N} f_{uta}$ since the 1986 edition of their specifications. The use of Eq. (17.6.1.2) with the load factors provided in 5.3 and the ϕ-factors provided in 17.5.3 result in design strengths consistent with ANSI/AISC 360.

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17.6.2 *Concrete breakout strength of anchors in tension,* N_{cb}

17.6.2.1 Nominal concrete breakout strength in tension, N_{cb} of a single anchor or N_{cbg} of an anchor group satisfying 17.5.1.4.1, shall be calculated by (a) or (b), respectively:

(a) For a single anchor

$$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \psi_a \psi_{ed,N} \psi_{c,N} \psi_{cp,N} \psi_{cm,N} N_b \quad (17.6.2.1a)$$

(b) For an anchor group

$$N_{cbg} = \frac{A_{Nc}}{A_{Nc}} \psi_a \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} \psi_{cm,N} N_b \quad (17.6.2.1b)$$

where ψ_a, $\psi_{ec,N}$, $\psi_{ed,N}$, $\psi_{c,N}$, $\psi_{cp,N}$, and $\psi_{cm,N}$ are given in 17.5.4.1, 17.6.2.3, 17.6.2.4, 17.6.2.5, 17.6.2.6, and 17.6.2.7, respectively.

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The limitation of $1.9f_{ya}$ on f_{uta} is to ensure that, under service load conditions, the anchor does not exceed f_{ya}. Although not a concern for standard structural steel anchors (maximum value of f_{uta}/f_{ya} is 1.6 for ASTM A307), the limitation is applicable to some stainless steels. The limit on f_{uta} of $1.9f_{ya}$ was determined by converting the LRFD provisions to corresponding service level conditions. From 5.3, 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 post-installed anchors having a reduced cross-sectional area anywhere along the anchor length, such as wedge-type anchors, the effective cross-sectional area of the anchor should be provided by the manufacturer. For threaded rods and headed bolts, ASME B1.1 defines $A_{se,N}$ as

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

where n_t is the number of threads per inch.

R17.6.2 *Concrete breakout strength of anchors in tension,* N_{cb}

R17.6.2.1 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. (17.6.2.1a) and (17.6.2.1b).

Figure R17.6.2.1(a) shows A_{Nco} and the development of Eq. (17.6.2.1.4). A_{Nco} is the maximum projected area for a single anchor. Figure R17.6.2.1(b) shows examples of the projected areas for various single-anchor and multiple-anchor arrangements. Because A_{Nc} is the total projected area for an anchor group, and A_{Nco} is the area for a single anchor, there is no need to include n, the number of anchors, in Eq. (17.6.2.1b). 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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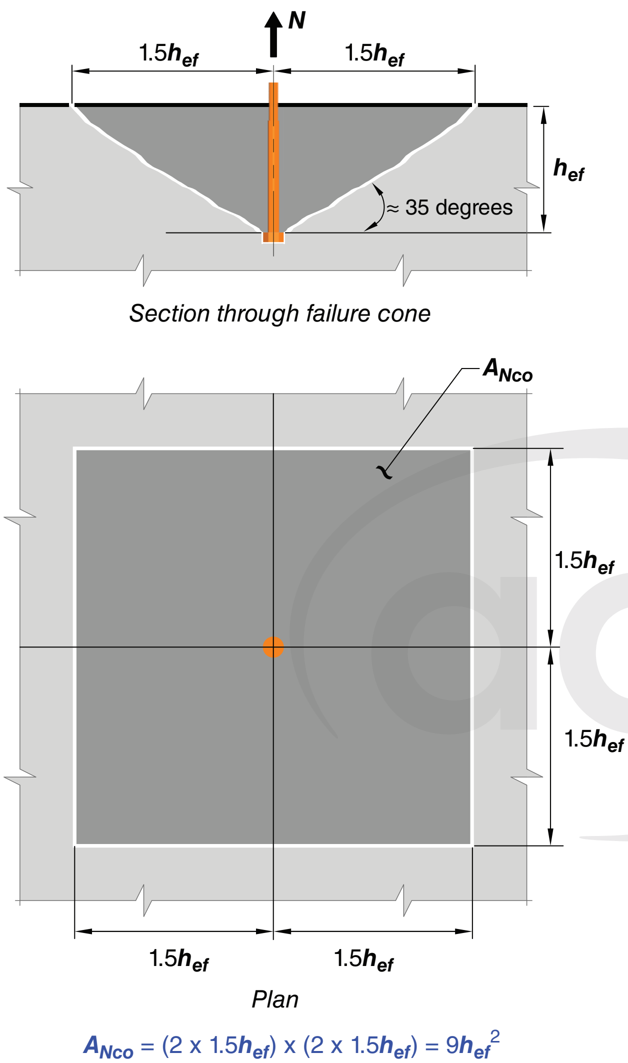
The critical edge distance for headed studs, headed bolts, expansion anchors, and undercut anchors is $1.5h_{ef}$



(a)

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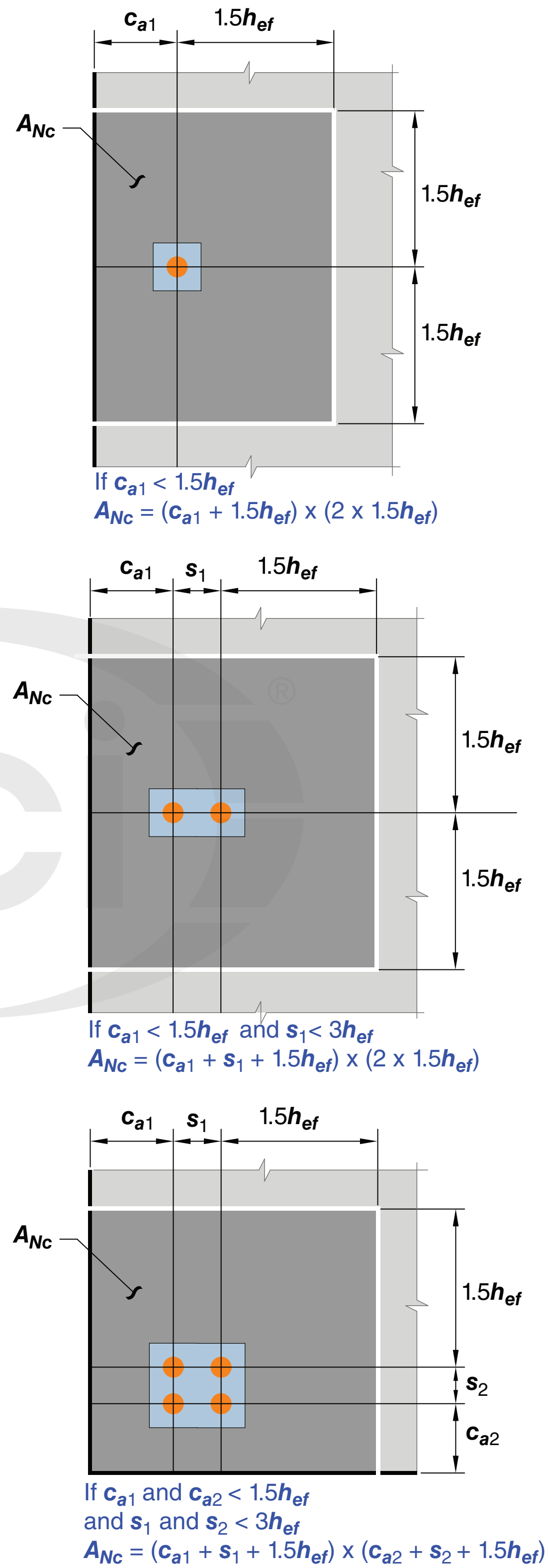

(b)

Fig. R17.6.2.1—(a) Calculation of A_{Nco} *and (b) calculation of* A_{Nc} *for single anchors and anchor groups.*

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17.6.2.1.1 A_{Nc} is the projected concrete failure area of a single anchor or of an anchor group that is approximated as the base of the rectilinear geometrical shape that results from projecting the failure surface outward $1.5h_{ef}$ from the centerlines of the anchor, or in the case of an anchor group, from a line through a row of adjacent anchors. A_{Nc} shall not exceed nA_{Nco}, where n is the number of anchors in the group that resist tension.

17.6.2.1.2 If anchors are located less than $1.5h_{ef}$ from three or more edges, the value of h_{ef} used to calculate A_{Nc} in accordance with 17.6.2.1.1, as well as for the equations in 17.6.2.1 through 17.6.2.4, shall be the greater of (a) and (b):

(a) $c_{a,max}/1.5$
(b) $s/3$, where s is the maximum spacing between anchors within the group.

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R17.6.2.1.2 For anchors located less than $1.5h_{ef}$ from three or more edges, the CCD Method (refer to R17.5.1.4), which is the basis for the equations in 17.6.2.1 through 17.6.2.4, gives overly conservative results for the tensile breakout strength (Lutz 1995). 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 the equations in 17.6.2.1 through 17.6.2.4 to $(c_{a,max})/1.5$, where $c_{a,max}$ is the greatest of the influencing edge distances that do not exceed the actual $1.5h_{ef}$. In no case should $(c_{a,max})/1.5$ 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 use of a calculated strength based on individual breakout volumes for an anchor group configuration. This approach is illustrated in Fig. R17.6.2.1.2. In this example, the proposed limit on the value of h_{ef} to be used in calculations 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 calculating the resistance even if the actual embedment depth is greater.

The requirement of 17.6.2.1.2 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 tensile load. The value of h_{ef} used in 17.6.2.1 through 17.6.2.4 is determined when (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. R17.6.2.1.2, point "A" shows the intersection of the assumed failure surface for limiting h_{ef} with the concrete surface.

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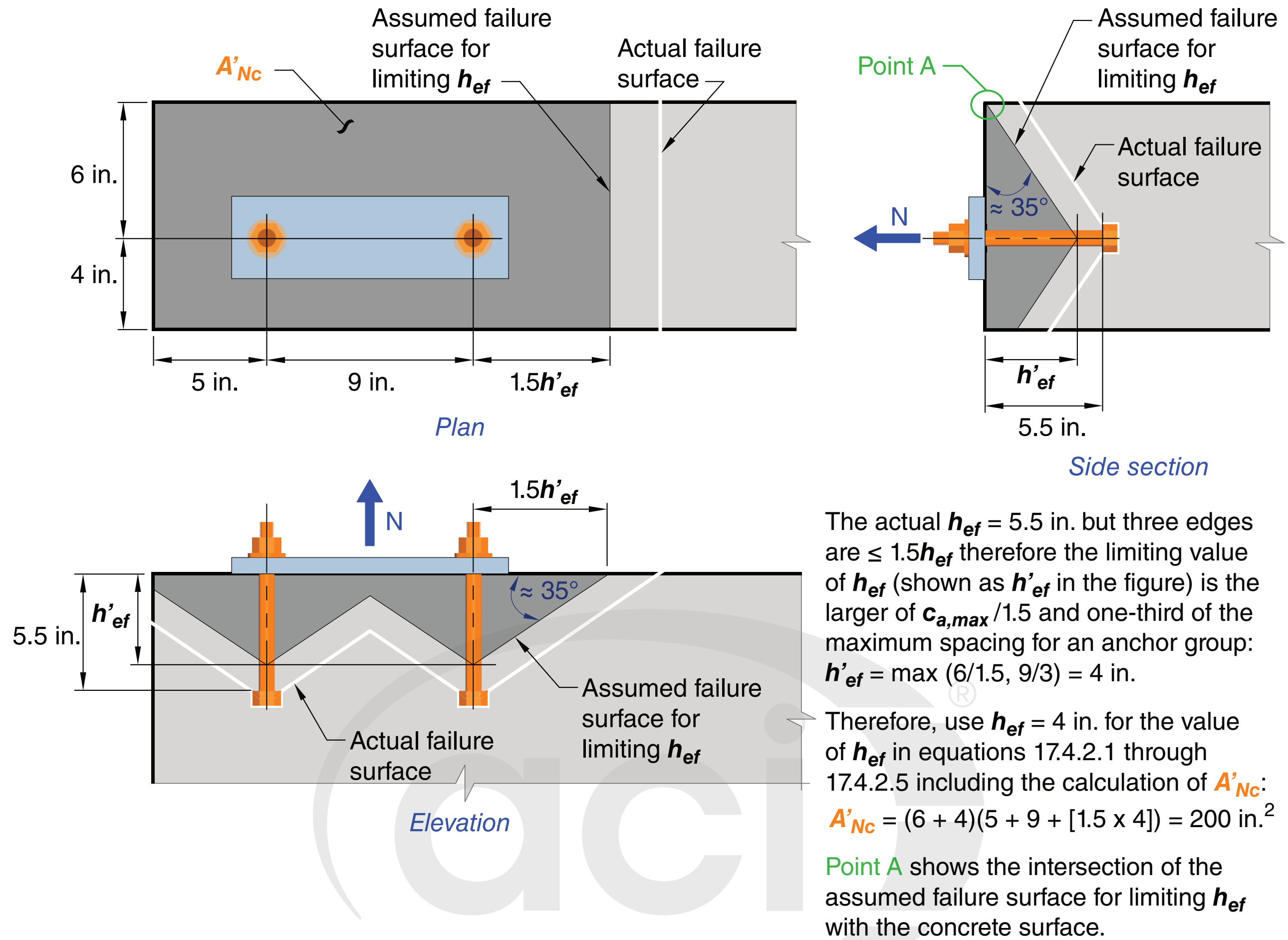

Fig. R17.6.2.1.2—Example of tension where anchors are located in narrow members.

17.6.2.1.3 If 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

17.6.2.1.4 A_{Nco} is the projected concrete failure area of a single anchor with an edge distance of at least $1.5h_{ef}$ and shall be calculated by Eq. (17.6.2.1.4).

$$A_{Nco} = 9h_{ef}^2 \qquad (17.6.2.1.4)$$

17.6.2.2 *Basic single anchor breakout strength*, N_b

17.6.2.2.1 Basic concrete breakout strength of a single anchor in tension in cracked concrete, N_b, shall be calculated by Eq. (17.6.2.2.1), except as permitted in 17.6.2.2.3

R17.6.2.2 *Basic single anchor breakout strength*, N_b

R17.6.2.2.1 The equation for the basic concrete breakout strength was derived assuming concrete breakout with an angle of approximately 35 degrees, considering fracture

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$$N_b = k_c \lambda_a \sqrt{f_c'}\, h_{ef}^{1.5} \qquad (17.6.2.2.1)$$

where k_c = 24 for cast-in anchors and 17 for post-installed anchors.

17.6.2.2.2 k_c for post-installed anchors shall be permitted to be increased based on ACI CODE-355.2 or ACI CODE-355.4 product-specific tests, but shall not exceed 24.

17.6.2.2.3 For single cast-in headed studs and headed bolts with 11 in. $\leq h_{ef} \leq$ 25 in., N_b shall be calculated by:

$$N_b = 16\lambda_a \sqrt{f_c'}\, h_{ef}^{5/3} \qquad (17.6.2.2.3)$$

17.6.2.3 *Breakout eccentricity factor,* $\psi_{ec,N}$

17.6.2.3.1 Modification factor for anchor groups loaded eccentrically in tension, $\psi_{ec,N}$, shall be calculated by Eq. (17.6.2.3.1).

$$\psi_{ec,N} = \frac{1}{\left(1+\dfrac{e'_N}{1.5h_{ef}}\right)} \leq 1.0 \qquad (17.6.2.3.1)$$

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mechanics concepts (Fuchs et al. 1995; Eligehausen and Balogh 1995; Eligehausen and Fuchs 1988; *fib* 2011).

The values of k_c in Eq. (17.6.2.2.1) were determined from a large database of test results in uncracked concrete at the 5% fractile (Fuchs et al. 1995). The values were adjusted to corresponding k_c values for cracked concrete (Eligehausen and Balogh 1995; Goto 1971). Tests have shown that the values of k_c applicable to adhesive anchors are approximately equal to those derived for expansion anchors (Eligehausen et al. 2006a; Zhang et al. 2001).

R17.6.2.2.3 For anchors with a deeper embedment (h_{ef} > 11 in.), test evidence indicates the use of $h_{ef}^{1.5}$ can be overly conservative for some cases. An alternative expression (Eq (17.6.2.2.3)) is provided using $h_{ef}^{5/3}$ for evaluation of cast-in headed studs and headed bolts with 11 in. $\leq h_{ef} \leq$ 25 in. This expression can also be appropriate for some undercut post-installed anchors. However, for such anchors, the use of Eq. (17.6.2.2.3) should be justified by test results in accordance with 17.5.1.5. Experimental and numerical investigations indicate that Eq. (17.6.2.2.3) may be unconservative for h_{ef} > 25 in. if bearing pressure on the anchor head is at or near the limit permitted by Eq. (17.6.3.2.2a) (Ožbolt et al. 2007).

R17.6.2.3 *Breakout eccentricity factor,* $\psi_{ec,N}$

R17.6.2.3.1 Figure 17.6.2.3.1(a) shows an anchor group where all anchors are in tension but the resultant force is eccentric with respect to the centroid of the anchor group. Anchors can also be loaded in such a way that only some of the anchors are in tension (Fig. 17.6.2.3.1(b)). In this case, only the anchors in tension are to be considered for the calculation of e'_N. The eccentricity e'_N of the resultant tensile force is determined with respect to the center of gravity of the anchors in tension.

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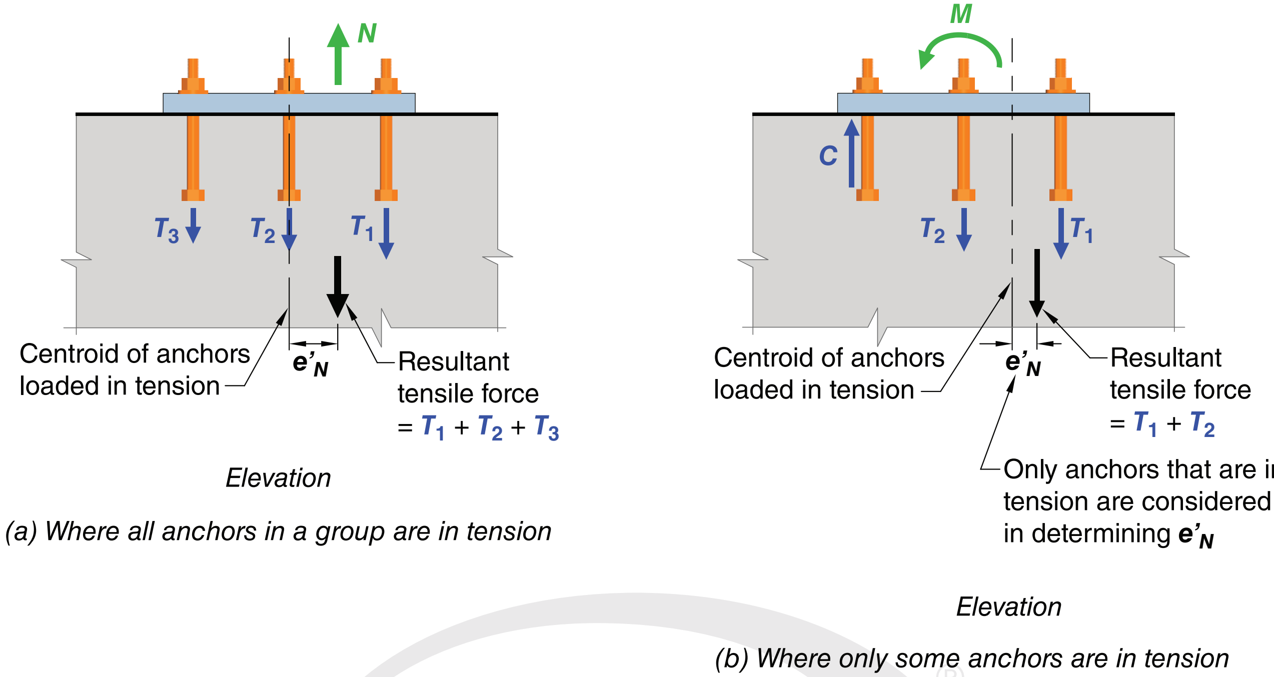

Fig. R17.6.2.3.1—Definition of e'_N *for an anchor group.*

17.6.2.3.2 If the loading on an anchor group is such that only some of the anchors in the group are in tension, only those anchors that are in tension shall be considered for determining eccentricity e'_N in Eq. (17.6.2.3.1) and for the calculation of N_{cbg} according to Eq. (17.6.2.1b).

17.6.2.3.3 If the loading is eccentric with respect to two orthogonal axes, $\psi_{ec,N}$ shall be calculated for each axis individually, and the product of these factors shall be used as $\psi_{ec,N}$ in Eq. (17.6.2.1b).

17.6.2.4 *Breakout edge effect factor,* $\psi_{ed,N}$

17.6.2.4.1 Modification factor for edge effects for single anchors or anchor groups loaded in tension, $\psi_{ed,N}$, shall be determined by (a) or (b).

(a) If $c_{a,min} \geq 1.5h_{ef}$, then

$$\psi_{ed,N} = 1.0 \qquad (17.6.2.4.1a)$$

(b) If $c_{a,min} < 1.5h_{ef}$, then

$$\psi_{edN} = 0.7 + 0.3\frac{c_{a,min}}{1.5\,h_{ef}} \qquad (17.6.2.4.1b)$$

R17.6.2.4 *Breakout edge effect factor,* $\psi_{ed,N}$

R17.6.2.4.1 If anchors are located close to an edge such that there is insufficient space for a complete breakout volume to develop, the strength of the anchor is further reduced beyond that reflected in A_{Nc}/A_{Nco}. If the smallest side cover distance is at least $1.5h_{ef}$, the design model assumes a complete breakout volume 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 (Fuchs et al. 1995).

17.6.2.5 *Breakout cracking factor,* $\psi_{c,N}$

17.6.2.5.1 Modification factor for the influence of cracking in anchor regions at service load levels, $\psi_{c,N}$, shall be determined by (a) or (b):

R17.6.2.5 *Breakout cracking factor,* $\psi_{c,N}$

R17.6.2.5.1 Post-installed anchors that do not meet the requirements for use in cracked concrete according to ACI CODE-355.2 or ACI CODE-355.4 should be used only in regions that will remain uncracked. The analysis for the deter-

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(a) For anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, $\psi_{c,N}$ shall be permitted to be: $\psi_{c,N}$ = 1.25 for cast-in anchors $\psi_{c,N}$ = 1.4 for post-installed anchors, if the value of k_c used in Eq. (17.6.2.2.1) is 17. If the value of k_c used in Eq. (17.6.2.2.1) is taken from the ACI CODE-355.2 or ACI CODE-355.4 product evaluation report for post-installed anchors:

(i) $\psi_{c,N}$ shall be based on the ACI CODE-355.2 or ACI CODE-355.4 product evaluation report for anchors qualified for use in both cracked and uncracked concrete

(ii) $\psi_{c,N}$ shall be taken as 1.0 for anchors qualified for use in uncracked concrete.

(b) For anchors located in a region of a concrete member where 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, and 17.6.2.6 shall be satisfied.

17.6.2.5.2 Post-installed anchors shall be qualified for use in cracked concrete in accordance with ACI CODE-355.2 or ACI CODE-355.4. Cracking in the concrete shall be controlled by flexural reinforcement distributed in accordance with 24.3.2, or equivalent crack control shall be provided by confining reinforcement.

17.6.2.6 *Breakout splitting factor,* $\psi_{cp,N}$

17.6.2.6.1 Modification factor for post-installed anchors designed for uncracked concrete in accordance with 17.6.2.5 without supplementary reinforcement to control splitting, $\psi_{cp,N}$, shall be determined by (a) or (b) using the critical distance c_{ac} as defined in 17.9.5.

(a) If $c_{a,min} \geq c_{ac}$, then

$$\psi_{cp,N} = 1.0 \qquad (17.6.2.6.1a)$$

(b) If $c_{a,min} < c_{ac}$, then

$$\psi_{cp,N} = \frac{c_{a,min}}{c_{ac}} \geq \frac{1.5h_{ef}}{c_{ac}} \qquad (17.6.2.6.1b)$$

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

17.6.2.7 *Breakout compression field factor,* $\psi_{cm,N}$

17.6.2.7.1 Modification factor for breakout compression field effect, $\psi_{cm,N}$, to be applied to all tension-loaded anchors as part of a tension-compression couple where the tension-loaded anchors are located $1.5h_{ef}$ or farther from any free

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mination of crack formation should include the effects of restrained shrinkage (refer to 24.4.2). The anchor qualification tests of ACI CODE-355.2 or ACI CODE-355.4 require that anchors in cracked concrete zones perform well in a crack that is 0.012-in. wide. If wider cracks are expected, reinforcement to control the crack width to approximately 0.012 in. should be provided. Refer to ACI PRC-224 for more information.

The concrete breakout strengths given by Eq. (17.6.2.2.1) and (17.6.2.2.3) assume cracked concrete ($\psi_{c,N}$ = 1.0) with $\psi_{c,N}k_c$ = 24 for cast-in anchors and 17 for post-installed anchors. If the uncracked concrete $\psi_{c,N}$ factors are applied (1.25 for cast-in and 1.4 for post-installed), $\psi_{c,N}k_c$ factors become 30 for cast-in anchors and 24 for post-installed anchors. This agrees with field observations and tests demonstrating cast-in anchor strength exceeds that of post-installed for both cracked and uncracked concrete.

R17.6.2.6 *Breakout splitting factor,* $\psi_{cp,N}$

R17.6.2.6.1 The design provisions in 17.6 are based on the assumption that the basic concrete breakout strength can be achieved if the minimum edge distance $c_{a,min}$ equals $1.5h_{ef}$. Test results (Asmus 1999), however, indicate that many torque-controlled and displacement-controlled expansion anchors and some undercut anchors require edge distances exceeding $1.5h_{ef}$ to achieve the basic concrete breakout strength if tested in uncracked concrete without supplementary reinforcement to control splitting. When a tensile 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 given in 17.6.2.1. To account for this potential splitting mode of failure, the basic concrete breakout strength is reduced by a factor $\psi_{cp,N}$ if $c_{a,min}$ is less than the critical edge distance c_{ac}.

R17.6.2.6.2 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, the reduction factor $\psi_{cp,N}$ is taken as 1.0.

R17.6.2.7 *Breakout compression field factor,* $\psi_{cm,N}$

R17.6.2.7.1 For grouted baseplates or baseplates in direct contact with the concrete, where the internal lever arm, z, of the tension-compression couple resulting from a moment on an anchor plate is sufficiently small relative to the anchor embedment depth, the compression field developed in the

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concrete edge and where the ratio of the resultant compression to tension forces is greater than 0.8 shall be calculated by:

$$\psi_{cm,N} = 2 - \frac{z}{1.5h_{ef}} \geq 1.0 \qquad (17.6.2.7.1)$$

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concrete inhibits formation of the tension breakout cone associated with the tension-loaded anchor(s) as shown in Fig. R17.6.2.7.1. This effect is accounted for with $\psi_{cm,N}$ (Eligehausen et al. 2006b). The effect of the compression field on the breakout strength of the tension-loaded anchors is neglected in cases where: a) the tension and compression resultants are separated by more than $1.5h_{ef}$; b) the ratio between the resultant compression and tension forces acting on the group is reduced, for example, by uplift on the connection; or c) the breakout strength is influenced by concrete edges. Determination of the compression resultant location and the value of z for a given combination of applied moments and axial force should correspond to a reasonable engineering model.

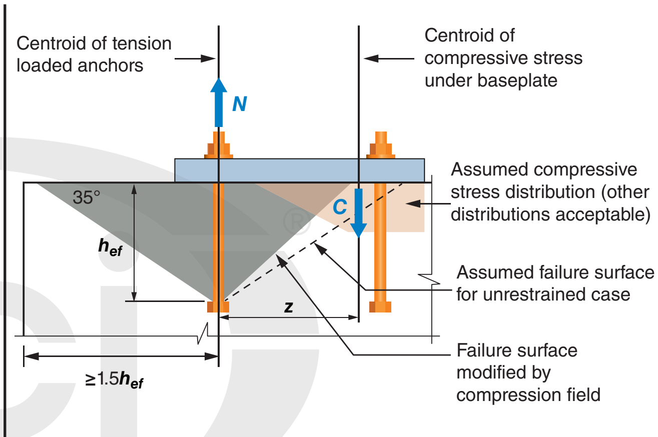

Fig. R17.6.2.7.1—Example of anchor group subjected to an overturning moment.

17.6.2.7.2 For all other cases, $\psi_{cm,N}$ shall be taken as 1.0.

R17.6.2.7.2 In all cases, it is conservative to take the value of $\psi_{cm,N} = 1.0$.

17.6.3 *Pullout strength of a single cast-in anchor or a single post-installed expansion, screw, or undercut anchor in tension,* N_{pn}

17.6.3.1 Nominal pullout strength of a single cast-in anchor or a single-post-installed expansion, screw, or undercut anchor in tension, N_{pn}, shall be calculated by:

$$N_{pn} = \psi_a \psi_{c,P} N_p \qquad (17.6.3.1)$$

where $\psi_{c,P}$ is given in 17.6.3.3.

R17.6.3 *Pullout strength of a single cast-in anchor or a single post-installed expansion, screw, or undercut anchor in tension,* N_{pn}

R17.6.3.1 The design requirements for pullout are applicable to cast-in anchors and post-installed expansion, screw, and undercut anchors. They are not applicable to adhesive anchors, which are instead evaluated for bond failure in accordance with 17.6.5.

17.6.3.2 *Basic single anchor pullout strength,* N_p

17.6.3.2.1 For post-installed expansion, screw, 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 CODE-355.2. It is not permissible to calculate the pullout strength in tension for such anchors.

R17.6.3.2 *Basic single anchor pullout strength,* N_p

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17.6.3.2.2 For single anchors, it shall be permitted to evaluate the pullout strength in tension, N_p, for use in Eq. (17.6.3.1) in accordance with (a) or (b). Alternatively, it shall be permitted to use values of N_p based on the 5% fractile of tests performed and evaluated in the same manner as the ACI CODE-355.2 procedures but without the benefit of friction.

(a) For cast-in headed studs and headed bolts, N_p shall be calculated by:

$$N_p = 8A_{brg}f_c' \qquad (17.6.3.2.2a)$$

(b) For J- or L-bolts, N_p shall be calculated by:

$$N_p = 0.9f_c'e_hd_a \qquad (17.6.3.2.2b)$$

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

17.6.3.3 *Pullout cracking factor,* $\psi_{c,P}$

17.6.3.3.1 Modification factor to account for the influence of cracking in anchor regions at service load levels, $\psi_{c,P}$, shall be determined by (a) or (b):

(a) For anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, $\psi_{c,P}$ shall be permitted to be 1.4.
(b) For anchors located in a region of a concrete member where analysis indicates cracking at service load levels, $\psi_{c,P}$ shall be taken as 1.0.

17.6.4 *Concrete side-face blowout strength of headed anchors in tension,* N_{sb}

17.6.4.1 For a single headed anchor with deep embedment close to an edge ($h_{ef} > 2.5c_{a1}$), the nominal side-face blowout strength, N_{sb}, shall be calculated by:

$$N_{sb} = \psi_a 160 c_{a1}\sqrt{A_{brg}}\lambda_a\sqrt{f_c'} \qquad (17.6.4.1)$$

17.6.4.1.1 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 $1.0 \leq c_{a2}/c_{a1} \leq 3.0$.

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R17.6.3.2.2 The pullout strength equations given in 17.6.3.2.2(a) and 17.6.3.2.2(b) are only applicable to cast-in headed and hooked anchors (Kuhn and Shaikh 1996; *fib* 2011); they are not applicable to post-installed expansion, screw, and undercut anchors that use various mechanisms for end anchorage unless the validity of the pullout strength equations is verified by tests.

The value calculated from Eq. (17.6.3.2.2a) corresponds to the force at which crushing of the concrete occurs due to bearing of the anchor head (*fib* 2011; ACI CODE-349). It is not the force required to pull the anchor completely out of the concrete; therefore, the equation does not contain a term relating to embedment depth. Local crushing of the concrete greatly reduces the stiffness of the connection, and generally will be the beginning of a pullout failure. The pullout strength in tension of headed studs or headed bolts can be increased by providing reinforcement, such as closely spaced spirals, throughout the head region. This increase can be demonstrated by tests, as required by the Licensed Design Professional for the specific application.

Equation (17.6.3.2.2b) for hooked bolts was developed by Lutz (1995) based on the results of Kuhn and Shaikh (1996). 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 a pullout failure. The limits on $e_h$are based on the range of variables used in the three test programs reported in Kuhn and Shaikh (1996).

R17.6.4 *Concrete side-face blowout strength of headed anchors in tension,* N_{sb}

R17.6.4.1 The design requirements for side-face blowout are based on the recommendations of Furche and Eligehausen (1991) and are applicable to headed anchors that usually are cast-in. Splitting during installation rather than side-face blowout generally governs post-installed anchors and is evaluated by ACI CODE-355.2 requirements.

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17.6.4.2 For multiple headed anchors with deep embedment close to an edge ($h_{ef} > 2.5c_{a1}$) and anchor spacing less than $6c_{a1}$, the nominal strength of those anchors susceptible to a side-face blowout failure, N_{sbg}, shall be calculated by:

$$N_{sbg} = \left(1 + \frac{s}{6c_{a1}}\right)N_{sb} \qquad (17.6.4.2)$$

where s is the distance between the outer anchors along the edge, and N_{sb} is obtained from Eq. (17.6.4.1) without modification for a perpendicular edge distance.

17.6.5 *Bond strength of adhesive anchors in tension,* N_a *or* N_{ag}

17.6.5.1 Nominal bond strength in tension, N_a of a single adhesive anchor or N_{ag} of an adhesive anchor group satisfying 17.5.1.4.1, shall be calculated by (a) or (b), respectively.

(a) For a single adhesive anchor:

$$N_a = \frac{A_{Na}}{A_{Nao}}\psi_a \psi_{ed,Na} \psi_{cp,Na} N_{ba} \qquad (17.6.5.1a)$$

(b) For an adhesive anchor group:

$$N_{ag} = \frac{A_{Na}}{A_{Nao}}\psi_a \psi_{ec,Na} \psi_{ed,Na} \psi_{cp,Na} N_{ba} \qquad (17.6.5.1b)$$

where ψ_a, $\psi_{ec,Na}$, $\psi_{ed,Na}$, and $\psi_{cp,Na}$ are given in 17.5.4.1, 17.6.5.3, 17.6.5.4, and 17.6.5.5, respectively.

COMMENTARY

R17.6.4.2 To calculate nominal side-face blowout strength for multiple headed anchors, only those anchors close to an edge ($c_{a1} < 0.4h_{ef}$) that are loaded in tension should be considered. Their strength is compared to the portion of the tensile load applied to those anchors.

R17.6.5 *Bond strength of adhesive anchors in tension,* N_a *or* N_{ag}

R17.6.5.1 Evaluation of bond strength applies only to adhesive anchors. Single anchors with small embedment loaded to failure in tension may exhibit concrete breakout failures, while deeper embedments produce bond failures. Adhesive anchors that exhibit bond failures when loaded individually may exhibit concrete failures in a group or in a near-edge condition. In all cases, the strength in tension of adhesive anchors is limited by concrete breakout strength as given by Eq. (17.6.2.1a) and (17.6.2.1b) (Eligehausen et al. 2006a).

The influence of anchor spacing and edge distance on both bond strength and concrete breakout strength must be evaluated for adhesive anchors. The influence of anchor spacing and edge distance on the nominal bond strength of adhesive anchors in tension are included in the modification factors A_{Na}/A_{Nao} and $\psi_{ed,Na}$ in Eq. (17.6.5.1a) and (17.6.5.1b).

The influence of nearby edges and adjacent loaded anchors on bond strength is dependent on the volume of concrete mobilized by a single adhesive anchor. In contrast to the projected concrete failure area concept used in Eq. (17.6.2.1a) and (17.6.2.1b) to calculate the breakout strength of an adhesive anchor, the influence area associated with the bond strength of an adhesive anchor used in Eq. (17.6.5.1a) and (17.6.5.1b) is not a function of the embedment depth, but rather a function of the anchor diameter and characteristic bond stress. The critical distance c_{Na} is assumed the same whether the concrete is cracked or uncracked. For simplicity, the relationship for c_{Na} in Eq. (17.6.5.1.2b) uses τ_{uncr}, the characteristic bond stress in uncracked concrete. This has been verified by experimental and numerical studies (Eligehausen et al. 2006a). Figure R17.6.5.1(a) shows A_{Nao} and the development of Eq. (17.6.5.1.2a). A_{Nao} is the projected influence area for the bond strength of a single adhesive anchor. Figure R17.6.5.1(b) shows an example of the projected influence area for an anchor group. Because, in this case, A_{Na} is the projected influence area for an anchor group, and A_{Nao} is the projected influence area for a single anchor, there is no need to include n, the number of anchors, in Eq. (17.6.5.1b). If individual anchors in a group (anchors loaded by a common base plate or attachment) are positioned in such a way that the projected influence areas of the individual anchors overlap, the value of A_{Na} is less than nA_{Nao}.

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17.6.5.1.1 A_{Na} is the projected influence area of a single adhesive anchor or an adhesive anchor group that is approximated as a rectilinear area that projects outward a distance c_{Na} from the centerline of the adhesive anchor, or in the case of an adhesive anchor group, from a line through a row of adjacent adhesive anchors. A_{Na} shall not exceed nA_{Nao}, where n is the number of adhesive anchors in the group that resist tension.

17.6.5.1.2 A_{Nao} is the projected influence area of a single adhesive anchor with an edge distance of at least c_{Na}:

$$A_{Nao} = (2c_{Na})^2 \qquad (17.6.5.1.2a)$$

where

$$c_{Na} = 10d_a\sqrt{\frac{\tau_{uncr}}{1100}} \qquad (17.6.5.1.2b)$$

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The tensile strength of closely spaced adhesive anchors with low bond strength may significantly exceed the value given by Eq. (17.6.5.1b). A correction factor is given in the literature (Eligehausen et al. 2006a) to address this issue, but for simplicity, this factor is not included in the Code.

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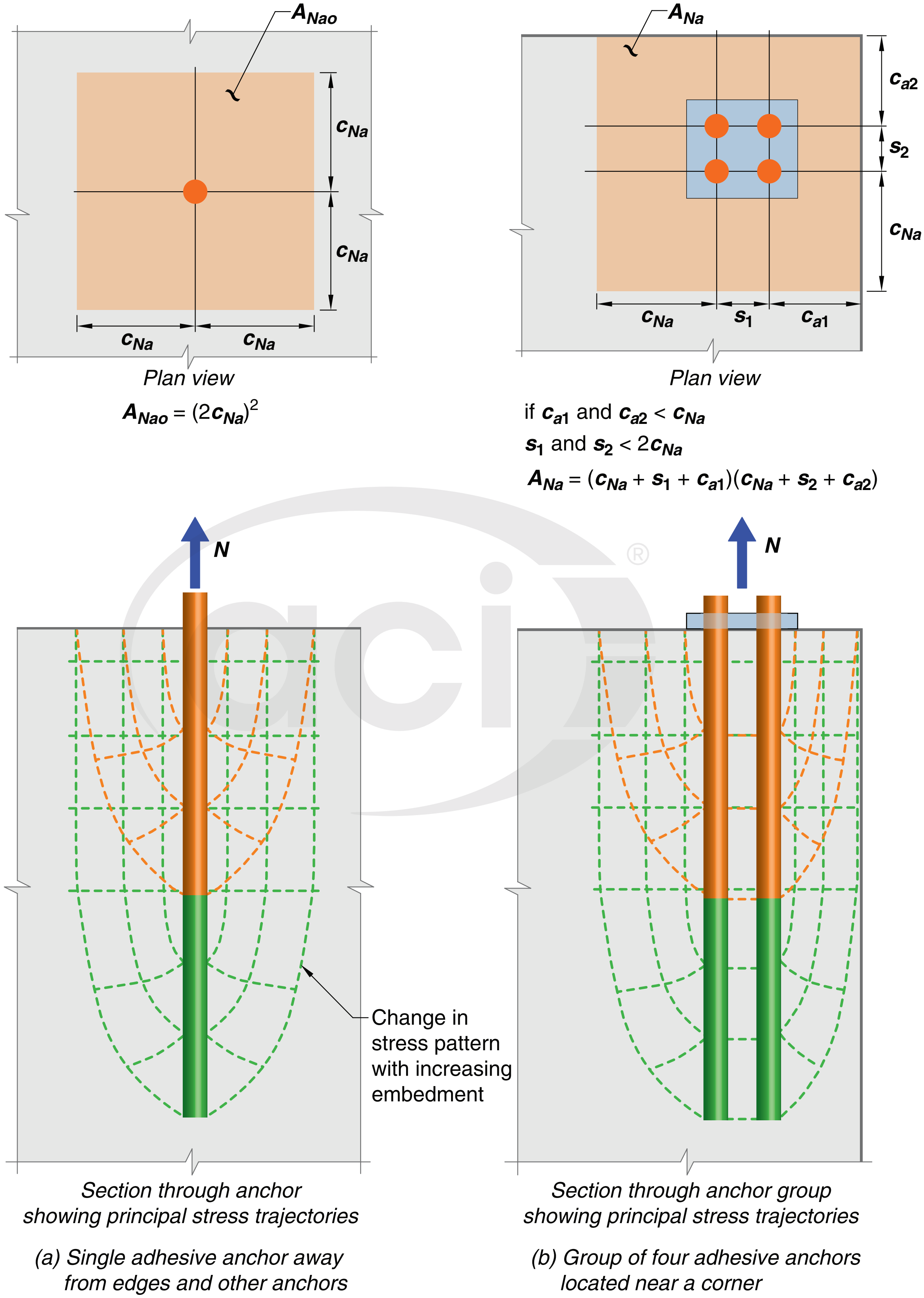

Fig. R17.6.5.1—Calculation of influence areas A_{Nao} *and* A_{Na}.

CODE

17.6.5.2 *Basic single anchor bond strength,* N_{ba}

17.6.5.2.1 Basic bond strength of a single adhesive anchor in tension in cracked concrete, N_{ba}, shall be calculated by Eq. (17.6.5.2.1)

$$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef} \tag{17.6.5.2.1}$$

17.6.5.2.2 Characteristic bond stress, τ_{cr}, shall be taken as the 5 percent fractile of results of tests performed and evaluated in accordance with ACI CODE-355.4.

17.6.5.2.3 If analysis indicates cracking at service load levels, adhesive anchors shall be qualified for use in cracked concrete in accordance with ACI CODE-355.4.

17.6.5.2.4 For adhesive anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, τ_{uncr} shall be permitted to be used in place of τ_{cr} in Eq. (17.6.5.2.1) and shall be taken as the 5 percent fractile of results of tests performed and evaluated according to ACI CODE-355.4.

17.6.5.2.5 It shall be permitted to use the minimum characteristic bond stress values in Table 17.6.5.2.5, provided (a) through (e) are satisfied:

(a) Anchors shall meet the requirements of ACI CODE-355.4
(b) Anchors shall be installed in holes drilled with a rotary impact drill or rock drill
(c) Concrete compressive strength at time of anchor installation shall be at least 2500 psi
(d) Concrete age at time of anchor installation shall be at least 21 days
(e) Concrete temperature at time of anchor installation shall be at least 50°F

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R17.6.5.2 *Basic single anchor bond strength,* N_{ba}

R17.6.5.2.1 The equation for basic bond strength of adhesive anchors as given in Eq. (17.6.5.2.1) represents a uniform bond stress model that has been shown to provide the best prediction of adhesive anchor bond strength based on numerical studies and comparisons of different models to an international database of experimental results (Cook et al. 1998). The basic bond strength is valid for bond failures that occur between the concrete and the adhesive as well as between the anchor and the adhesive.

R17.6.5.2.2 Characteristic bond stresses should be based on tests performed in accordance with ACI CODE-355.4 and should reflect the particular combination of installation and use conditions anticipated during construction and during anchor service life. If product-specific information is unavailable at the time of design, Table 17.6.5.2.5 provides lower-bound default values.

R17.6.5.2.5 The characteristic bond stresses in Table 17.6.5.2.5 are the minimum values permitted for adhesive anchor systems qualified in accordance with ACI CODE-355.4 for the tabulated installation and use conditions. Use of these values is restricted to the combinations of specific conditions listed; values for other combinations of installation and use conditions should not be inferred. If both sustained tension and earthquake-induced forces are required to be resisted by the anchors, the applicable factors given in the footnotes of Table 17.6.5.2.5 should be multiplied together. The table assumes a concrete age of at least 21 days and a concrete compressive strength of at least 2500 psi.

The terms "indoor" and "outdoor" as used in Table 17.6.5.2.5 refer to a specific set of installation and service environments. Indoor conditions represent anchors installed in dry concrete with a rotary impact drill or rock drill and subjected to limited concrete temperature variations over the service life of the anchor. Outdoor conditions are assumed to occur if, at the time of installation, the concrete is exposed to weather that might leave the concrete wet. Anchors installed in outdoor conditions are also assumed to be subject to greater concrete temperature variations such as might be associated with freezing and thawing or elevated temperatures resulting from direct sun exposure. While the indoor/

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Table 17.6.5.2.5—Minimum characteristic bond stresses[1][2]

Installation and service environment	Moisture content of concrete at time of anchor installation	Peak in-service temperature of concrete, °F	τ_{cr}, psi	τ_{uncr}, psi
Outdoor	Dry to fully saturated	175	200	650
Indoor	Dry	110	300	1000

[1]If anchor design includes sustained tension, multiply values of τ_{cr} and τ_{uncr} by 0.4.
[2]If anchor design includes earthquake-induced forces for structures assigned to SDC C, D, E, or F, multiply values of τ_{cr} by 0.8 and τ_{uncr} by 0.4.

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outdoor characterization is useful for many applications, there may be situations in which a literal interpretation of the terms “indoor” and “outdoor” do not apply. For example, anchors installed before the building envelope is completed may involve drilling in saturated concrete. As such, the minimum characteristic bond stress associated with the outdoor condition in Table 17.6.5.2.5 applies, regardless of whether the service environment is “indoor” or “outdoor.”

Rotary impact drills and rock drills produce non-uniform hole geometries that are generally favorable for bond. Installation of adhesive anchors in core-drilled holes may result in substantially lower characteristic bond stresses. Because this effect is highly product dependent, design of anchors to be installed in core-drilled holes should adhere to the product-specific characteristic bond stresses established through testing in accordance with ACI CODE-355.4.

The characteristic bond stresses associated with specific adhesive anchor systems are dependent on a number of parameters. Consequently, care should be taken to include all parameters relevant to the value of characteristic bond stress used in the design. These parameters include but are not limited to:

(a) Type and duration of loading—bond strength is reduced for sustained tension

(b) Concrete cracking—bond strength is higher in uncracked concrete

(c) Anchor size—bond strength is generally inversely proportional to anchor diameter

(d) Drilling method—bond strength may be lower for anchors installed in core-drilled holes

(e) Degree of concrete saturation at time of hole drilling and anchor installation—bond strength may be reduced due to concrete saturation

(f) Concrete temperature at time of installation—installation of anchors in cold conditions may result in retarded adhesive cure and reduced bond strength

(g) Concrete age at time of installation—installation in early-age concrete may result in reduced bond strength (refer to R17.2.2)

(h) Peak concrete temperatures during anchor service life—under specific conditions (for example, anchors in thin concrete members exposed to direct sunlight), elevated concrete temperatures can result in reduced bond strength

(i) Chemical exposure—anchors used in industrial environments may be exposed to increased levels of contaminants that can reduce bond strength over time

Anchors tested and assessed under ACI CODE-355.4 may in some cases not be qualified for all of the installation and service environments represented in Table 17.6.5.2.5. Therefore, where the minimum values given in Table 17.6.5.2.5 are used for design, the relevant installation and service environments should be specified in accordance with 26.7.1(i), (j), (k), and (l), and only anchors that have been qualified under ACI CODE-355.4 for the installation and service environ-

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ments corresponding to the characteristic bond stress taken from Table 17.6.5.2.5 should be specified.

Characteristic bond stresses associated with qualified adhesive anchor systems for a specific set of installation and use conditions may substantially exceed the minimum values provided in Table 17.6.5.2.5. For example, 1/2-in. to 3/4-in. diameter anchors installed in impact-drilled holes in dry concrete where use is limited to indoor conditions in uncracked concrete as described above may exhibit characteristic bond stresses τ_{uncr} in the range of 2000 to 2500 psi.

17.6.5.3 *Bond eccentricity factor,* $\psi_{ec,Na}$

17.6.5.3.1 Modification factor for adhesive anchor groups loaded eccentrically in tension, $\psi_{ec,Na}$, shall be calculated by Eq (17.6.5.3.1).

$$\psi_{ec,Na} = \frac{1}{\left(1 + \frac{e'_N}{c_{Na}}\right)} \leq 1.0 \qquad (17.6.5.3.1)$$

17.6.5.3.2 If the loading on an adhesive anchor group is such that only some of the adhesive anchors are in tension, only those adhesive anchors that are in tension shall be considered for determining eccentricity e'_N in Eq. (17.6.5.3.1) and for the calculation of N_{ag} according to Eq. (17.6.5.1b).

17.6.5.3.3 If a load is eccentric about two orthogonal axes, $\psi_{ec,Na}$ shall be calculated for each axis individually, and the product of these factors shall be used as $\psi_{ec,Na}$ in Eq. (17.6.5.1b).

R17.6.5.3 *Bond eccentricity factor,* $\psi_{ec,Na}$

R17.6.5.3.1 Refer to R17.6.2.3.1.

17.6.5.4 *Bond edge effect factor,* $\psi_{ed,Na}$

17.6.5.4.1 Modification factor for edge effects for single adhesive anchors or adhesive anchor groups in tension, $\psi_{ec,Na}$, shall be determined by (a) or (b) using the critical distance c_{Na} as defined in Eq. (17.6.5.1.2b).

(a) If $c_{a,min} \geq c_{Na}$, then

$$\psi_{ed,Na} = 1.0 \qquad (17.6.5.4.1a)$$

(b) If $c_{a,min} < c_{Na}$, then

$$\psi_{ed,Na} = 0.7 + 0.3\frac{c_{a,min}}{c_{Na}} \qquad (17.6.5.4.1b)$$

R17.6.5.4 *Bond edge effect factor,* $\psi_{ed,Na}$

R17.6.5.4.1 If anchors are located close to an edge, their strength is further reduced beyond that reflected in A_{Na}/A_{Nao}. The factor $\psi_{ed,Na}$ accounts for the edge effect (Fuchs et al. 1995; Eligehausen et al. 2006a).

17.6.5.5 *Bond splitting factor,* $\psi_{cp,Na}$

17.6.5.5.1 Modification factor for adhesive anchors designed for uncracked concrete in accordance with 17.6.5.1 without supplementary reinforcement to control splitting, $\psi_{ec,Na}$, shall be determined by (a) or (b) where c_{ac} is defined in 17.9.5

(a) If $c_{a,min} \geq c_{ac}$, then

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$$\psi_{cp,Na} = 1.0 \qquad (17.6.5.5.1a)$$

(b) If $c_{a,min} < c_{ac}$, then

$$\psi_{ed,Na} = \frac{c_{a,min}}{c_{ac}} \geq \frac{c_{Na}}{c_{ac}} \qquad (17.6.5.5.1b)$$

17.6.5.5.2 For all other cases, $\psi_{cp,Na}$ shall be taken as 1.0.

17.7—Shear strength

17.7.1 *Steel strength of anchors in shear,* V_{sa}

17.7.1.1 Nominal steel strength of anchors in shear as governed by the steel, V_{sa}, shall be evaluated based on the properties of the anchor material and the physical dimensions of the anchors. If concrete breakout is a potential failure mode, the required steel shear strength shall be consistent with the assumed breakout surface.

17.7.1.2 Nominal strength of an anchor in shear, V_{sa}, shall not exceed (a) through (c):

(a) For cast-in headed stud anchor

$$V_{sa} = A_{se,V} f_{uta} \qquad (17.7.1.2a)$$

where $A_{se,V}$ is the effective cross-sectional area of an anchor in shear, in.^{2}, and f_{uta} used for calculations shall not exceed either $1.9f_{ya}$ or 125,000 psi.

(b) For cast-in headed bolt and hooked bolt anchors and for post-installed adhesive anchors

$$V_{sa} = 0.6A_{se,V} f_{uta} \qquad (17.7.1.2b)$$

where $A_{se,V}$ is the effective cross-sectional area of an anchor in shear, in.^{2}, and the value of f_{uta} shall not exceed either $1.9f_{ya}$ or 125,000 psi.

(c) For post-installed mechanical anchors, V_{sa} shall be based on the 5 percent fractile of results of tests performed and evaluated in accordance with ACI CODE-355.2. It is not permissible to calculate the steel strength in shear for such anchors.

17.7.1.2.1 If anchors are used with built-up grout pads, nominal strength V_{sa} calculated in accordance with 17.7.1.2 shall be multiplied by 0.80.

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R17.7—Shear strength

R17.7.1 *Steel strength of anchors in shear,* V_{sa}

R17.7.1.1 The shear applied to each anchor in an anchor group may vary depending on assumptions for the concrete breakout surface and load redistribution (refer to R17.7.2.1).

R17.7.1.2 The nominal shear strength of anchors is best represented as a function of f_{uta} rather than f_{ya} because the large majority of anchor materials do not exhibit a well-defined yield point. Welded studs develop a higher steel shear strength than headed anchors due to the fixity provided by the weld between the studs and the base plate. The use of Eq. (17.7.1.2a) and (17.7.1.2b) with the load factors of 5.3 and the ϕ-factors of 17.5.3 result in design strengths consistent with ANSI/AISC 360.

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 R17.6.1.2.

For post-installed adhesive anchors having a reduced cross-sectional area anywhere along the anchor length, the effective cross-sectional area of the anchor should be provided by the manufacturer. For threaded rods and headed bolts, ASME B1.1 defines $A_{se,V}$ as

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

where n_t is the number of threads per inch.

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17.7.2 *Concrete breakout strength of anchors in shear,* V_{cb}

17.7.2.1 Nominal concrete breakout strength in shear, V_{cb} of a single anchor or V_{cbg} of an anchor group satisfying 17.5.1.4.1, shall be calculated in accordance with (a) through (d):

(a) For shear perpendicular to the edge on a single anchor

$$V_{cb} = \frac{A_{Vc}}{A_{Vco}} \psi_a \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b \qquad (17.7.2.1a)$$

(b) For shear perpendicular to the edge on an anchor group

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}} \psi_a \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b \qquad (17.7.2.1b)$$

(c) For shear parallel to an edge, V_{cb} or V_{cbg} shall be permitted to be twice the value of the shear calculated by Eq. (17.7.2.1a) or (17.7.2.1b), respectively, with the shear assumed to act perpendicular to the edge and $\psi_{ed,V}$ taken equal to 1.0.

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

where ψ_a, $\psi_{ec,V}$, $\psi_{ed,V}$, $\psi_{c,V}$, and $\psi_{h,V}$ are given in 17.5.4.1, 17.7.2.3, 17.7.2.4, 17.7.2.5, and 17.7.2.6, respectively.

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R17.7.2 *Concrete breakout strength of anchors in shear,* V_{cb}

R17.7.2.1 The shear strength equations were developed from the CCD Method (refer to R17.5.1.4). They assume a breakout angle of approximately 35 degrees (refer to Fig. R17.5.1.4b) 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 of A_{Vc}/A_{Vco} in Eq. (17.7.2.1a) and (17.7.2.1b), and $\psi_{ec,V}$ in Eq. (17.7.2.1b). For anchors far from the edge, 17.7.2 usually will not govern. For these cases, 17.7.1 and 17.7.3 often govern.

Figure R17.7.2.1a shows A_{Vco} and the development of Eq. (17.7.2.1.3). A_{Vco} is the maximum projected area for a single anchor that approximates the surface area of the full breakout volume for an anchor unaffected by edge distance, spacing, or depth of member. Figure R17.7.2.1b shows examples of the projected areas for various single-anchor and multiple-anchor arrangements. A_{Vc} approximates the full surface area of the breakout for the particular arrangement of anchors. Because A_{Vc} is the total projected area for an anchor group, and A_{Vco} is the area for a single anchor, there is no need to include the number of anchors in the equation.

As shown in the examples in Fig. R17.7.2.1b of two-anchor groups loaded in shear, when using Eq. (17.7.2.1b) for cases where the anchor spacing s is greater than the edge distance to the near-edge anchor $c_{a1,1}$, both assumptions for load distribution illustrated in Cases 1 and 2 should be considered. This is because the anchors nearest to the free edge could fail first or the entire group could fail as a unit with the failure surface originating from the anchors farthest from the edge. For Case 1, the steel shear strength is provided by both anchors. For Case 2, the steel shear strength is provided entirely by the anchor farthest from the edge; no contribution of the anchor near the edge is considered. In addition, checking the near-edge anchor for concrete breakout under service loads is advisable to preclude undesirable cracking at service conditions. If the anchor spacing s is less than the edge distance to the near-edge anchor, the failure surfaces may merge (Eligehausen et al. 2006b) and Case 3 of Fig. R17.7.2.1b may be taken as a conservative approach.

If the anchors are welded to a common plate (regardless of anchor spacing s), when the anchor nearest the front edge begins to form a breakout failure, shear is transferred to the stiffer and stronger rear anchor. For this reason, only Case 2 need be considered, which is consistent with Section 6.5.5 of the *PCI Design Handbook* (PCI MNL 120). For determination of steel shear strength, it is conservative to consider only the anchor farthest from the edge. However, for anchors having a ratio of $s/c_{a1,1}$ less than 0.6, both the front and rear anchors may be assumed to resist the shear (Anderson and Meinheit 2007). For ratios of $s/c_{a1,1}$ greater than 1, it is advisable to check concrete breakout of the near-edge anchor to preclude undesirable cracking at service conditions.

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Further discussion of design for multiple anchors is given in Primavera et al. (1997).

For anchors near a corner required to resist a shear force with components normal to each edge, a satisfactory solution is to check the connection independently 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 Eligehausen et al. (2006a).

The detailed provisions of 17.7.2.1(a) apply to the case of shear directed toward an edge. If the shear is directed away from the edge, the strength will usually be governed by 17.7.1 or 17.7.3. The case of shear parallel to an edge is shown in Fig. R17.7.2.1c. The maximum shear that can be applied parallel to the edge, $V_{||}$, as governed by concrete breakout, is twice the maximum shear that can be applied perpendicular to the edge, $V_\perp$. For a single anchor required to resist shear near a corner (refer to Fig. R17.7.2.1d), the provisions for shear applied perpendicular to the edge should be checked in addition to the provisions for shear applied parallel to the edge.

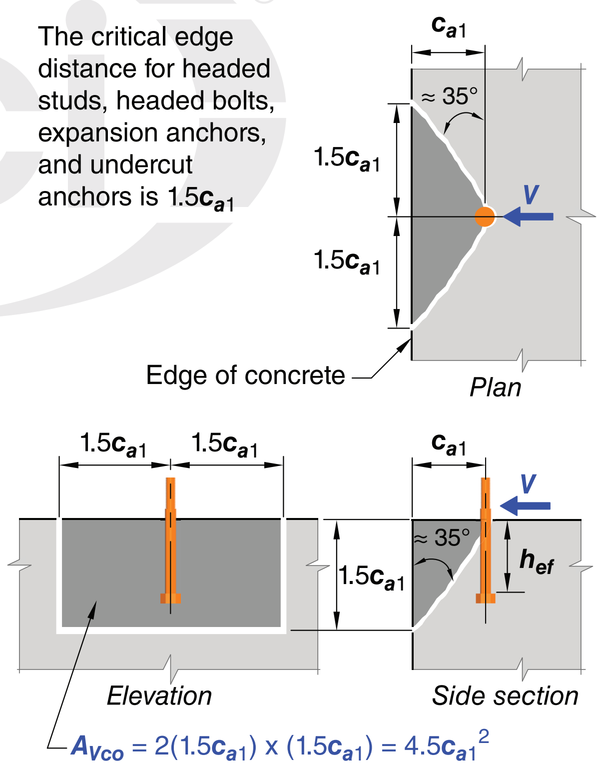

Fig. R17.7.2.1a—Calculation of A_{Vco}.

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COMMENTARY

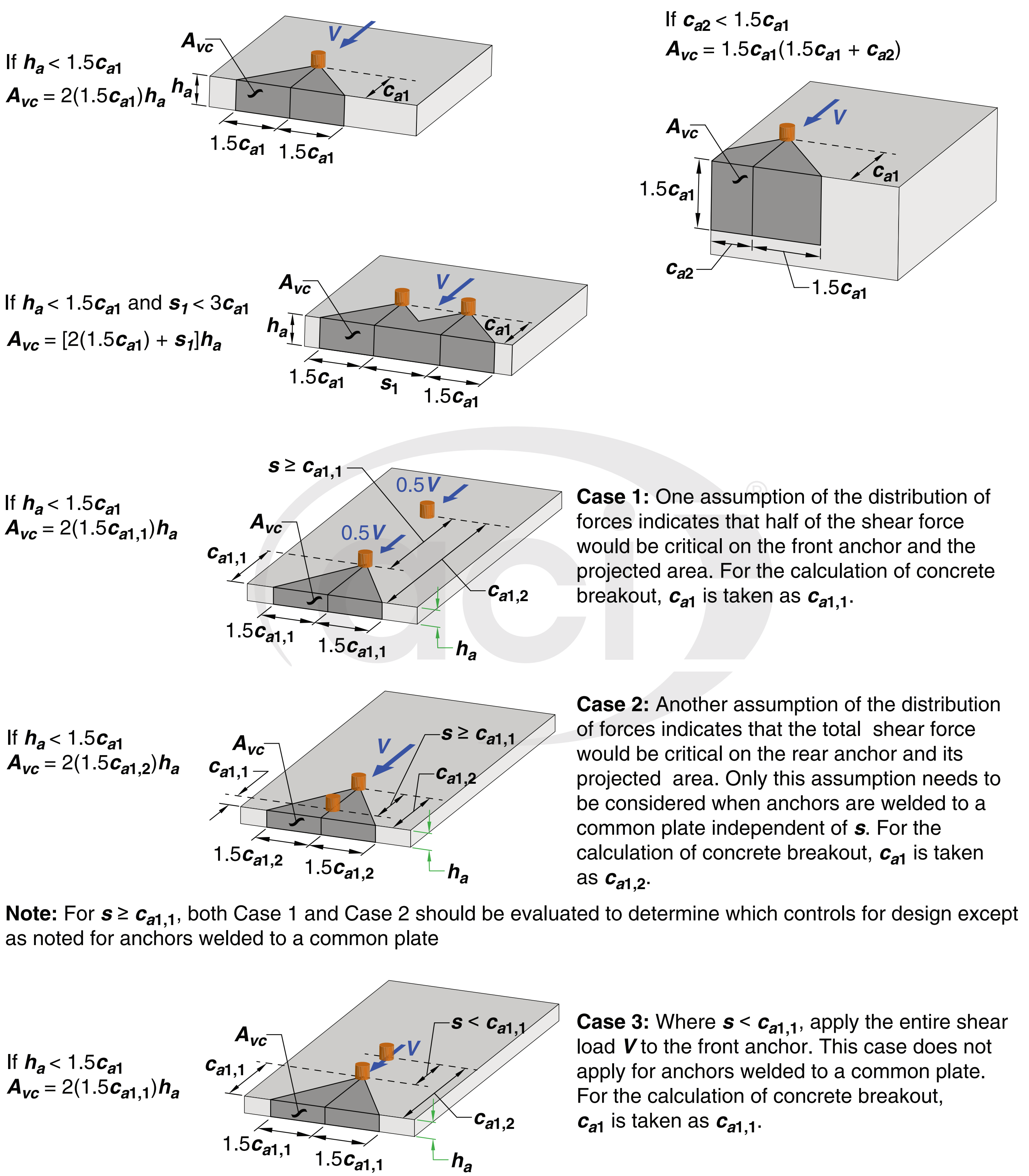

Fig. R17.7.2.1b—Calculation of A_{vc} for single anchors and anchor groups.

CODE

17.7.2.1.1 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 an anchor group. 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

COMMENTARY

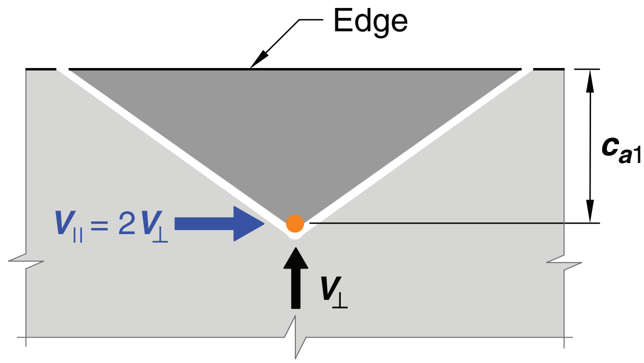

Fig. R17.7.2.1c—Shear force parallel to an edge.

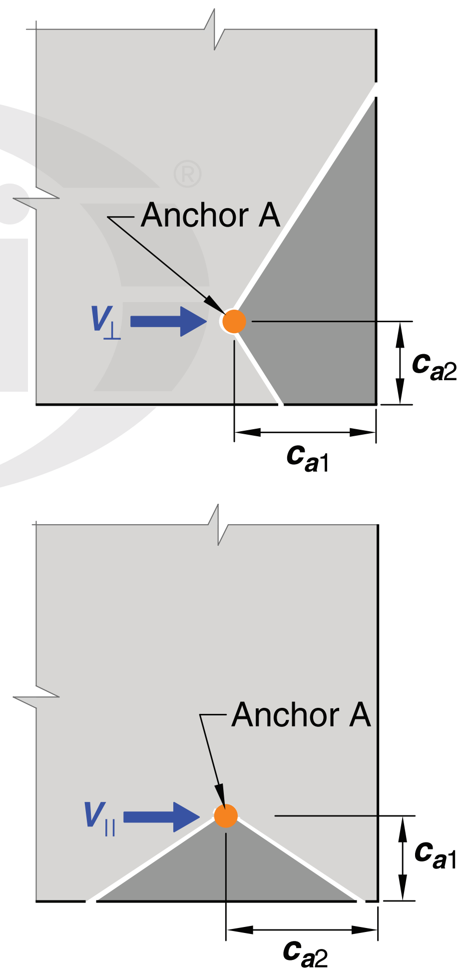

Fig. R17.7.2.1d—Shear near a corner.

CODE

this axis. A_{Vc} shall not exceed nA_{Vco}, where n is the number of anchors in the group.

17.7.2.1.2 If anchors are located in narrow sections of limited thickness such that both edge distances c_{a2} and thickness h_a are less than $1.5c_{a1}$, the value of c_{a1} used to calculate A_{Vc} in accordance with 17.7.2.1.1 as well as for the equations in 17.7.2.1 through 17.7.2.6 shall not exceed the greatest of (a) through (c).

(a) $c_{a2}/1.5$, where c_{a2} is the greatest edge distance
(b) $h_a/1.5$
(c) $s/3$, where s is the maximum spacing perpendicular to direction of shear, between anchors within a group

COMMENTARY

R17.7.2.1.2 For anchors located in narrow sections of limited thickness where the edge distances perpendicular to the direction of load and the member thickness are less than $1.5c_{a1}$, the shear breakout strength calculated by the CCD Method (refer to R17.5.1.4) is overly conservative. These cases were studied for the Kappa Method (Eligehausen and Fuchs 1988), and the problem was pointed out by Lutz (1995). Similar to the approach used for concrete breakout strength in tension in 17.6.2.1.2, the concrete breakout strength in shear for this case is more accurately evaluated if the value of c_{a1} used in 17.7.2.1 through 17.7.2.6 and in the calculation of A_{Vc} is limited to the maximum of two-thirds of the greater of the two edge distances perpendicular to the direction of shear, two-thirds of the member thickness, and one-third of the maximum spacing between individual anchors within the group, measured perpendicular to the direction of shear. The limit on c_{a1} of at least one-third of the maximum spacing between anchors within the group prevents the use of a calculated strength based on individual breakout volumes for an anchor group configuration.

This approach is illustrated in Fig. R17.7.2.1.2. In this example, the limiting value of c_{a1} is denoted as c'_{a1} and is used to calculate A_{Vc}, A_{Vco}, $\psi_{ed,V}$, and $\psi_{h,V}$ as well as V_b (not shown). The requirement of 17.7.2.1.2 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. The value of c_{a1} used to calculate A_{Vc} and to be used in 17.7.2.1 through 17.7.2.6 is determined when (a) an outer boundary of the failure surface first intersects the concrete surface; or (b) the intersection of the breakout surface between individual anchors within the group first intersects the concrete surface. For the example shown in Fig. R17.7.2.1.2, point "A" shows the intersection of the assumed failure surface for limiting c_{a1} with the concrete surface.

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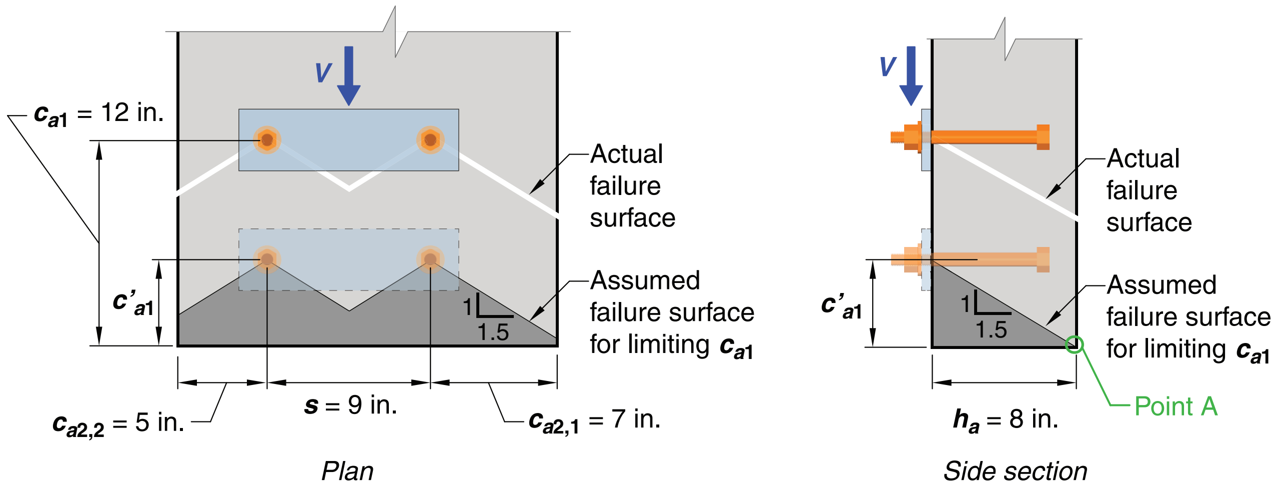

1. The actual c_{a1} = 12 in.
2. The two edge distances c_{a2} as well as h_a are all less than $1.5c_{a1}$.
3. The limiting value of c_{a1} (shown as c'_{a1} in the figure) to be used to calculate A_{Vc} and to be used in 17.7.2.1 through 17.7.2.6 is the largest of the following:

   $(c_{a2,max})/1.5 = (7)/1.5 = 4.67$ in.

   $(h_a)/1.5 = (8)/1.5 = 5.33$ in. (controls)

   $s/3 = 9/3 = 3$ in.
4. For this case, A_{Vc}, A_{Vco}, $\psi_{ed,V}$, and $\psi_{h,V}$ are:

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

   $A_{Vco} = 4.5(5.33)^2 = 128 \text{ in.}^2$

   $\psi_{ed,V} = 0.7 + 0.3(5)/5.33 = 0.98$

   $\psi_{h,V}$ = 1.0 because $c_{a1} = (h_a)/1.5$. Point A shows the intersection of the assumed failure surface with the concrete surface that establishes the limiting value of c_{a1}.

Fig. R17.7.2.1.2—Example of shear where anchors are located in narrow members of limited thickness.

17.7.2.1.3 A_{Vco} is the projected area for a single anchor in a deep member with a distance from edges of at least $1.5c_{a1}$ in the direction perpendicular to the shear. It shall be permitted to calculate A_{Vco} by Eq. (17.7.2.1.3), which gives the area of the base of a half-pyramid with a side length parallel to the edge of $3c_{a1}$ and a depth of $1.5c_{a1}$.

$$A_{Vco} = 4.5(c_{a1})^2 \qquad (17.7.2.1.3)$$

17.7.2.1.4 If 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 resisted by this critical anchor row alone.

CODE

17.7.2.2 *Basic single anchor breakout strength,* V_b

17.7.2.2.1 Basic concrete breakout strength of a single anchor in shear in cracked concrete, V_b, shall not exceed the lesser of (a) and (b):

(a) $$V_b = \left(7\left(\frac{\ell_e}{d_a}\right)^2\sqrt{d_a}\right)\lambda_a\sqrt{f_c'}(c_{a1})^{1.5} \qquad (17.7.2.2.1a)$$

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; $\ell_e = 2d_a$ for torque-controlled expansion anchors with a distance sleeve separated from expansion sleeve; $\ell_e \leq 8d_a$ in all cases.

(b) $$V_b = 9\lambda_a\sqrt{f_c'}\,(c_{a1})^{1.5} \qquad (17.7.2.2.1b)$$

17.7.2.2.2 For cast-in headed studs, headed bolts, or hooked bolts that are continuously welded to steel attachments, basic concrete breakout strength of a single anchor in shear in cracked concrete, V_b, shall be the lesser of Eq. (17.7.2.2.1b) and Eq. (17.7.2.2.2) provided that (a) through (d) are satisfied.

$$V_b = \left(8\left(\frac{\ell_e}{d_a}\right)^{0.2}\sqrt{d_a}\right)\lambda_a\sqrt{f_c'}(c_{a1})^{1.5} \qquad (17.7.2.2.2)$$

where ℓ_e is defined in 17.7.2.2.1.

(a) Steel attachment thickness is the greater of $0.5d_a$ and 3/8 in.
(b) Anchor spacing s is at least 2.5 in.
(c) Reinforcement is provided at the corners if $c_{a2} \leq 1.5h_{ef}$
(d) For anchor groups, the strength is calculated based on the strength of the row of anchors farthest from the edge.

17.7.2.3 *Breakout eccentricity factor,* $\psi_{ec,V}$

17.7.2.3.1 Modification factor for anchor groups loaded eccentrically in shear, $\psi_{ec,V}$, shall be calculated by Eq. (17.7.2.3.1).

$$\psi_{ec,V} = \frac{1}{\left(1 + \dfrac{e'_V}{1.5c_{a1}}\right)} \leq 1.0 \qquad (17.7.2.3.1)$$

COMMENTARY

R17.7.2.2 *Basic single anchor breakout strength,* V_b

R17.7.2.2.1 Like the concrete breakout tensile strength, the concrete breakout shear strength does not increase with the failure surface, which is proportional to $(c_{a1})^2$. Instead, the strength increases proportionally to $(c_{a1})^{1.5}$ due to the size effect. The constant, 7, in the shear strength equation (17.7.2.2.1a) was determined from test data reported in Fuchs et al. (1995) at the 5% fractile adjusted for cracking.

The strength is also influenced by the anchor stiffness and the anchor diameter (Fuchs et al. 1995; Eligehausen and Balogh 1995; Eligehausen et al. 1987, 2006b; Eligehausen and Fuchs 1988). The influence of anchor stiffness and diameter is not apparent in large-diameter anchors (Lee et al. 2010), resulting in a limitation on the shear breakout strength provided by Eq. (17.7.2.2.1b).

R17.7.2.2.2 For cast-in headed bolts continuously welded to an attachment, test data (Shaikh and Yi 1985) show that somewhat higher shear strength exists, possibly due to the stiff welded connection clamping the bolt more effectively than an attachment with an anchor gap. Because of this, the basic shear breakout strength for such anchors is increased, but the upper limit of Eq. (17.7.2.2.1b) is imposed because tests on large-diameter anchors welded to steel attachments are not available to justify a higher value than Eq. (17.7.2.2.1b). The design of supplementary reinforcement is discussed in *fib* (2011), Eligehausen et al. (1987, 2006b), and Eligehausen and Fuchs (1988).

R17.7.2.3 *Breakout eccentricity factor,* $\psi_{ec,V}$

R17.7.2.3.1 This section provides a modification factor for an eccentric shear toward an edge on an anchor group. If the shear originates above the plane of the concrete surface, the shear should first be resolved as a shear in the plane of the concrete surface, acting in combination with a moment that may or may not also cause tension in the anchors, depending on the normal force. Figure R17.7.2.3.1 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

COMMENTARY

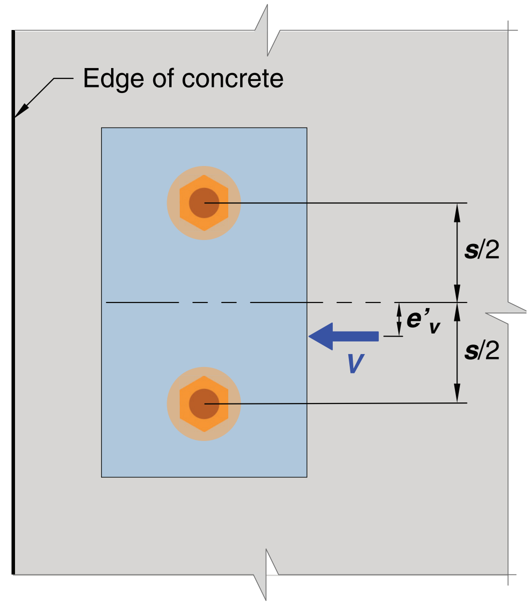

Fig. R17.7.2.3.1—Definition of e'V *for an anchor group.*

17.7.2.3.2 If the loading on an anchor group is such that only some of the anchors in the group are in shear, only those anchors that are in shear in the same direction shall be considered for determining the eccentricity e'_V in Eq. (17.7.2.3.1) and for the calculation of V_{cbg} according to Eq. (17.7.2.1b).

17.7.2.4 *Breakout edge effect factor,* $\psi_{ed,V}$

17.7.2.4.1 Modification factor for edge effects for single anchors or anchor groups loaded in shear, $\psi_{ed,V}$, shall be determined by (a) or (b) using the lesser value of c_{a2}.

(a) If $c_{a2} \geq 1.5c_{a1}$, then

$$\psi_{ed,V} = 1.0 \qquad (17.7.2.4.1a)$$

(b) If $c_{a2} < 1.5c_{a1}$, then

$$\psi_{ed,V} = 0.7 + 0.3\frac{c_{a2}}{1.5c_{a1}} \qquad (17.7.2.4.1b)$$

17.7.2.5 *Breakout cracking factor,* $\psi_{c,V}$

17.7.2.5.1 Modification factor for the influence of cracking in anchor regions at service load levels and presence or absence of supplementary reinforcement, $\psi_{c,V}$, shall be determined as follows:

(a) For anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, $\psi_{c,V}$ shall be permitted to be 1.4.

(b) For anchors located in a region of a concrete member where analysis indicates cracking at service load levels, $\psi_{c,V}$ shall be in accordance with Table 17.7.2.5.1.

CODE

Table 17.7.2.5.1—Modification factor where analysis indicates cracking at service load levels, $\psi_{c,V}$

Condition	$\psi_{c,V}$
Anchors without supplementary reinforcement or with edge reinforcement smaller than a No. 4 bar	1.0
Anchors with reinforcement of at least a No. 4 bar or greater between the anchor and the edge	1.2
Anchors with reinforcement of at least a No. 4 bar or greater between the anchor and the edge, and with the reinforcement enclosed within stirrups spaced at not more than 4 in.	1.4

17.7.2.6 *Breakout thickness factor,* $\psi_{h,V}$

17.7.2.6.1 Modification factor for anchors located in a concrete member where $h_a < 1.5c_{a1}$, $\psi_{h,V}$ shall be calculated by Eq. (17.7.2.6.1)

$$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0 \qquad (17.7.2.6.1)$$

17.7.3 *Concrete pryout strength of anchors in shear,* V_{cp} *or* V_{cpg}

17.7.3.1 Nominal pryout strength, V_{cp} of a single anchor or V_{cpg} of an anchor group satisfying 17.5.1.3.1, shall not exceed (a) or (b), respectively.

(a) For a single anchor

$$V_{cp} = k_{cp}N_{cp} \qquad (17.7.3.1a)$$

(b) For an anchor group

$$V_{cpg} = k_{cp}N_{cpg} \qquad (17.7.3.1b)$$

where

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

17.7.3.1.1 For cast-in anchors and post-installed expansion, screw, and undercut anchors, N_{cp} shall be taken as N_{cb} calculated by Eq. (17.6.2.1a), and for adhesive anchors, N_{cp} shall be the lesser of N_a calculated by Eq. (17.6.5.1a) and N_{cb} calculated by Eq. (17.6.2.1a).

17.7.3.1.2 For cast-in anchors and post-installed expansion, screw, and undercut anchors, N_{cpg} shall be taken as N_{cbg} calculated by Eq. (17.6.2.1b), and for adhesive anchors, N_{cpg} shall be the lesser of N_{ag} calculated by Eq. (17.6.5.1b) and N_{cbg} calculated by Eq. (17.6.2.1b).

COMMENTARY

R17.7.2.6 *Breakout thickness factor,* $\psi_{h,V}$

R17.7.2.6.1 For anchors located in a concrete member where $h_a < 1.5c_{a1}$, tests (*fib* 2011; Eligehausen et al. 2006b) have shown that the concrete breakout strength in shear is not directly proportional to the member thickness h_a. The factor $\psi_{h,V}$ accounts for this effect.

R17.7.3 *Concrete pryout strength of anchors in shear,* V_{cp} *or* V_{cpg}

R17.7.3.1 Fuchs et al. (1995) 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. Because it is possible that the bond strength of adhesive anchors could be less than the concrete breakout strength, it is necessary to consider both 17.6.2.1 and 17.6.5.1 to calculate pryout strength.

CODE

17.8—Tension and shear interaction

17.8.1 Unless tension and shear interaction effects are considered in accordance with 17.5.2.3, a single anchor resisting both tension and shear shall satisfy 17.8.2 and 17.8.4, and anchor groups shall satisfy 17.8.3 and 17.8.4. The value of ϕ shall correspond to the governing failure mode in accordance with 17.5.3.

17.8.2 *Single anchor concrete strength*

For a single anchor satisfying the strength requirements of Table 17.5.2, interaction between tension and shear shall satisfy Eq. (17.8.2).

$$\left(\frac{N_{ua}}{\phi N_{n,c}}\right)^{\frac{5}{3}} + \left(\frac{V_{ua}}{\phi V_{n,c}}\right)^{\frac{5}{3}} \leq 1 \qquad (17.8.2)$$

where $\phi N_{n,c}$ and $\phi V_{n,c}$ represent the minimum calculated strength in accordance with Table 17.5.2 as follows:

$\phi N_{n,c}$ = the least of ϕN_{cb}, ϕN_a, ϕN_{pn}, and ϕN_{sb}, lb

$\phi V_{n,c}$ = the lesser of ϕV_{cb} and ϕV_{cp}, lb

COMMENTARY

R17.8—Tension and shear interaction

R17.8.1 The tension-shear interaction expression has traditionally been expressed as

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

where ς varies from 1 to 2. The use of ς = 5/3 for concrete failure modes has been the basis of the Code for several editions. The use of ς = 2 for steel failure modes in bolts is in accordance with long-standing practice (Chesson et al. 1965). Interaction is checked for both concrete and steel failure modes independently as shown in Fig. R17.8.1. Previous versions of these provisions provided a trilinear expression for checking interaction, regardless of failure mode. This has been replaced by a less conservative approach.

Any other interaction expression that is verified by test data can be used to satisfy 17.5.2.3.

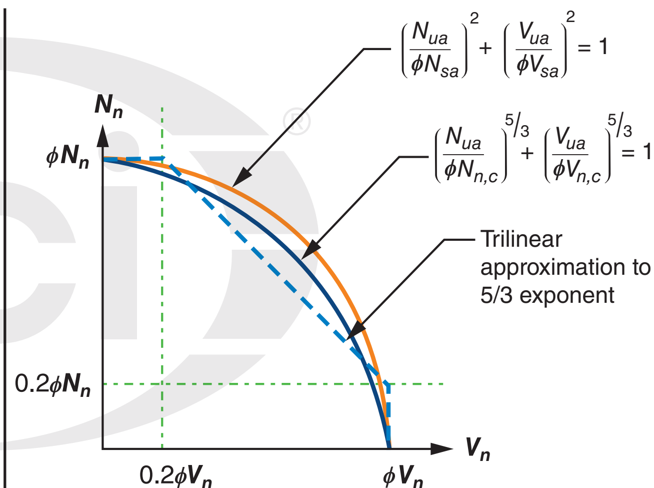

Fig. R17.8.1—Shear and tensile load interaction equation (single anchor case).

CODE

17.8.3 *Anchor group concrete strength*

For an anchor group satisfying the strength requirements of Table 17.5.2, interaction between tension and shear shall satisfy Eq. (17.8.3).

$$\left(\frac{N_{ua,g}}{\phi N_{ncg}}\right)^{\frac{5}{3}} + \left(\frac{V_{ua,g}}{\phi V_{n,cg}}\right)^{\frac{5}{3}} \leq 1 \qquad (17.8.3)$$

where $\phi N_{n,cg}$ and $\phi V_{n,cg}$ represent the minimum calculated strength in accordance with Table 17.5.2 as follows:

$\phi N_{n,cg}$ = the least of ϕN_{cbg}, ϕN_{ag}, and ϕN_{sbg}, lb

$\phi V_{n,cg}$ = the lesser of ϕV_{cbg} and ϕV_{cpg}, lb

17.8.4 *Single anchor or anchor group steel strength*

For a single anchor or an anchor group satisfying the strength requirements of Table 17.5.2, the most highly stressed anchor in tension and shear shall satisfy Eq. (17.8.4).

$$\left(\frac{N_{ua}}{\phi N_{sa}}\right)^{2} + \left(\frac{V_{ua}}{\phi V_{sa}}\right)^{2} \leq 1 \qquad (17.8.4)$$

17.9—Edge distances, spacings, and thicknesses to preclude splitting failure

17.9.1 Minimum spacings and edge distances for anchors and minimum thicknesses of members shall conform to this section, unless supplementary reinforcement is provided to control splitting. Lesser values from product-specific tests performed in accordance with ACI CODE-355.2 or ACI CODE-355.4 shall be permitted.

17.9.2 Unless determined in accordance with 17.9.3, minimum spacing parameters shall conform to Table 17.9.2(a).

COMMENTARY

R17.8.4 *Single anchor or anchor group steel strength*

The interaction check for steel is intended for the shaft of the anchor (anchor bolt, cast-in headed stud, threaded rod, screw). It is not intended for shear lugs.

R17.9—Edge distances, spacings, and thicknesses to preclude splitting failure

R17.9.1 Minimum spacings, edge distances, and thicknesses are dependent on the anchor characteristics. Installation forces and torques in post-installed anchors can cause splitting of the surrounding concrete. Such splitting also can be produced in subsequent torquing during connection of attachments to anchors including cast-in anchors. The primary source of values for minimum spacings, edge distances, and thicknesses of post-installed anchors should be the product-specific tests of ACI CODE-355.2 and ACI CODE-355.4. In some cases, however, specific products are not known in the design stage. Approximate values are provided for use in design.

R17.9.2 Edge cover for anchors with deep embedments can have a significant effect on the side face blowout strength provided in 17.6.4. It is therefore advantageous to increase edge cover beyond that required in 20.5.1.3 to increase side-face blowout strength.

Drilling holes for post-installed anchors can cause microcracking. The requirement for edge distance to be at least twice the nominal maximum aggregate size is to reduce effects of such microcracking.

CODE

Table 17.9.2(a)—Minimum spacing and edge distance requirements

Spacing parameter	Anchor type: Cast-in anchors — Not torqued	Anchor type: Cast-in anchors — Torqued	Anchor type: Post-installed — Adhesive, expansion, and undercut anchors	Anchor type: Post-installed — Screw anchors
Minimum anchor spacing	$4d_a$	$6d_a$	$6d_a$	Greater of $0.6h_{ef}$ and $6d_a$
Minimum edge distance	Specified cover requirements for reinforcement according to 20.5.1.3	$6d_a$	Greatest of (a), (b), and (c): (a) Specified cover requirements for reinforcement according to 20.5.1.3 (b) Twice the nominal maximum aggregate size (c) Minimum edge distance requirements according to ACI CODE-355.2 or ACI CODE-355.4, or Table 17.9.2(b) when product information is absent	

Table 17.9.2(b)—Minimum edge distance in absence of product-specific ACI CODE-355.2 or ACI CODE-355.4 test information

Post-installed anchor type	Minimum edge distance
Torque-controlled	$8d_a$
Displacement-controlled	$10d_a$
Screw	$6d_a$
Undercut	$6d_a$
Adhesive	$6d_a$

17.9.3 For anchors where installation does not produce a splitting force and that will not be torqued, if the edge distance or spacing is less than those given in 17.9.2, calculations shall be performed by substituting for d_a a lesser value d_a' that meets the requirements of 17.9.2. Calculated forces applied to the anchor shall be limited to the values corresponding to an anchor having a diameter of d_a'.

17.9.4 Value of h_{ef} for a post-installed expansion, screw, or undercut post-installed anchor shall not exceed the greater of two-thirds of the member thickness, h_a, and the member thickness minus 4 in., unless determined from tests in accordance with ACI CODE-355.2.

COMMENTARY

R17.9.3 In some cases, it may be desirable to use a larger-diameter anchor than the requirements of 17.9.2 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_a'.

R17.9.4 Splitting failures are caused by load transfer between the bolt and the concrete. The limitations on the value of h_{ef} do not apply to cast-in and adhesive anchors because the splitting forces associated with these anchor types are less than for expansion, screw, and undercut anchors.

For all post-installed anchors, the embedment depth for a given member thickness should be limited to avoid back-face blowout on the opposite side of the concrete member during hole drilling and anchor setting. This depth limit is dependent on many variables, including anchor type, drilling method, drilling technique, type and size of drilling equipment, presence of reinforcement, and strength and condition of the concrete.

CODE

17.9.5 Critical edge distance c_{ac} shall be in accordance with Table 17.9.5 unless determined from tension tests in accordance with ACI CODE-355.2 or ACI CODE-355.4.

Table 17.9.5—Critical edge distance

Post-installed anchor type	Critical edge distance c_{ac}
Torque-controlled	$4h_{ef}$
Displacement-controlled	$4h_{ef}$
Screw	$4h_{ef}$
Undercut	$2.5h_{ef}$
Adhesive	$2h_{ef}$

17.10—Earthquake-resistant anchor design requirements

17.10.1 Anchors in structures assigned to Seismic Design Category (SDC) C, D, E, or F shall satisfy the additional requirements of this section.

COMMENTARY

R17.9.5 The critical edge distance c_{ac} is required for design of post-installed anchors for use in uncracked concrete where no supplemental reinforcement is available to restrain splitting cracks. To permit the design of these types of anchors if product-specific information is not available, conservative default values for c_{ac} are provided. Alternately, product-specific values of c_{ac} may be determined in accordance with ACI CODE-355.2 or ACI CODE-355.4. Corner-test requirements in the aforementioned qualification standards may not be satisfied with $c_{a,min} = 1.5h_{ef}$ for many expansion, screw, undercut, and adhesive anchors due to tensile and flexural stresses associated with anchor installation and loading, which may result in a premature splitting failure.

R17.10—Earthquake-resistant anchor design requirements

R17.10.1 Unless 17.10.5.1 or 17.10.6.1 apply, all anchors in structures assigned to Seismic Design Categories (SDC) C, D, E, or F are required to satisfy the additional requirements of 17.10.2 through 17.10.7, regardless of whether earthquake-induced forces are included in the controlling load combination for the anchor design. In addition, all post-installed anchors in structures assigned to SDC C, D, E, or F must meet the requirements of ACI CODE-355.2 or ACI CODE-355.4 for prequalification of anchors to resist earthquake-induced forces. Ideally, for tension, anchor strength should be governed by yielding of the ductile steel element of the anchor. If the anchor cannot meet the specified ductility requirements of 17.10.5.3(a), then the attachment should be designed to yield if it is structural or light gauge steel, or designed to crush if it is wood. If ductility requirements of 17.10.5.3(a) are satisfied, then any attachments to the anchor should be designed not to yield. In designing attachments using yield mechanisms to provide adequate ductility, as permitted by 17.10.5.3(b) and 17.10.6.3(a), the ratio of specified yield strength to expected strength for the material of the attachment should be considered in determining the design force. The value used for the expected strength should consider both material overstrength and strain hardening effects. For example, the material in a connection element could yield and, due to an increase in its strength with strain hardening, cause a secondary failure of a sub-element or place extra force or deformation demands on the anchors. For a structural steel attachment, if only the specified yield strength of the steel is known, the expected strength should be taken as approximately 1.5 times the specified yield strength. If the actual yield strength of the steel is known, the expected strength should be taken as approximately 1.25 times the actual yield strength.

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

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17.10.2 Provisions of this chapter shall not apply to the design of anchors in plastic hinge zones of concrete structures resisting earthquake-induced forces.

17.10.3 Post-installed anchors shall be qualified for earthquake-induced forces in accordance with ACI CODE-355.2 or ACI CODE-355.4. The pullout strength, N_p, and steel strength in shear, V_{sa}, of post-installed expansion, screw, and undercut anchors shall be based on the results of the ACI CODE-355.2 Simulated Seismic Tests. For adhesive anchors, the steel strength in shear, V_{sa}, and the characteristic bond stresses, τ_{uncr} and τ_{cr}, shall be based on results of the ACI CODE-355.4 Simulated Seismic Tests.

17.10.4 Anchor reinforcement used in structures assigned to SDC C, D, E, or F shall be deformed reinforcement and shall be in accordance with the anchor reinforcement requirements of 20.2.2.

17.10.5 *Tensile loading design requirements*

17.10.5.1 If the tensile component of the strength-level earthquake-induced force applied to a single anchor or anchor group does not exceed 20% of the total factored anchor tensile force associated with the same load combination, it shall be permitted to design a single anchor or anchor group in accordance with 17.6 and the tensile strength requirements of Table 17.5.2.

17.10.5.2 If the tensile component of the strength-level earthquake-induced force applied to anchors exceeds 20% of the total factored anchor tensile force associated with the same load combination, anchors and their attachments shall be designed in accordance with 17.10.5.3. The anchor design tensile strength shall be determined in accordance with 17.10.5.4.

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R17.10.2 The possible higher levels of cracking and spalling in plastic hinge zones are beyond the conditions for which the nominal concrete-governed strength values in this chapter are applicable. Plastic hinge zones are considered to extend a distance equal to twice the member depth from any column or beam face, and also include any other sections in walls, frames, and slabs where yielding of reinforcement is likely to occur as a result of lateral displacements.

If anchors must be located in plastic hinge regions, they should be detailed so that the anchor forces are transferred directly to anchor reinforcement that is designed to transmit the anchor forces into the body of the member beyond the anchorage region. Configurations that rely on concrete tensile strength should not be used.

R17.10.3 Anchors that are not suitable for use in cracked concrete should not be used to resist earthquake-induced forces. Qualification of post-installed anchors for use in cracked concrete is an integral part of the qualification for resisting earthquake-induced forces in ACI CODE-355.2 and ACI CODE-355.4. The design values obtained from the Simulated Seismic Tests of ACI CODE-355.2 and ACI CODE-355.4 are expected to be less than those for static load applications.

R17.10.5 *Tensile loading design requirements*

R17.10.5.1 The requirements of 17.10.5.3 need not apply if the applied earthquake-induced tensile force is a small fraction of the total factored tensile force.

R17.10.5.2 If the ductile steel element is ASTM A36 or ASTM A307 steel, the f_{uta}/f_{ya} value is typically approximately 1.5, and the anchor can stretch considerably before rupturing at the threads. For other steels, calculations may need to be made to ensure that similar behavior can occur. Section R17.6.1.2 provides additional information on the steel properties of anchors. Use of upset threaded ends, whereby the threaded end of the anchor is enlarged to compensate for the area reduction associated with threading, can ensure that yielding occurs over the stretch length regardless of the tensile to yield strength ratio.

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17.10.5.3 Anchors and their attachments shall satisfy (a), (b), (c), or (d).

(a) For single anchors, the concrete-governed strength shall be greater than the steel strength of the anchor. For anchor groups, the ratio of the tensile load on the most highly stressed anchor to the steel strength of that anchor shall be equal to or greater than the ratio of the tensile load on anchors loaded in tension to the concrete-governed strength of those anchors. In each case:

(i) The steel strength shall be taken as 1.2 times the nominal steel strength of the anchor.

(ii) The concrete-governed strength shall be taken as the nominal strength considering pullout, side-face blowout, concrete breakout, and bond strength as applicable. For consideration of pullout in groups, the ratio shall be calculated for the most highly stressed anchor.

In addition, the following shall be satisfied:

(iii) Anchors shall transmit tensile loads via a ductile steel element with a stretch length of at least $8d_a$ unless otherwise determined by analysis.

(iv) Anchors that resist load reversals shall be protected against buckling.

(v) If connections are threaded and the ductile steel elements are not threaded over their entire length, the ratio of f_{uta}/f_{ya} shall be at least 1.3 unless the threaded portions are upset. The upset portions shall not be included in the stretch length.

(vi) Deformed reinforcing bars used as ductile steel elements to resist earthquake-induced forces shall be in accordance with the anchor reinforcement requirements of 20.2.2.

(b) Anchor or anchor groups shall be designed for the maximum tension that can be transmitted to the anchor or group of anchors based on the development of a ductile yield mechanism in the attachment in tension, flexure, shear, or bearing, or a combination of those conditions, considering both material overstrength and strain-hardening effects for the attachment. The anchor design tensile strength shall be calculated in accordance with 17.10.5.4.

(c) Anchor or anchor groups shall be designed for the maximum tension that can be transmitted to the anchors by a non-yielding attachment. The anchor design tensile strength shall be calculated in accordance with 17.10.5.4.

(d) Anchor or anchor groups shall be designed for the maximum tension obtained from factored load combinations that include E, with E_h increased by Ω_o. The anchor design tensile strength shall be calculated in accordance with 17.10.5.4.

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R17.10.5.3 Four options are provided for determining the required anchor or attachment strength to protect against nonductile tensile failure:

In option (a), anchor ductility requirements are imposed, and the required anchor strength is that determined using strength-level earthquake-induced forces acting on the structure. Research (Hoehler and Eligehausen 2008; Vintzileou and Eligehausen 1992) has shown that if the steel of the anchor yields before the concrete anchorage fails, no reduction in the anchor tensile strength is needed for earthquake–induced forces. Ductile steel anchors should satisfy the definition for steel element, ductile in Chapter 2. To facilitate comparison between steel strength, which is based on the most highly-stressed anchor, and concrete strength based on group behavior, the design is performed on the basis of the ratio of applied load to strength for the steel and concrete, respectively.

For some structures, anchors provide the best locations for energy dissipation in the nonlinear range of response. The stretch length of the anchor, shown in Fig. R17.10.5.3, affects the lateral displacement capacity of the structure; therefore, that length needs to be sufficient such that the displacement associated with the design-basis earthquake can be achieved (FEMA P-750). Observations from earthquakes indicate that the provision of a stretch length of $8d_a$ results in good structural performance. If the required stretch length is calculated, the relative stiffness of the connected elements needs to be considered. When an anchor is subject to load reversals, and its yielding length outside the concrete exceeds $6d_a$, buckling of the anchor in compression is likely. Buckling can be restrained by placing the anchor in a tube. However, care must be taken that the tube does not share in resisting the tensile load assumed to act on the anchor. For anchor bolts that are not threaded over their length, it is important to ensure that yielding occurs over the unthreaded portion of the bolt within the stretch length before failure in the threads. This is accomplished by maintaining sufficient margin between the specified yield and tensile strengths of the bolt. It should be noted that the available stretch length may be adversely influenced by construction techniques (for example, the addition of leveling nuts to the examples illustrated in Fig. R17.10.5.3).

In option (b), the anchor is designed for the tensile force associated with the expected strength of the attachment. Care must be taken in design to consider the consequences of potential differences between the specified yield strength and the expected strength of the attachment. An example is the design of connections of intermediate precast walls where a connection not designed to yield should develop at least $1.5S_y$, where S_y is the nominal strength of the yielding element based on its specified yield strength (refer to 18.5.2.2). Similarly, steel design manuals require structural steel connections that are designated nonyielding and part of the seismic load path to have design strengths that exceed a multiple of the nominal strength. That multiple depends on a factor relating the likely actual to specified yield strength

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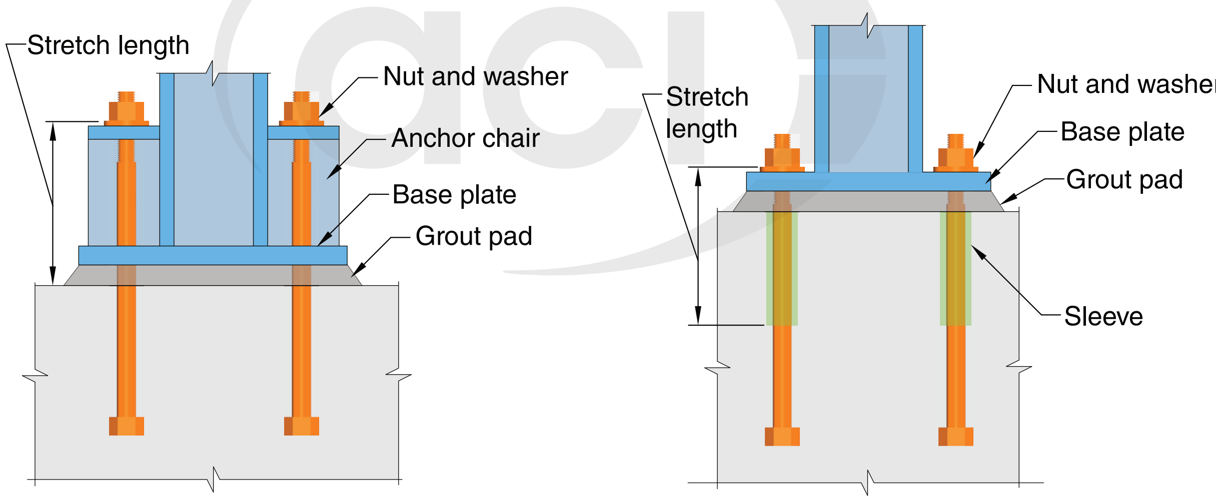
of the material and an additional factor exceeding unity to account for material strain hardening. For attachments of cold-formed steel or wood, similar principles should be used to determine the expected strength of the attachment in order to determine the required strength of the anchors.

Additional guidance on the use of options (a) through (d) is provided in the 2009 edition of the NEHRP Recommended Seismic Provisions for New Buildings and Other Structures (FEMA P-750). The design of anchors in accordance with option (a) should be used only if the anchor yield behavior is well defined and if the interaction of the yielding anchor with other elements in the load path has been adequately addressed. For the design of anchors in accordance with option (b), the force associated with yield of a steel attachment, such as an angle, baseplate, or web tab, should be the expected strength rather than the specified yield strength of the steel. Option (c) may apply to cases, such as the design of sill bolts where crushing of the wood limits the force that can be transferred to the bolt, or where the provisions of the American National Standards Institute/American Institute of Steel Construction (AISC) Code Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341) specify design loads based on member strengths.



(a) Anchor chair *(b) Sleeve*

Fig. R17.10.5.3—Illustrations of stretch length.

17.10.5.4 The anchor design tensile strength shall be calculated from (a) through (e) for the failure modes given in Table 17.5.2 assuming the concrete is cracked unless it can be demonstrated that the concrete remains uncracked.

(a) ϕN_{sa} for a single anchor, or for the most highly stressed individual anchor in an anchor group

(b) $0.75\phi N_{cb}$ or $0.75\phi N_{cbg}$, except that N_{cb} or N_{cbg} need not be calculated if anchor reinforcement satisfying 17.5.2.1(a) is provided

(c) $0.75\phi N_{pn}$ for a single anchor or for the most highly stressed individual anchor in an anchor group

(d) $0.75\phi N_{sb}$ or $0.75\phi N_{sbg}$

R17.10.5.4 The reduced anchor nominal tensile strengths associated with concrete failure modes is to account for increased cracking and spalling in the concrete resulting from earthquake effects. Because earthquake-resistant design generally assumes that all or portions of the structure are loaded beyond yield, it is likely that the concrete is cracked throughout for the purpose of calculating anchor strength. In locations where it can be demonstrated that the concrete does not crack, uncracked concrete may be assumed in calculating anchor strength as governed by concrete failure modes.

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(e) $0.75\phi N_a$ or $0.75\phi N_{ag}$

where ϕ is in accordance with 17.5.3.

17.10.5.5 If anchor reinforcement is provided in accordance with 17.5.2.1(a), no reduction in design tensile strength beyond that given in Table 21.2.1 for anchor reinforcement shall be required.

17.10.6 *Shear design requirements*

17.10.6.1 If the shear component of the strength-level earthquake-induced force applied to a single anchor or anchor group does not exceed 20 percent of the total factored anchor shear associated with the same load combination, it shall be permitted to design a single anchor or anchor group in accordance with 17.7 and the shear strength requirements of 17.5.2.

17.10.6.2 If the shear component of the strength-level earthquake-induced force applied to anchors exceeds 20% of the total factored anchor shear associated with the same load combination, anchors and their attachments shall be designed in accordance with 17.10.6.3. The anchor design shear strength for resisting earthquake-induced forces shall be determined in accordance with 17.7.

17.10.6.3 Anchors and their attachments shall satisfy (a), (b) or (c).

(a) Anchor or anchor groups shall be designed for the maximum shear that can be transmitted to the anchor or anchor groups based on the development of a ductile yield mechanism in the attachment in tension, flexure, shear, or bearing, or a combination of those conditions, and considering both material overstrength and strain-hardening effects in the attachment.

(b) Anchor or anchor groups shall be designed for the maximum shear that can be transmitted to the anchors by a non-yielding attachment.

(c) Anchor or anchor groups shall be designed for the maximum shear obtained from factored load combinations that include E, with E_h increased by Ω_o.

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R17.10.5.5 If anchor reinforcement conforming to 17.5.2.1(a) is provided, the strength of the connection is assumed to be controlled by yielding of the anchor reinforcement and not concrete breakout, and the factor ϕ applied to this case is given in Table 21.2.1.

R17.10.6 *Shear design requirements*

R17.10.6.1 The requirements of 17.10.6.3 need not apply if the applied earthquake-induced shear is a small fraction of the total factored shear.

R17.10.6.2 If the shear component of the earthquake-induced force applied to the anchor exceeds 20% of the total anchor shear force, three options are recognized to determine the required shear strength to protect the anchor or anchor group against premature shear failure.

R17.10.6.3 Option (a) of 17.10.5.3 is not permitted for shear because the cross section of the steel element of the anchor cannot be configured so that steel failure in shear provides any meaningful degree of ductility.

Design of the anchor or anchor group for the strength associated with force-limiting mechanisms under option (b), such as the bearing strength at holes in a steel attachment or the combined crushing and bearing strength for wood members, may be particularly relevant. Tests on typical anchor bolt connections for wood-framed structural walls (Fennel et al. 2009) demonstrated that wood components attached to concrete with minimum edge distances exhibited ductile behavior. Wood "yield" (crushing) was the first limiting state and resulted in nail slippage in shear. Nail slippage combined with bolt bending provided the required ductility and toughness for the structural walls and limited the loads acting on the bolts. Procedures for defining bearing and shear limit states for connections to cold-formed steel are described in AISI S100, and examples of strength calculations are provided in the AISI manual (AISI D100). In such cases, exceeding the bearing strength may lead to tearing and an unacceptable loss of connectivity. If anchors are located far from edges, it may not be possible to design such that anchor reinforcement controls the anchor strength. In such cases, anchors should be designed for overstrength in accordance with option (c).

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17.10.7 *Tension and shear interaction*

17.10.7.1 Single anchors or anchor groups that resist both tensile and shear forces shall be designed in accordance with 17.8, and the anchor design tensile strength calculated in accordance with 17.10.5.4.

17.11—Attachments with shear lugs

17.11.1 *General*

17.11.1.1 It is permitted to design attachments with shear lugs in accordance with 17.11.1.1.1 through 17.11.1.1.9. Alternatively, it is permitted to design using alternative methods if adequate strength and load transfer can be demonstrated by analysis or tests.

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R17.11—Attachments with shear lugs

R17.11.1 *General*

R17.11.1.1 The provisions of 17.11 cover concrete failure modes of attachments with shear lugs. These provisions do not cover the steel or welding design of the attachment base plate or shear lugs.

Attachments with shear lugs may be embedded in cast-in-place or precast concrete, or post-installed by using a blockout in the concrete that receives the shear lug and is then filled with a fluid, non-shrink grout as shown in Fig. R17.11.1.1a. Base plates with anchors provide moment resistance, which prevents pryout action on the shear lugs. Attachments with embedded shapes and without base plates and anchors, which must resist moment by pryout action on the embedment, are not covered in this section.

Bearing strength in shear refers to the strength prior to concrete fracture in front of the shear lug. Bearing failure occurs at small displacements (Cook and Michler 2017). Following bearing failure, there is a significant decrease in strength and increase in lateral displacement leading eventually to steel failure of the anchors (Fig. R17.11.1.1b) at lateral displacements at least an order of magnitude greater than that corresponding to bearing failure.

Types of attachments with shear lugs that satisfy 17.11.1.1.1 through 17.11.1.1.9 are shown in Fig. R17.11.1.1a. Shear lugs that are different than those covered in 17.11.1.1.1 through 17.11.1.1.9, such as shear lugs composed of steel pipe or attachments with shear lugs where the top of plate is located below the concrete surface, can be used provided adequate strength and load transfer can be demonstrated by analysis or tests.

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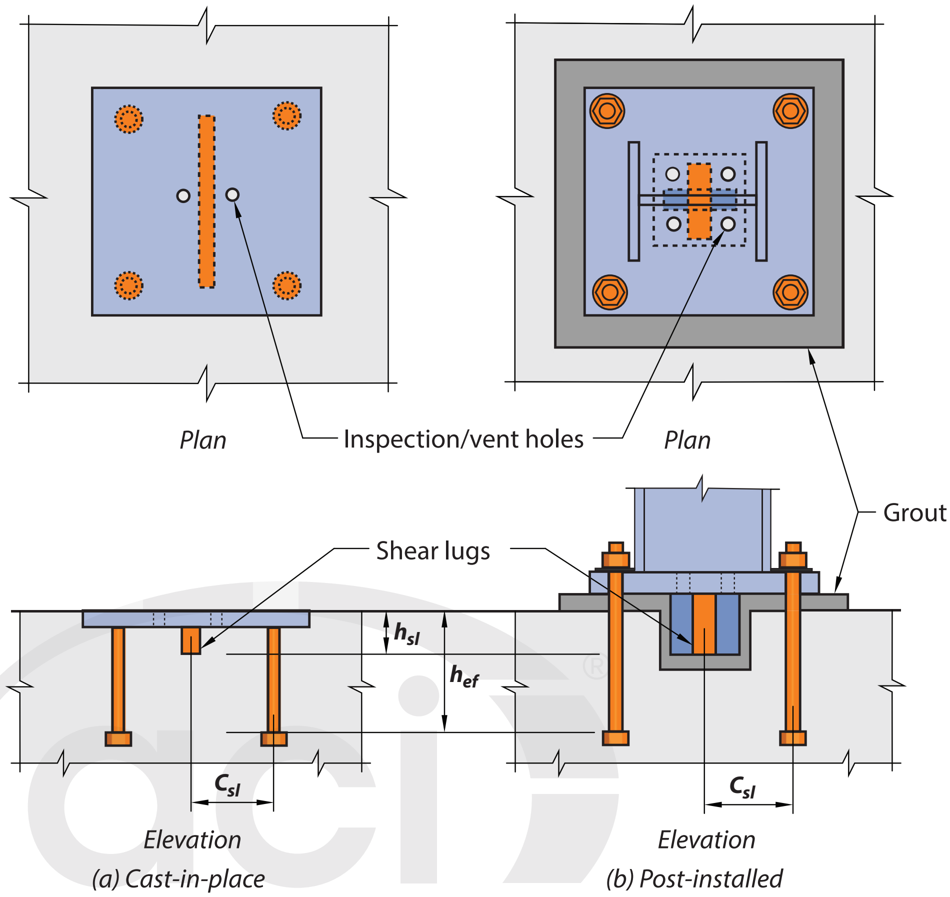

Fig. R17.11.1.1a—Examples of attachments with shear lugs.

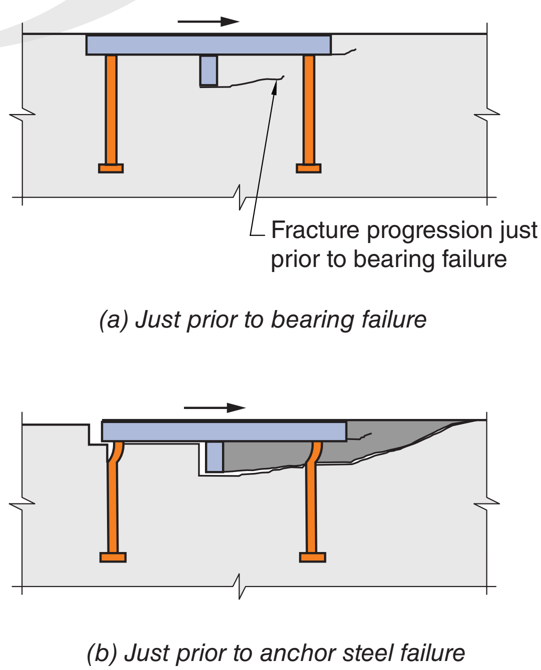

Fig. R17.11.1.1b—Bearing failure and subsequent anchor steel failure for embedded plate with shear lug (if concrete breakout is not applicable).

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17.11.1.1.1 Shear lugs shall be constructed of rectangular plates, or steel shapes composed of plate-like elements, welded to an attachment base plate.

17.11.1.1.2 A minimum of four anchors shall be provided that satisfy the requirements of Chapter 17 with the exception of the requirements of 17.5.1.3(f), (g), and (h) and the corresponding requirements of Table 17.5.2 for steel strength of anchors in shear, concrete breakout strength of anchors in shear, and concrete pryout strength of anchors in shear.

17.11.1.1.3 For anchors welded to the attachment base plate, tension and shear interaction requirements of 17.8 shall include a portion of the total shear on the anchor.

17.11.1.1.4 Bearing strength in shear shall satisfy $\phi V_{brg,sl} \geq V_u$ with $\phi = 0.65$.

17.11.1.1.5 Nominal bearing strength in shear, $V_{brg,sl}$, shall be determined by 17.11.2.

17.11.1.1.6 Concrete breakout strength of the shear lug shall satisfy $\phi V_{cb,sl} \geq V_u$ with $\phi = 0.65$.

17.11.1.1.7 Nominal concrete breakout strength, $V_{cb,sl}$, shall be determined by 17.11.3.

17.11.1.1.8 For attachments with anchors in tension, both (a) and (b) shall be satisfied:

(a) $h_{ef}/h_{sl} \geq 2.5$

(b) $h_{ef}/c_{sl} \geq 2.5$

17.11.1.1.9 The moment from the couple developed by the bearing reaction on the shear lug and the shear shall be considered in the design of the anchors for tension.

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R17.11.1.1.3 Although neglected in the bearing strength evaluation in 17.11.2, welded anchors resist a portion of the shear load because they displace the same as the shear lug. The portion of the applied shear, V_u, that each anchor carries, $V_{ua,i}$, is given by

$$V_{ua,i} = V_u\left(\frac{2d_a^2}{A_{ef,sl} + n2d_a^2}\right)$$

The effective bearing area of an anchor is assumed to be the diameter of the anchor multiplied by an effective bearing depth of twice its diameter (Cook and Michler 2017). The bearing reaction on the anchor is not large enough to fail the anchor in shear alone but does need to be considered in tension and shear interaction for steel failure (refer to 17.8).

R17.11.1.1.8 The lower bound limitations on the ratios of anchor embedment depth to shear lug embedment depth and anchor embedment depth to the distance between the centerline of the anchors in tension and the centerline of the shear lug in the direction of shear are based on available test data. The required lower limits reduce potential interaction between concrete breakout of the anchors in tension and bearing failure in shear of the shear lug.

R17.11.1.1.9 The bearing reaction on shear lugs occurs further below the surface of the concrete than the bearing reaction on anchors and embedded plates. As a result, the couple caused by the bearing reaction and the shear load needs to be considered when determining anchor tension.

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17.11.1.2 Horizontally installed steel base plates with shear lugs shall have a minimum 1 in. diameter hole along each of the long sides of the shear lug.

17.11.2 *Bearing strength in shear of attachments with shear lugs,* $V_{brg,sl}$

17.11.2.1 Nominal bearing strength in shear of a shear lug, $V_{brg,sl}$, shall be calculated as:

$$V_{brg,sl} = 1.7 f_c' A_{ef,sl} \psi_{brg,sl} \quad (17.11.2.1)$$

where $\psi_{brg,sl}$ is given in 17.11.2.2.

17.11.2.1.1 The effective bearing area, $A_{ef,sl}$, shall be below the surface of the concrete, perpendicular to the applied shear, and composed of areas according to (a) through (d):

(a) Bearing area of shear lugs located within $2t_{sl}$ of the bottom surface of the base plate if the top or bottom surface of the base plate is flush with the surface of the concrete

(b) Bearing area of shear lugs located within $2t_{sl}$ of the surface of the concrete if the base plate is above the surface of the concrete

(c) Bearing area of shear lugs located within $2t_{sl}$ of the interface with stiffeners

(d) Bearing area on the leading edge of stiffeners below the surface of the concrete

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R17.11.1.2 Base plate holes are necessary to verify proper concrete or grout consolidation around the shear lug and to avoid trapping air immediately below a horizontal plate. Holes in the base plate should be placed close to each face of the shear lug. For a single shear lug, place at least one inspection hole near the center of each long side of the shear lug. For a cruciform-shaped shear lug, four inspection holes are recommended, one per quadrant. For other configurations or long shear lug lengths, the licensed design professional should specify inspection hole locations that will permit adequate observation and allow trapped air to escape.

R17.11.2 *Bearing strength in shear of attachments with shear lugs,* $V_{brg,sl}$

R17.11.2.1 The nominal bearing strength in shear of a shear lug, $V_{brg,sl}$, given by Eq. (17.11.2.1) is based on a uniform bearing stress of $1.7f_c'$ acting over the effective area of the shear lug as discussed in Cook and Michler (2017). Although the bearing strength in shear of attachments with shear lugs is a function of bearing on the shear lug, embedded plate (if present), and welded anchors (if present), the method presented in 17.11.2 only includes the contribution of shear lugs. Cook and Michler (2017) discuss development of the method and a less conservative procedure to include bearing on the embedded plate and welded anchors.

R17.11.2.1.1 Figure R17.11.2.1.1 shows examples of effective bearing areas. The effective bearing area for stiffened shear lugs is applicable to both welded plates and steel shapes composed of plate-like elements in which case the web would be the stiffening element. The limit of a distance of $2t_{sl}$ in determining the effective bearing area is described in Cook and Michler (2017).

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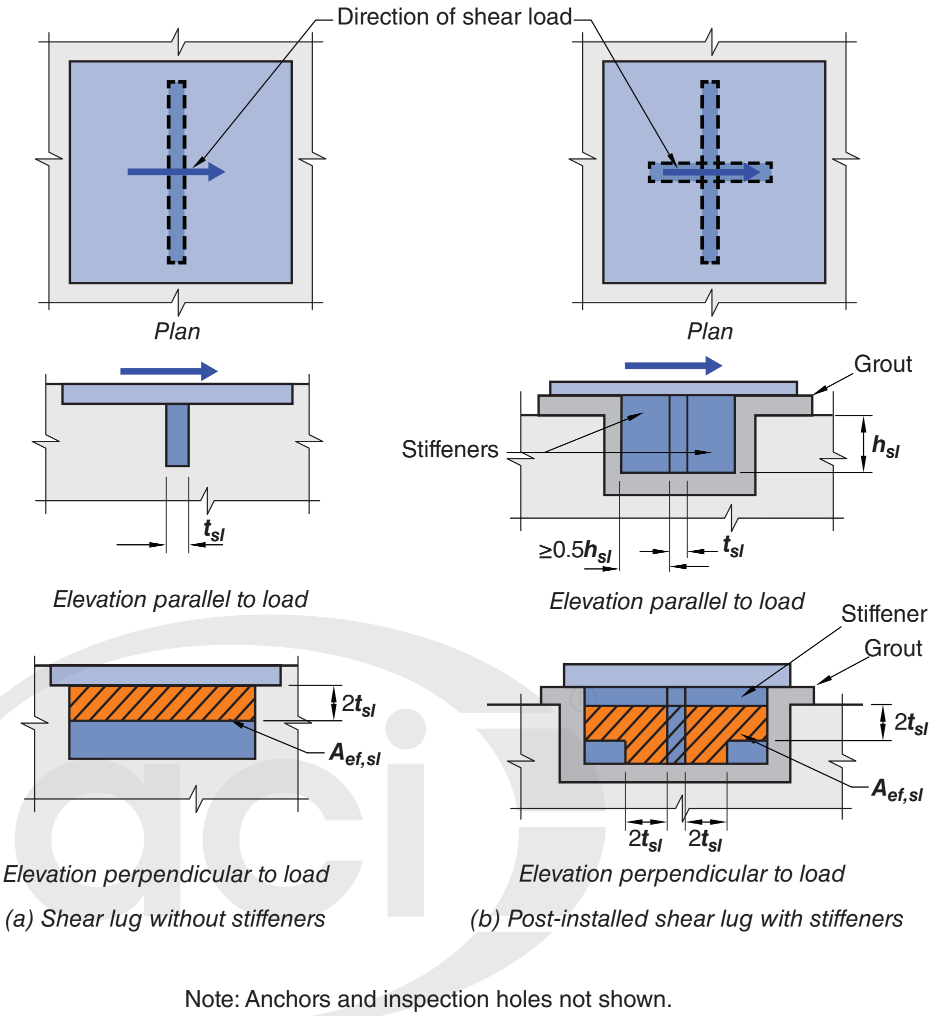

Fig. R17.11.2.1.1—Examples of effective bearing areas for attachments with shear lugs.

17.11.2.2 *Bearing factor,* $\psi_{brg,sl}$

17.11.2.2.1 Modification factor, $\psi_{brg,sl}$, for the effects of axial load, P_u, on bearing strength in shear, shall be determined by (a), (b), or (c):

(a) For applied axial tension:

$$\psi_{brg,sl} = 1 + \frac{P_u}{nN_{sa}} \leq 1.0 \qquad (17.11.2.2.1a)$$

where P_u is negative for tension and n is the number of anchors in tension.

(b) For no applied axial load:

$$\psi_{brg,sl} = 1 \qquad (17.11.2.2.1b)$$

(c) For applied axial compression:

$$\psi_{brg,sl} = 1 + 4\frac{P_u}{A_{bp}f_c'} \leq 2.0 \qquad (17.11.2.2.1c)$$

where P_u is positive for compression.

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17.11.2.3 If used, the length of shear lug stiffeners in the direction of the shear load shall not be less than $0.5h_{sl}$.

17.11.2.4 For attachments with multiple shear lugs arranged perpendicular to the direction of applied shear, the bearing strength of the individual shear lugs may be considered to be additive provided the shear stress on a shear plane in the concrete at the bottom of the shear lugs, and extending between the shear lugs, does not exceed $0.2f_c'$. The nominal bearing strength of each individual lug shall be determined by Eq. (17.11.2.1) using the effective area of the lug.

17.11.3 *Concrete breakout strength of shear lug,* $V_{cb,sl}$

17.11.3.1 Nominal concrete breakout strength of a shear lug for shear perpendicular to the edge, $V_{cb,sl}$, shall be determined from 17.7.2 using Eq. (17.7.2.1a), where V_b is calculated using Eq. (17.7.2.2.1b) with c_{a1} taken as the distance from the bearing surface of the shear lug to the free edge and where A_{Vc} is the projected area of the failure surface on the side of the concrete member.

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R17.11.2.4 The limitation for considering multiple shear lugs to be effective is based on the maximum limits for shear friction in Table 22.9.4.4 and two tests reported in Rotz and Reifschneider (1984). The area of the shear plane is the clear distance between adjacent shear lugs measured in the direction of the applied shear multiplied by the width of the shear lugs perpendicular to the applied shear.

R17.11.3 *Concrete breakout strength of shear lug,* $V_{cb,sl}$

R17.11.3.1 The method for evaluating concrete breakout strength where shear is perpendicular to an edge is similar to that used in 17.7.2 for anchors. The difference is in the determination of A_{Vc}, which is illustrated in Fig. R17.11.3.1. The method has been confirmed by tests where the shear lug is concentrically loaded in shear (Gomez et al. 2009; Cook and Michler 2017). With shear transferred by the shear lug, embedded plate (if present), and welded anchors (if present), the bearing surfaces all displace the same amount with any incremental change in applied shear. This behavior is similar to connections with anchors welded to steel attachments where concrete edge failure originates from the row of anchors farthest from the edge. In anchorages with shear lugs, the effective contributions to concrete breakout strength from the bearing areas of the shear lug and embedded plate (if present) dominate over the contribution from the effective bearing area of anchors farther from the edge than the shear lug. As a result, concrete breakout strength for the anchorage should be determined based on the concrete breakout surface originating at the shear lug (Fig. R17.11.3.1).

The nominal concrete breakout strength of a shear lug is based on Eq. (17.7.2.2.1b) for V_b that applies to concrete edge failure in shear for large diameter anchors.

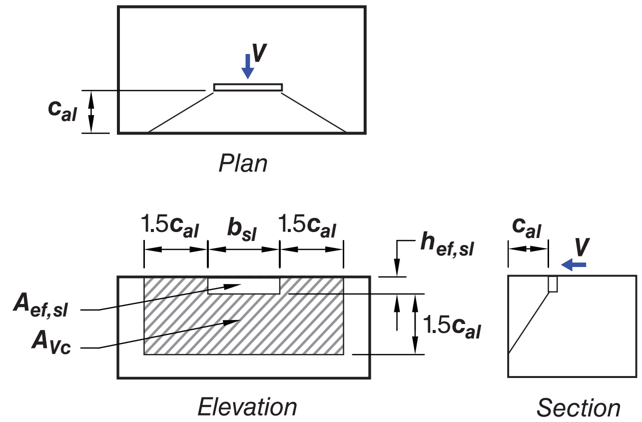

Fig. R17.11.3.1—Example of A_{Vc} *for a shear lug near an edge.*

CODE

17.11.3.1.1 A_{Vc} is the projected concrete failure area on the side face of the concrete that is approximated as the rectangular shape resulting from projecting horizontally $1.5c_{a1}$ from the edge of the shear lug and projecting vertically $1.5c_{a1}$ from the edge of the effective depth of the shear lug, $h_{ef,sl}$. The effective area of the shear lug, $A_{ef,sl}$, shall not be included. The effective embedment depth of the shear lug, h_{ef}, shall be taken as the distance from the concrete surface to the bottom of the effective bearing area, $A_{ef,sl}$.

17.11.3.2 Nominal concrete breakout strength of a shear lug for shear parallel to the edge shall be permitted to be determined in accordance with 17.7.2.1(c) using Eq. (17.7.2.1(a)) with c_{a1} taken as the distance from the edge to the center of the shear lug and with $\psi_{ec,V}$ taken as 1.0.

17.11.3.3 For shear lugs located at a corner, the limiting concrete breakout strength shall be determined for each edge, and the minimum value shall be used.

17.11.3.4 For cases with multiple shear lugs, the concrete breakout strength shall be determined for each potential breakout surface.

COMMENTARY

R17.11.3.2 The concrete breakout strength for shear lugs loaded parallel to the edge is based on 17.7.2.1(c) for concrete failure with load applied parallel to the free edge, assuming shear lug breakout behavior is similar to that of a single anchor.

R17.11.3.3 The concrete breakout strength for shear lugs located near a corner is based on 17.7.2.1(d) for anchors.

R17.11.3.4 The concrete breakout strength for multiple shear lugs is based on R17.7.2.1 and shown in Fig. R17.7.2.1b Case 1 and Case 2.