Title no. 110-S34

Recommended Procedures for Development and Splicing of Post-Installed Bonded Reinforcing Bars in Concrete Structures

by Finley A. Charney, Kamalika Pal, and John Silva

The use of post-installed anchors for connections in both new and existing structures is commonplace and growing. A related type of post-installed connection that has been employed for decades is the bonding of reinforcing bars into holes drilled into concrete to facilitate structural extensions and strengthening of reinforced concrete structures. Although ACI 318-11 includes provisions for the design of adhesive anchors in concrete (anchor rods bonded with adhesive in a drilled hole), it does not address the design of post-installed reinforcing bars. This paper addresses the issue by providing background into the adhesive anchor design and development length provisions of ACI 318 as well as the provisions for post-installed reinforcing bars available in international standards. The paper makes recommendations for the development of a new procedure that is applicable to the design of post-installed reinforcing bars.

Keywords: adhesive; anchorage; bond; development; post-installed; reinforcement.

INTRODUCTION

The use of post-installed anchors for connections in both new and existing structures is commonplace and growing (Fig. 1). A related type of post-installed connection that has been employed for decades is the bonding of reinforcing bars into holes drilled into concrete to facilitate structural extensions and strengthening. A variety of cases may be considered in this respect: creating a lap splice of a post-installed reinforcing bar with existing reinforcing to facilitate the extension of an existing slab (Fig. 2(a)) or an existing column (Fig. 2(b)), providing starter bars for a new column on an existing foundation (Fig. 2(c)), or installing dowels for a new corbel to be added to an existing column (Fig. 2(d)).

Anchorage to concrete is addressed in ACI 318-11,[1] Appendix D, which includes provisions for the design of adhesive anchors in addition to post-installed expansion and undercut anchors and cast-in L-, J-, and headed bolts. Prior to the issuance of the 2011 edition of ACI 318[1] and the companion qualification standard ACI 355.4[2] in late 2011, however, provisions for adhesive anchors did not exist in the code. Therefore, since 2006, most adhesive anchors have been tested, assessed, and designed using procedures provided by the ICC Evaluation Service in AC308.[3]

The adhesive anchor design provisions in AC308[3] are intended to augment the anchor design provisions of earlier versions of ACI 318, Appendix D, by adding the expressions for determining the bond strength of the anchor or anchor group. This limit state replaces the check for pullout resistance that applies to post-installed expansion or undercut anchors.

AC308[3] further provides the criteria for determining the bond strength and other parameters used in adhesive anchor design. AC308[3] specifically excludes the

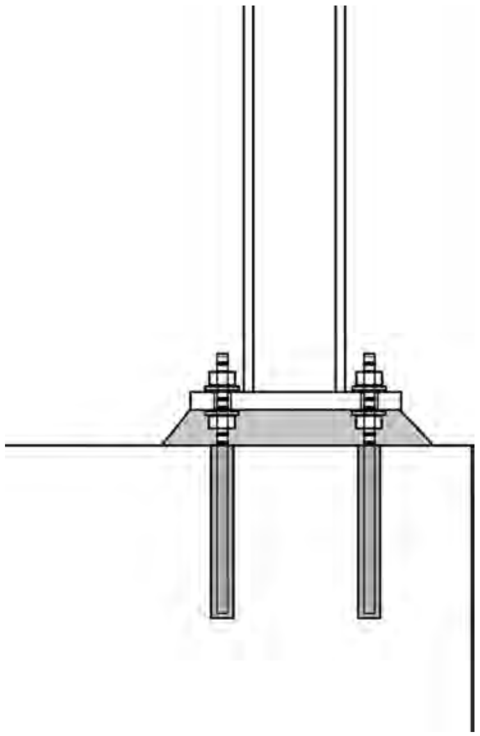

Fig. 1—Adhesive anchors used to secure column baseplate.

qualification of adhesive anchor systems for post-installed reinforcing bar lap-splice-type applications, as described in Fig. 2(a) and (b). Applications, as shown in Fig. 2(c) and (d), are potentially addressed, provided they do not exceed the maximum bond length of 20 bar diameters specified in AC308.[3] The adhesive anchor design provisions included in ACI 318-11[1] are in substantial agreement with those incorporated into AC308.[3] The provisions and nomenclature from ACI 318-11[1] are used herein. ACI 318-11[1] and ACI 355.4-11[2] likewise do not address post-installed reinforcing bar lap-splice-type applications.

Embedment requirements for cast-in reinforcing bars are addressed through the development length and splice provisions of Chapter 12 of ACI 318-11.[1] These provisions likewise do not address post-installed reinforcing. Thus, at

ACI Structural Journal, V. 110, No. 3, May-June 2013.
MS No. S-2011-182.R2 received September 29, 2011, and reviewed under Institute publication policies. Pertinent discussion including author's closure, if any, will be published in the March-April 2014 *ACI Structural Journal* if the discussion is received by November 1, 2013.

ACI member **Finley A. Charney** *is a Professor in the Department of Civil and Environmental Engineering at Virginia Polytechnic Institute and State University, Blacksburg, VA. His research interests include the design and behavior of structures subjected to extreme loads.*

Kamalika Pal *is a Technical Services Engineer with Hilti North America, Tulsa, OK.*

John Silva, *FACI, is Director of Codes and Standards for Hilti North America and a licensed structural engineer. His research interests include anchorage to concrete, seismic strengthening, and seismic restraint of nonstructural components.*

present, there are no U.S. standards for the design of post-installed adhesively bonded reinforcing bars to be used as reinforcing—that is, as depicted in Fig. 2. For the purposes of this discussion, such applications will be referred to simply as "post-installed reinforcing bars."

The European Organization for Technical Approvals (EOTA) developed a guide document for post-installed reinforcing bar connections that has been in use in Europe for over a decade. This document, TR023,[4] provides testing and assessment procedures to determine the fitness of proprietary systems for anchoring post-installed reinforcing bars. TR023[4] asserts that development length concepts applicable to cast-in bars can be used directly if it can be shown that adhesively bonded reinforcing bars demonstrate strength characteristics that are comparable to those required for cast-in deformed bars. Similar concepts are the basis of design recommendations provided by Spieth,[5] Kunz and Münger,[6] Simons,[7] and Simons and Eligehausen.[8]

RESEARCH SIGNIFICANCE

As stated previously, there are no U.S. standards available for the design of post-installed reinforcing bars in concrete structures. Design guidelines can be proposed, however, based on research already performed. The objective of this paper is to serve as a basis for the development of such guidelines and identify areas of needed research.

Distinguishing post-installed adhesive anchors from post-installed reinforcing bars

Design of post-installed adhesive anchors under ACI 318-11,[1] Appendix D, generally proceeds under the assumption that the free (unembedded) end of the anchor will be used to secure a structural element—often a structural steel shape, as shown in Fig. 1. In contrast, the applications shown in Fig. 2 involve reinforcing bars post-installed in existing concrete at one end and cast into new concrete at the opposite end. Theoretically, the loading on the bars is exclusively tensile, resulting either from direct tension forces in the concrete member or

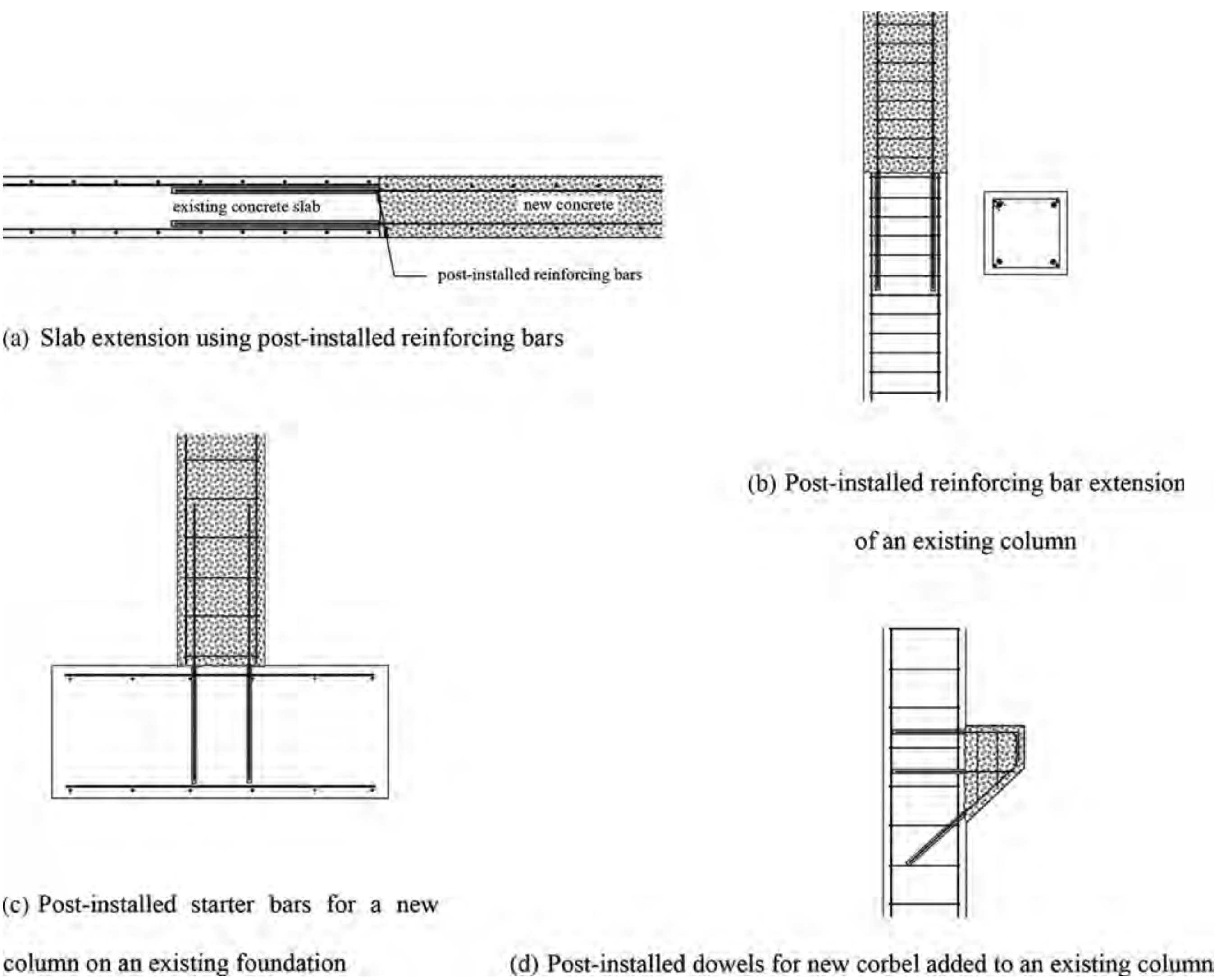

Fig. 2—Typical applications of post-installed reinforcing bars.

from the strut-and-tie action associated with shear friction. Incidental dowel action resulting from interface shear is generally discounted.

A second—but equally critical—distinction between the connection types illustrated in Fig. 2 and the baseplate detail shown in Fig. 1 is associated with failure modes assumed for the design. ACI 318-11,[1] Appendix D, lists a variety of failure modes that must be accounted for in the design of post-installed anchors. These include steel failure, pullout, various forms of concrete breakout, and bond failure. Not directly accounted for, however, is splitting of the concrete—a failure mode that is highly dependent on a number of factors that can be difficult to quantify. Rather than explicitly calculating the splitting resistance of an anchorage, ACI 318-11,[1] Appendix D, provides edge distance, spacing, and member thickness requirements that are intended to preclude this failure mode. In contrast, the principal failure mode addressed by the development length provisions of ACI 318-11,[1] Chapter 12, is splitting. The complementary nature of the two sets of provisions permits the potential formulation of a harmonized approach to the design of post-installed reinforcing bars that considers all relevant failure modes.

It is clear that the applications in Fig. 2(c) and (d) could be designed using the basic principles of anchor theory as stipulated by ACI 318-11,[1] Appendix D, for post-installed adhesive anchors. However, the applications in Fig. 2(a) and (b) involve the transmission of force between parallel reinforcing bars and are likely governed by splitting. These types of connections are more closely related to the principles of bond, development length, and splicing of reinforcement, as covered in ACI 318-11,[1] Chapter 12. A methodology for determining the required embedment of post-installed reinforcing bars that addresses both the splitting-dominated behavior of near-edge bars and the classical anchor behavior of bars embedded in larger concrete masses far from edges would address all of these cases.

REQUIRED PARAMETERS FOR ASSESSMENT AND DESIGN

The development of a methodology for determining the embedment length and splice length of post-installed reinforcing bars must necessarily draw extensively from cast-in reinforcement development length theory. This theory must be amended, however, to include certain concepts already incorporated into the design of adhesive anchors, including all parameters relevant for the bond strength of the adhesive (for example, hole drilling and cleaning, temperature, and loading type).

Any procedure for the design of adhesively bonded reinforcing bars must consider the body of research already performed on such reinforcement, design procedures that have been developed for adhesive anchors, and recommendations already provided by other standards organizations.[4,5,7,9,10]

Critical installation parameters and system assessment

The choice of anchor type, adhesive, and installation technique is interrelated and is a function of use, loading direction, loading duration, and a variety of environmental considerations. In nearly all cases, regardless of the system selected, a hole is drilled and cleaned, the adhesive is injected, the bar is inserted, and the adhesive is allowed to cure. Bond stress values provided in product literature and in assessment documents—for example, reports issued by the ICC Evaluation Service—are generally predicated on the use of a particular hole-drilling method and associated hole-cleaning and preparation procedure. One should strictly adhere to these requirements to avoid unanticipated reductions in bond strength.

Hole drilling—Holes for adhesive anchors are sized to keep the annular gap between the anchor rod and the concrete as small as practically possible. This minimizes the effects of adhesive shrinkage and improves the stiffness of the resulting connection. Holes for use with threaded rods are generally not more than 1/8 in. (3 mm) larger in diameter than the nominal rod diameter. Drilling methods include hammer drilling, compressed air drilling, and diamond core drilling. A summary of the effect of use of different drilling methods on bond behavior is provided in Eligehausen et al.[11]

Hole cleaning—Hole cleaning, as specified by the manufacturer, typically consists of initial dust removal using a vacuum or compressed air followed by a mechanical scouring operation (for example, wire brush) and a final dust removal procedure. Use of compressed air alone (without scouring) is generally not adequate to remove the dust from the sides of the hole. Previous investigations[12,13] have indicated that failure to clean the hole in accordance with the manufacturer's specifications can reduce the strength of the connection by as much as 50%, depending on the type of adhesive and the method of adhesive injection.

Adhesive type and delivery system—Various types of adhesive anchor systems are in use in different parts of the world, such as Europe and Asia, including cartridge injection systems, capsule systems, and bulk delivery systems, whereas cartridge injection systems are predominant in the United States.

It is important that delivery of the adhesive into the hole be accomplished with an absolute minimum of entrained air. This is usually achieved by injecting the adhesive from the bottom of the hole. For deeper holes, an extension is attached to the nozzle to permit injection from the bottom of the hole. For horizontal and overhead applications, the extension may be equipped with a stopper that contains the adhesive during injection. Holes are typically filled from one-half to two-thirds of their total length to ensure that the annular gap around the anchor element is filled with adhesive to the surface of the concrete.

Installation of reinforcing bars of larger diameter in deep holes may require use of case-specific methods to ensure that the installation proceeds without premature setting of the adhesive and/or binding of the bar before or after insertion to the required embedment.

Assessment

The assessment of the adhesive anchor system under AC308[3] and ACI 355.4-11[2] results in design bond stresses and other parameters to be used in conjunction with the design model developed by Eligehausen et al.[14] at the University of Stuttgart, which is the basis for the design provisions in AC308[3] and ACI 318-11.[1] Factors included in the evaluation include performance in uncracked and cracked concrete service conditions, response to variations in concrete temperature, sensitivity to deviations from specified hole-cleaning procedures, and response to long-term loading. The assessment specifically excludes development of bond stresses for post-installed reinforcing bars,[2] whereby such an assessment would necessarily include the effects of near-edge performance and longer embedment lengths than typically considered for anchors.

An assessment procedure that addresses the post-installed reinforcing bar application for U.S. practice is not yet available. Such an assessment procedure should include many of the criteria addressed in the aforementioned documents but should additionally set minimum thresholds on the design bond stress to ensure behavior that is compatible with cast-in-place reinforcing. It is also reasonable to expect that the assessment should mandate an acceptable stiffness range to preclude zipper-type failures due to shear lag, as discussed by Spieth[5]; this is a parameter not currently considered in TR023.[4] The assessment must also include an evaluation of the adhesive delivery system and requirements for installers to ensure that installation in larger hole diameters and lengths can be executed properly—for example, without air entrainment.

DEVELOPMENT LENGTH CALCULATIONS—CAST-IN REINFORCING BARS

A review of the procedures to determine the required development and splice lengths for cast-in reinforcing bars, as provided in Chapter 12 of ACI 318-11,[1] follows. The expression currently used for determination of the basic development length ℓ_d is given as

$$\ell_d = \left[\frac{3}{40} \frac{f_y}{\lambda \sqrt{f_c'}} \frac{\psi_t \psi_e \psi_s}{\dfrac{c_b + K_{tr}}{d_b}} \right] d_b \text{ (lb, in.)} \quad (1)$$

where the confinement term $(c_b + K_{tr})/d_b$ may not be taken as greater than 2.5; and K_{tr} is a function of the cross-sectional area of transverse reinforcement available to restrain splitting cracks along the bar being developed. (Refer to Appendix A* for SI-metric equivalents for the equations presented in this paper that are unit-dependent.)

In Eq. (1), f_y and f_c' are the tension yield and compressive strengths (psi) of the reinforcing steel and concrete, respectively; d_b is the diameter of the bar being developed; λ is a modifier for lightweight concrete; c_b is the minimum edge distance; K_{tr} is a confinement term; and ψ_t, ψ_e, and ψ_s are modifiers for top reinforcement, epoxy coating, and bar size, respectively.

It is important to note that a strength reduction factor ϕ of 0.8 was used in the development of the expression on which Eq. (1) is based; however, simplifications to the original equation developed by Orangun et al.[15] led to the ϕ factor effectively becoming 0.9.[16]

Under normal circumstances, the size of the reinforcing bar being developed is based on calculations that include a strength reduction factor (for example, 0.9 for flexure) and, thus, the overall strength reduction for development length is the product of these two factors. For typical flexure, the combined strength reduction factor inherent in Eq. (1) is approximately 0.8 (0.9 × 0.9).

*The Appendix is available at **www.concrete.org** in PDF format as an addendum to the published paper. It is also available in hard copy from ACI headquarters for a fee equal to the cost of reproduction plus handling at the time of the request.

While not explicitly stated in ACI 318-11,[1] the term "development length" implies that this is the minimum bond length required to develop the expected yield strength of the reinforcement at the critical section. The physical mechanism that allows the strength of the bar to be developed is more complex than that idealized by bond stress alone, as it relies on bearing between the concrete and the projecting ribs on the bar, the distance between the embedded bar and the closest free edge of the concrete, and the presence (or lack thereof) of confining reinforcement. The confinement term in Eq. (1), $(c_b + K_{tr})/d_b$, accounts for both confining reinforcement and edge distance. According to the ACI 318-11[1] commentary, where this term is less than 2.5, splitting failure is expected to occur. For values greater than 2.5, pullout—as characterized by crushing of the concrete around the bar lugs—is the presumed concrete failure mode under an overload condition. Where the computed confinement term is greater than or equal to 2.5, ACI 318-11[1] requires a limiting value of 2.5 to be used in the computation of development length—that is, shorter lengths that might lead to concrete breakout failure are precluded. Nevertheless, failure due to bar rupture is unlikely due to the fact that the actual rupture strength of the bar is significantly greater than the nominal yield strength.

In the section on excess reinforcement, ACI 318-11[1] permits a linear reduction of the development length where the area of steel provided exceeds the area of steel required. In such cases, it becomes more likely that concrete breakout—not pullout or splitting—will be the controlling failure mode in overload conditions. As mentioned previously, failure by concrete breakout is explicitly considered in Appendix D of ACI 318-11[1] but not in Chapter 12.

While Eq. (1) is not a bond stress equation per se, it is instructive to recast it in terms of an equivalent bond stress equation as follows.

Let τ_{eq} be the equivalent bond stress. Then

$$A_s \cdot f_y = \tau_{eq} \cdot \pi \cdot d_b \cdot \ell_d \quad (2)$$

Using $A_s = \pi(d_b)^2/4$ and substituting Eq. (1) for ℓ_d on the right-hand side of Eq. (2) results in the following

$$\tau_{eq} = 3.33 \cdot \lambda \cdot \sqrt{f_c'} \cdot \left(\frac{c_b + K_{tr}}{d_b} \right) \left(\frac{1}{\psi_t \cdot \psi_e \cdot \psi_s} \right) \text{ (lb, in.)} \quad (3)$$

For post-installed reinforcing bars, with ψ_t and ψ_e set to unity, the expressions for required bond stress for smaller- and larger-diameter bars are as follows.

No. 6 (No. 19 [19.05 mm]) bars and smaller

$$\tau_{eq} = 4.16 \cdot \lambda \cdot \sqrt{f_c'} \cdot \left(\frac{c_b + K_{tr}}{d_b} \right) \leq 10.3 \cdot \sqrt{f_c'} \text{ (lb, in.)} \quad (4a)$$

No. 7 (No. 22 [22.23 mm]) bars and larger

$$\tau_{eq} = 3.33 \cdot \lambda \cdot \sqrt{f_c'} \cdot \left(\frac{c_b + K_{tr}}{d_b} \right) \leq 8.33 \cdot \sqrt{f_c'} \text{ (lb, in.)} \quad (4b)$$

where the upper limits on development length assume the maximum value of 2.5 for the confinement factor. Table 1 provides calculated development lengths and the associated effective uniform bond stresses as a function of bar diameter for a range of reinforcing bar sizes.

The bond stress values in Table 1 pertain to cast-in deformed reinforcing bars. However, if it can be shown that the behavior of post-installed bonded reinforcing bars at ultimate is comparable (for the same concrete strengths and concrete types) to that of cast-in bars, it can be argued that the use of the expressions for development length provided in ACI 318-11,[1] Chapter 12, is acceptable for the post-installed case as well.

It is important to note that development length, as specified in ACI 318-11,[1] Chapter 12, is not dependent on the degree of cracking in the concrete in which the bars are embedded, whereas the anchor provisions of ACI 318-11,[1] Appendix D, require significant adjustments to the design resistance where anchors are located in regions where cracking is expected. Furthermore, the development length and splice provisions of ACI 318-11,[1] Chapter 12, consider the influence of transverse reinforcement only (as a means of limiting splitting crack opening). In contrast, anchor theory does not consider the influence of reinforcing bars placed orthogonally to the direction of applied tension but instead provides specific provisions for so-called anchor reinforcement (reinforcement placed parallel to the anchor direction and developed within the presumed failure body). While at first glance these approaches seem incompatible, they are in fact closely related. As stated previously, ACI 318-11,[1] Appendix D, does not contain explicit predictor equations for splitting failure. Instead, the anchor edge distance, anchor spacing, and member thickness are controlled such that splitting should not preclude other failure modes. (An exception is made in connection with the term $\psi_{cp,N}$ defined in ACI 318-11,[1] Section D.5.2.7. This term, described in more detail in the section on ACI 318-11,[1] Appendix D, concrete breakout provisions of this paper, applies when "supplementary reinforcement to control splitting" is not present. In this case, it may be assumed that the supplementary reinforcement is perpendicular to the anchor tension load direction, in contrast to the use of this term elsewhere in ACI 318-11,[1] Appendix D.)

Because the splitting failure mode is one of the two admissible failure modes associated with the development length provisions of ACI 318-11,[1] Chapter 12, the explicit inclusion of the influence of transverse reinforcement to control the splitting crack width is appropriate. On the other hand, anchor reinforcement, as defined in ACI 318-11,[1] Appendix D, is closely related to the (noncontact) lap splice provisions of ACI 318-11,[1] Chapter 12. Note that, in accordance with Eq. (1), where no confining reinforcement is present, the confinement term achieves a maximum at a value of c_b/d_b equal to 2.5. That is, beyond an edge distance of $2.5d_b$, the presence of transverse reinforcement does not influence the development length. For reasons of practicality, the use of post-installed reinforcing bars generally occurs at edge distances in excess of 2.5 bar diameters, so the presence of transverse reinforcement is not decisive.

For cases where post-installed reinforcing bars are to be spliced with existing reinforcing, it must be kept in mind that the capacity of the existing bar, as dictated by the development length provisions, may govern the splice length. For example, if a post-installed bar is spliced with a bar that is placed near the top of a wall, the ψ_t term does not apply to the post-installed bar but may apply to the cast-in bar.

Table 1—Required development length and effective uniform bond stress as function of bar diameter in accordance with ACI 318-11,[1] Section 12.2*†

Bar size	Concrete strength, psi					
	3000	4000	5000	6000	7000	8000
≤No. 6 (No. 19 [19.05 mm])	26.3 in.	22.8 in.	20.4 in.	18.6 in.	17.2 in.	16.1 in.
	566 psi	653 psi	730 psi	800 psi	864 psi	924 psi
≥No. 7 (No. 22 [22.23 mm])	32.9 in.	28.5 in.	25.5 in.	23.2 in.	21.5 in.	20.1 in.
	456 psi	527 psi	589 psi	645 psi	697 psi	745 psi

*ASTM A615 Grade 60 (414 MPa) reinforcing bar (Grade 60 [414 MPa] reinforcing bar has minimum yield strength of 60 ksi [413.68 MPa]).
†$(c_b + K_{tr})/d_b = 2.5$.
Notes: 1 in. = 25.4 mm; 1000 psi = 7 MPa.

ANCHOR THEORY AS APPLIED TO POST-INSTALLED REINFORCING BARS

In general, the design of anchors consists of checking single anchors or groups of anchors for tension, shear, or combined tension and shear. As discussed previously, reinforcing bars are not typically designed for dowel action; thus, the design of post-installed reinforcing bars used as reinforcing is focused on the tension case only.

Design requirements for cast-in and post-installed anchors are given in ACI 318-11,[1] Appendix D.

ACI 318-11,[1] Appendix D, concrete breakout provisions

ACI 318-11,[1] Appendix D, provides five basic tension failure modes for which strength calculations are explicitly required:

1. Bolt rupture (Section D.5.1);
2. Concrete breakout (Section D.5.2);
3. Anchor pullout (Section D.5.3);
4. Side-face blowout failure (Section D.5.4); and
5. Bond failure (Section D.5.5).

The pullout provisions in Section D.5.3 are related to the behavior of headed, undercut, and expansion anchors and do not apply to adhesive anchors because these provisions do not consider the effect of anchor spacing and edge distance on pullout strength. Instead, in ACI 318-11,[1] the bond failure mode is explicitly addressed with a formulation closely related to the concrete breakout predictive equations. That is, the effect of anchor spacing and edge distance on bond strength is directly incorporated. Because the concrete breakout strength associated with a given embedment depth represents the "maximum carrying capacity" of the concrete locally around the anchor, it is the lesser of the concrete breakout strength and the computed bond strength that is decisive for adhesive anchor tension design.

For anchors in tension, the basic strength requirement in ACI 318-11,[1] Appendix D, is given by

$$\phi N_n \leq N_{ua} \tag{5}$$

where N_n is the computed nominal strength (steel fracture, concrete capacity, and pullout); N_{ua} is the factored design load; and ϕ is the strength reduction factor. The strength reduction factors for concrete failure modes are a function of load type (tension or shear) failure mode, presence of supplementary reinforcement, sensitivity to installation procedures, and expected reliability. They range between 0.65 and

0.45. The strength reduction factors for steel failure modes are 0.75 and 0.65 for ductile and brittle steels, respectively.

The nominal steel strength in tension of an anchor or uniformly loaded anchor group is given by

$$N_{sa} = n \cdot A_{se,N} \cdot f_{uta} \tag{6}$$

where n is the number of anchors in the group; $A_{se,N}$ is the effective cross-sectional area of a single anchor in tension; and f_{uta} is the steel-specified minimum ultimate strength. The commentary of ACI 318-11,[1] Appendix D, states that the steel fracture capacity is best represented by f_{uta} instead of f_{ya} because the large majority of anchor materials do not exhibit a well-defined yield point. On the other hand, deformed reinforcing bars, which are assumed to be inherently ductile when used as reinforcing in accordance with ACI 318-11,[1] are specified in terms of yield—not ultimate—strength. For these reasons, the ACI 318-11,[1] Appendix D, approach to the steel resistance is not applicable for the design of post-installed reinforcing bars.

The breakout strength is based on the Concrete Capacity Design (CCD) Method, as described by Fuchs et al.[17] In terms of ACI 318-11,[1] the breakout strength of an anchor group loaded in direct tension is given by

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ed,N} \cdot \psi_{c,N} \cdot \psi_{cp,N} \cdot \psi_{ec,N} \cdot N_b \tag{7}$$

The term $\psi_{ec,N}$ is related to anchor groups loaded in eccentric tension. As it is assumed that for the purposes of post-installed reinforcing bars, the eccentricity of the tension is zero, this term is neglected.

The term $\psi_{c,N}$ accounts for the anticipated condition of the concrete in the vicinity of the anchor (cracked/uncracked) over the anchor service life. This is typically accounted for in the efficiency factor, k_c, value used for design (refer to Eq. (9)). The term $\psi_{cp,N}$ is a unique modifier related to post-installed anchors that accounts for localized hoop stresses generated (primarily) by expansion anchors and other anchors that must be tightened with a torque wrench as part of the installation. A detailed description of these terms may be obtained from ACI 318-11,[1] Appendix D. As neither of these terms is uniquely relevant to the post-installed reinforcing bar case, they are not discussed further herein.

The quantity A_{Nc} is the actual projected concrete failure area of a single anchor or group of anchors. This failure area may be affected (reduced) by the proximity of the anchor or anchor group to the edge of the concrete element to which the anchors are attached. The quantity A_{Nc} is the theoretical projected failure area for a single anchor that is located so far from the edge that the failure is not influenced by the proximity to an edge. For a single anchor, the ratio A_{Nc}/A_{Nco} will always be less than or equal to 1.0. For a group of closely spaced anchors away from an edge, the ratio will be less than or equal to the number of anchors in the group.

The critical dimensions (spacing and edge distance beyond which the anchor resistance is not influenced) used for the evaluation of A_{Nc} and A_{Nco} are $3h_{ef}$ and $1.5h_{ef}$, respectively, corresponding roughly to a breakout surface with an angle of inclination to the horizontal of 35 degrees. For example, if a single anchor is located a distance further than $1.5h_{ef}$ from any edge, the ratio A_{Nc}/A_{Nco} is unity.

The term $\psi_{ed,N}$ is an additional modifier for near-edge anchors that reflects the disturbed stress state caused by the presence of the edge. It is given in ACI 318-11[1] as follows

$$\psi_{ed,N} = 0.7 + 0.3 \cdot \frac{c_{a,min}}{1.5h_{ef}} \leq 1.0 \tag{8}$$

where $c_{a,min}$ is the edge distance; and h_{ef} is the anchor embedment. As with the ratio A_{Nc}/A_{Nco}, the term is taken as unity when the anchor is no closer than $1.5h_{ef}$ from any edge.

The basic breakout strength of a single anchor far from the edge, N_b, is

$$N_b = k_c \cdot \lambda \cdot \sqrt{f_c'} \cdot h_{ef}^{1.5} \text{ (lb, in.)} \tag{9}$$

where the effectiveness factor k_c for cracked concrete is based on product-specific testing but is limited to a maximum of 24; λ is the lightweight concrete adjustment factor; f_c' is the concrete compressive strength; and h_{ef} is the anchor embedment depth. The adjustment factor for lightweight materials is specified in ACI 318-11,[1] Section 8.6, and is the same as used in development length calculations.

It is important to note that the term λ in Eq. (9) has been replaced by λ_a in ACI 318-11[1], where λ_a is taken as 0.8λ for adhesive anchor concrete failure and 0.6λ for adhesive anchor bond failure. For typical lightweight concrete, these reductions become 0.68 and 0.51, respectively.

ACI 318-11,[1] Appendix D, bond strength provisions

ACI 318-11[1] restricts the maximum allowable adhesive anchor embedment to 20 times the anchor diameter. These restrictions represent the limits over which the bond model was verified both experimentally and analytically.[18]

As discussed previously, whereas pullout strength is not relevant for the design of adhesive anchors, ACI 318-11[1] provides a procedure for computing the bond strength associated with a single adhesive anchor or group of anchors, considering both edge distance and anchor spacing. The expressions for computing the bond capacity are based on the uniform bond model presented by Eligehausen et al.[14] and are very similar in appearance to those used for computing concrete breakout capacity. For a group of anchors, the bond strength is given as

$$N_{ag} = \frac{A_{Na}}{A_{Nao}} \cdot \psi_{ec,Na} \cdot \psi_{ed,Na} \cdot \psi_{cp,Na} \cdot N_{ba} \tag{10}$$

As with the expressions for concrete breakout, the terms A_{Na} and A_{Nao} represent the projected area of the failure surface of the anchor group and a single anchor, respectively. The difference here is that the critical spacing (or edge distance) on which these terms is evaluated is not a function of the anchor embedment but instead is rather dependent on the anchor diameter and the design bond stress for uncracked concrete. This critical spacing is given as

$$c_{Na} = 10 \cdot d_a \cdot \sqrt{\frac{{}_{uncr}}{1100}} \text{ (lb, in.)} \tag{11}$$

For τ_{uncr} in the range of 2000 psi (14 MPa), the critical spacing is approximately $13.5d_a$. As noted previously, the equivalent term for concrete breakout is $1.5h_{ef}$.

The term $\psi_{cp,Na}$ is generally not relevant for the post-installed reinforcing bar case and is not discussed further herein. For additional information on this factor, refer to ACI 318-11.[1] The multiplier $\psi_{ed,Na}$ is relevant when the anchor is closer than c_{Na} to an edge and is of the same form as Eq. (8) with the term c_{Na} substituted for $1.5h_{ef}$ as follows

$$\psi_{ed,N} = 0.7 + 0.3 \cdot \frac{c_{a,min}}{c_{Na}} \leq 1.0 \tag{12}$$

According to the uniform bond model, the basic bond strength of a single anchor in cracked concrete is given by

$$N_{ba} = \tau_{cr} \cdot \lambda_a \cdot \pi \cdot d_a \cdot h_{ef} \tag{13}$$

where τ_{cr} is the design bond stress evaluated from tests on anchors in cracks. In the case of anchors in concrete that will remain uncracked at service load levels, this term may be replaced by the τ_{uncr} design bond stress evaluated from tests in uncracked concrete.

The bond strength is based on test methods and statistical analysis procedures that are described in ACI 355.4-11.[2] Values for the design bond stress, as given in evaluation reports, are dependent on a number of factors. These include expected concrete temperature extremes over the anchor service life as well as installation conditions (for example, damp or dry concrete) and bar size. Tests for the response to sustained tension loading at elevated temperature are also reflected in the design bond stress values.

For cases where adhesive anchors are subjected to sustained tension loads, a supplementary check on the resistance of the most-loaded anchor for a reduced design bond stress is required. The reduction on the design bond stress specified in ACI 318-11[1] is 0.55. This check is required over and above the extensive testing for sustained loads and the corresponding reductions in design bond stress.

APPLICATION TO POST-INSTALLED REINFORCING BARS

The procedures for adhesive anchors contained in ACI 318-11[1] are clearly applicable to the design of anchors consisting of deformed reinforcing and, as indicated, design bond stress values are currently available via evaluation reports issued by ICC-ES. These bond strength values are used exclusively in the assessment of the bond strength capacity of a single anchor or anchor group. With the publication of ACI 318-11[1] and ACI 355.4-11,[2] a design methodology is in place for post-installed reinforcing bars used—for example, as anchor elements in the application shown in Fig. 1.

Considering the applications shown in Fig. 2(a) and (b), while ACI 318-11,[1] Chapter 12, does not apply to post-installed reinforcement, it is conceivable that the development length and tension splice design procedures of Chapter 12 be used if it could be shown that the bond strengths associated with post-installed reinforcing bars are comparable to those shown in Table 1 for cast-in reinforcing bars.

The basic requirements for hole drilling, hole cleaning, installation, and curing are the same for post-installed reinforcing bars as for adhesive anchors. These requirements must be extended to hole depths substantially larger than 20 bar diameters through suitable testing and assessment of the adhesive anchor system. Such an assessment must include verification that deep holes can be drilled close to edges and can be adequately cleaned. In addition, it must be verified that the adhesive can be properly placed in these longer and often larger-diameter holes. Finally, the assessment must consider the effect of large diameter and bond length on the design bond stress and whether the stiffness of the adhesive is suitable to ensure force transfer over a sufficient bond length to avoid zipper-type failure of the embedment.

Other issues that arise in the consideration of what bond stresses to use for post-installed reinforcing bars include:

1. Whether the substantial reductions in the design bond stress applied to post-installed adhesive anchors subject to sustained tension loading are applicable to post-installed reinforcing bars;
2. Whether bond stresses resulting from testing of anchors in cracks are applicable;
3. Whether a strength reduction factor should be applied to the bond stress; and
4. Whether an overstrength factor on bar yield is required to achieve development in the context of ACI 318-11[1] development length provisions.

The assumptions related to bond strength and bar yield are discussed further in the following section.

Bond strength determination

The selection of the appropriate bond strength for establishing the development length of post-installed reinforcing bars will depend on several factors. One of the most decisive factors is whether the concrete is considered to be cracked or uncracked. For anchor design, the assessment of whether to use cracked or uncracked concrete strengths is determined by an evaluation of the likelihood that the concrete in the anchor vicinity will crack over the anchor service life. For post-installed reinforcing bar connections, these assumptions are less likely to be applicable. The use of cracked or uncracked concrete bond stress values may also be dictated by the specifics of the application. For example, it may be appropriate to use a cracked concrete bond stress value (and cracked k_c value) for the placement of reinforcing dowels in an existing shear wall as part of work to add a new (onlay) shear wall because the performance of the dowels could be materially affected by shear cracking in the existing wall under earthquake conditions. Other factors that may affect the bond stress, such as temperature, may also be less relevant for longer and larger post-installed reinforcing bar applications. Nevertheless, these should be considered in the context of the specific application as well. Finally, use of the relatively conservative anchor-based strength reduction factors (the aforementioned Issue 3) offsets the need to consider a wider range of influencing factors.

Assumptions for reinforcing bar yield

As noted in the previous section on development length calculations—cast-in reinforcing bars, the development length provisions of ACI 318-11[1] are intended to result in yielding of the bar prior to failure, either due to splitting or pullout. The degree to which overstrength of the reinforcing bar is taken into account is not clear. For the purposes of establishing yield in a bar designed using anchor concepts, it seems reasonable to use the common assumption of 125% of the nominal yield value.[19]

RECOMMENDED DESIGN METHODOLOGY

On the basis of the previous discussion, a recommended design methodology is provided in the following sections.

Design basis

For the design of single anchors far from edges where the strength is governed by a steel limit state (for example, yield); concrete breakout; or bond failure, the following expressions apply

$$N_{steel} = \phi_s \cdot A_s \cdot f_s \tag{14}$$

$$N_{breakout} = \phi_{concrete} \cdot k \cdot \sqrt{f_c'} \cdot h_{ef}^{1.5} \tag{15}$$

$$N_{bond} = \phi_{concrete} \cdot \tau \cdot \pi \cdot d_b \cdot h_{ef} \tag{16}$$

To establish an embedded length for which the steel strength controls, Eq. (14) must be set equal to both Eq. (15) and (16) and in each case solved for h_{ef}, whereby the larger value of the two cases controls

$$\phi_s \cdot A_s \cdot f_s = \phi_{concrete} \cdot k \cdot \sqrt{f_c'} \cdot h_{ef}^{1.5}$$

giving

$$h_{ef} = \left(\frac{\phi_s \cdot A_s \cdot f_s}{\phi_{concrete} \cdot k \cdot \sqrt{f_c'}} \right)^{2/3} = \left(\frac{\phi_s}{\phi_{concrete}} \right)^{2/3} \left(\frac{A_s \cdot f_s}{k \cdot \sqrt{f_c'}} \right)^{2/3} \tag{17}$$

or

$$\phi_s \cdot A_s \cdot f_s = \phi_{concrete} \cdot \tau \cdot \pi \cdot d_b \cdot h_{ef}$$

giving

$$h_{ef} = \frac{\phi_s \cdot A_s \cdot f_s}{\phi_{concrete} \cdot \tau \cdot \pi \cdot d_b} = \left(\frac{\phi_s}{\phi_{concrete}} \right) \left(\frac{A_s \cdot f_s}{\tau \cdot \pi \cdot d_b} \right) \tag{18}$$

In general, the ratio of strength reduction factors for these two cases will exceed 1.0 and can range between 1.67 to 1.00, depending on the classification of the steel as ductile or brittle (in accordance with ACI 318-11,[1] Appendix D) and the reliability of the anchor system being considered. If one assumes a middle value of 1.3, Eq. (17) and (18) reduce to the following

For concrete breakout:

$$h_{ef} = \left(\frac{1.3 \cdot A_s \cdot f_s}{k \cdot \sqrt{f_c'}} \right)^{2/3} = 1.2 \left(\frac{A_s \cdot f_s}{k \cdot \sqrt{f_c'}} \right)^{2/3} \tag{19}$$

For bond: $$h_{ef} = \frac{1.3 \cdot A_s \cdot f_s}{\tau \cdot \pi \cdot d_b} = \frac{0.3 \cdot d_b \cdot f_s}{\tau} \tag{20}$$

Alternately, recognizing that the nominal concrete resistances are calculated as 5% fractile values (that is, values that will be exceeded by 95% of the population with a 90% confidence level), one can set all strength reduction factors equal to 1.0—that is, use nominal strengths to determine required development length and consider instead an overstrength factor of 125% on the steel limit state (yield).

Recommended design assumptions

As mentioned previously, an assessment procedure for post-installed reinforcing bar applications is needed. Lacking this, some simplifying assumptions are necessary to proceed with the design of post-installed reinforcing bars:

1. Bond stresses may conservatively be taken as those associated with cracked concrete.
2. Assume reinforcing steel to be developed when the force in the bar corresponds to 125% of specified yield.
3. Strength reduction factors (ϕ factors) on bond strength and concrete breakout failure as for anchor applications may be neglected in the determination of development length.
4. Reductions on bond strength for sustained load are not necessary for the general case. For specific cases—for example, where a small number of bars are subjected to direct tension as a result of dead loads—use of the reduced bond stress check may be appropriate.
5. Reductions on bond strength for concrete temperature; presence of water (for example, saturated concrete); and installation in lightweight concrete should be employed as appropriate for the application.
6. For cases where the force in the post-installed bar is being transferred to existing reinforcing, the development length requirement for the existing reinforcing should be satisfied in any case. That is, where the calculated development length for the post-installed bar is less than that required by the code for the cast-in bar that it is assumed to transfer load to, the splice length is dictated by the cast-in bar requirement. Where the splice is assumed to occur between bars of different sizes, the provisions of ACI 318-11[1] require that the larger of the development length of the larger bar and the tension splice length of the smaller bar be used, whereby the tension splice length may be 130% of the development length, depending on the percentage of reinforcing being spliced at a given section.

Suggested design procedure

The use of post-installed reinforcing bars can be grouped into three broad cases:

- Case I: Bars installed in near-edge conditions, typically to transfer load directly to existing reinforcing bars (that is, lap-spliced) in walls, slabs, and columns.
- Case II: Bars installed away from edges (relative to their embedment length) in the face of walls, slabs, and columns.
- Case III: Applications that lie somewhere between Cases I and II.

Case I applications are assumed to be dictated by splitting and, as such, should be addressed using the provisions of ACI 318-11,[1] Chapter 12. These cases almost uniformly involve splicing of new reinforcing to existing. Critical issues are the effective drilling of long holes at close edge distances without damage to the surrounding concrete and accurate location of existing reinforcing. It is noted that hole depths will almost certainly exceed the 20-bar-diameter limit allowed for adhesive anchors.

Case II applications are assumed to occur far enough from edges that splitting will not control the behavior and the concrete breakout and bond resistance is likewise unaffected by the edge. In such cases, the use of the design paradigm developed for adhesive anchors is appropriate, provided that sufficiently conservative assumptions are employed. A subset of Case II occurs in members of limited depth—that is, where cast-in reinforcing would ordinarily be provided with a hook. This is discussed further in the following. Note that Case II conditions would be applicable only when a single bar is placed further from any edge that the greater of $1.5h_{ef}$ (for breakout) and c_{Na} (for bond). Anchors placed closer to an edge than this would be subject to reductions associated with the area ratio and edge distance reduction factors ($\psi_{ed,N}$, $\psi_{ed,Na}$).

Case III applications are those for which a clear distinction between the use of Chapter 12 development length provisions and Appendix D anchor provisions of ACI 318-11[1] is not identifiable. Where using anchor theory to determine embedment, iteration is required because the area ratios and edge distance multiplier for breakout capacity (refer to Eq. (7)) is a function of the embedment.

In all cases where anchor provisions apply, it is appropriate to calculate the required development length both ways and use the lesser value.

Case I design—Step 1: Select an adhesive anchor system that offers design bond stresses equal to or exceeding the values back-calculated from the development length that would be associated with a cast-in bar in the same condition (refer to Table 1). The stiffness of the adhesive should also be checked to ensure that it falls within the normal range for epoxies and other common structural adhesives. Elevated adhesive stiffness could result in zipper-type failure of the tension-loaded bar. The selected adhesive anchor system should also offer a delivery system that is appropriate for the diameter and length of the hole required and that will permit the successful installation of the bar prior to expiration of the gel time.

Step 2: Determine the required edge distance to prevent damage to the surrounding concrete during drilling. This is associated with the drilling method used, the condition of the existing concrete, the presence of existing reinforcing steel, and the diameter and length of the hole required. For questionable cases, consultation with a drilling equipment supplier or specialty contractor is advisable.

Step 3: Determine the development length/splice length in accordance with ACI 318-11,[1] Chapter 12.

Case II design—Step 1: From the selected adhesive anchor system, determine the design bond stresses and k factors for establishment of the required development length.

Step 2: Calculate the required development length as required to satisfy the following

$$N_c \geq 1.25 \cdot A_b \cdot f_y \tag{21}$$

where f_y is the specified bar yield strength; and N_c represents the limiting strength associated with concrete breakout and bond failure evaluated from Eq. (7) and (10).

For the special case of a single bar, the ratio of areas terms reduce to 1. Solving Eq. (21) for the embedment depth and taking this as the development length in each case, these expressions can be simplified for concrete breakout and bond failure, respectively, as follows (compare Eq. (19) and (20))

$$\ell_d = 1.2\left[\frac{A_b \cdot f_y}{k_{cr} \cdot \sqrt{f_c'}}\right]^{2/3} \tag{22}$$

$$\ell_d = \frac{0.3 \cdot d_b \cdot f_y}{\tau_{cr}} \tag{23}$$

For a single Grade 60 No. 8 (No. 25 [25.4 mm]) bar to be installed in 5000 psi (34.4 MPa) concrete away from edges and assuming a value of 17 for k_{cr} and 900 psi (6.2 MPa) for $\tau_{k,cr}$, these expressions yield development lengths of 15 and 20 in. (381 and 508 mm) for concrete breakout and bond failure, respectively. For the controlling embedment of 20 in. (508 mm) to apply, these bars need to be further than the greater of $1.5h_{ef}$ ($=1.5\ell_d$) or c_{Na} from any edge. For this case, the controlling edge distance is approximately 30 in. (762 mm). Note also that the required development length of 20 in. (381 mm) corresponds to the upper limit of 20 bar diameters allowed in ACI 318-11,[1] Appendix D. For cases where greater development lengths are indicated, it may be appropriate to use a reduced value for the uniform bond strength to account for the uncertainties associated with application of the model to deeper embedments.

For all cases, it is conservative to use the development/splice length calculated in accordance with the provisions of ACI 318-11,[1] Chapter 12. For the bar in this example, taking the confinement term as 2.5 and all other modification factors as 1.0, the required development length per Eq. (1) is 28.5 in. (724 mm).

Case III design—Case III designs proceed essentially as for Case II, except that for cases where the edge proximity could affect the breakout and bond strength determination, the inclusion of the ratio of area terms and the edge modification terms ($\psi_{ed,N}$, $\psi_{ed,Na}$) is required. This substantially complicates the establishment of a closed-form solution for the development length. Furthermore, as the edge distance gets smaller, it is possible that the required development length, as dictated by concrete breakout or bond failure, may exceed that associated with splitting, as per the development length equations of ACI 318-11,[1] Chapter 12. In this respect, it is necessary to recognize when the use of the ACI 318-11,[1] Appendix D, expressions is no longer rational in terms of the anticipated failure mode. For most cases, edge distances less than six bar diameters should be evaluated solely in terms of the development length provisions of the code.

Special cases

As discussed previously, for cases where cast-in reinforcing would ordinarily be provided with a hook, assessment of the force path in terms of using a strut-and-tie model may be advisable. This case is addressed in detail in Hamad et al.[20]

The use of such strut-and-tie models is in accordance with the provisions established in ACI 318-11,[1] Appendix A.

RESEARCH NEEDS

Because many adhesives are capable of developing significantly higher bond strengths than those associated with cast-in bars, it is conceivable that the development length requirements of the code could be used for cases where sufficient transverse reinforcement is present to resist the higher splitting stresses. In this regard, previous investigations by Spieth[5] indicate the need to consider the stiffness of the bond mechanism associated with specific adhesive types to avoid zipper failures. Further work is needed, however, to determine the range of stiffness values that are acceptable for the total range of bar sizes and potential development lengths.

The applicability of the uniform bond stress assumption for adhesive anchors needs to be validated for embedments greater than 20 bar diameters.

The response of adhesively bonded reinforcing bars to fire exposure is not well-understood and there are no design procedures in place for such cases, nor are there accepted procedures of assessing the post-fire viability of adhesively bonded bars.

CONCLUSIONS

Despite their widespread use in construction, post-installed reinforcing bars are not currently addressed in U.S. building standards. Qualification standards for adhesive anchor systems to determine their appropriateness for such uses and design methodologies suited to the anticipated failure modes are both lacking. While it is probable that these issues will be addressed by ACI or another standards body at some future time, in the interim, a practical approach to the design problem that recognizes the assumptions inherent in the provisions for anchorage and development length in ACI 318 is offered. As with all design problems, engineering judgment is required in the application of this approach to the broad class of details associated with post-installed reinforcing bars.

REFERENCES

1. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary," American Concrete Institute, Farmington Hills, MI, 2011, 503 pp.

2. ACI Committee 355, "Qualification of Post-Installed Adhesive Anchors in Concrete (ACI 355.4-11) and Commentary," American Concrete Institute, Farmington Hills, MI, 2011, 55 pp.

3. ICC Evaluation Service Inc., "AC308—Acceptance Criteria for Post-Installed Adhesive Anchors in Concrete Elements," International Code Council, Whittier, CA, 2009, 122 pp.

4. TR023, "Assessment of Post-Installed Rebar Connections," *ETA Technical Report*, European Organisation for Technical Approvals (EOTA), Brussels, Belgium, 2006, 20 pp.

5. Spieth, H., "Structural Behavior and Design of Rebar (Tragverhalten und Bemessung von Eingemörtelten Bewehrungsstäben)," doctoral thesis, University of Stuttgart, Stuttgart, Germany, 2002, 285 pp. (in German)

6. Kunz, J., and Münger, F., "Splitting and Bond Failure of Post-Installed Rebar Splices and Anchorings," *Bond in Concrete—From Research to Standards*, Budapest, Hungary, 2002, pp. 447-454.

7. Simons, I., "Bond Behavior of Rebar Rods under Cyclic Loading (Verbundverhalten von Bewehrungsstahl Stäbe unter Zyklischer Belastung)," doctoral thesis, University of Stuttgart, Stuttgart, Germany, 2007, 246 pp. (in German)

8. Simons, I., and Eligehausen, R., "Structural Behavior and Design of Rebar Rods under Cyclic Loading in Uncracked and Cracked Concrete (Tragverhalten und Bemessung von eingemörtelten Bewehrungsstäben unter zyklischer Beanspruchung im ungerissenen und gerissenen Beton)," *Beton- und Stahlbetonbau*, V. 103, 2008, pp. 598-608. (in German)

9. Appl, J., "Structural Behavior of Composite Anchors under Tensile Load (Tragverhalten von Verbunddübeln unter Zugbelastung)," doctoral thesis, University of Stuttgart, Stuttgart, Germany, 2008, 252 pp. (in German)

10. Spieth, H.; Ozbolt, J.; Eligehausen, R.; and Appl, J., "Numerical and Experimental Analysis of Post-Installed Rebars Spliced with Cast-In-Place Rebars," *International Symposium on Connections between Steel and Concrete*, RILEM Publications, Stuttgart, Germany, 2001, pp. 889-898.

11. Eligehausen, R.; Mallée, R.; and Silva, J., *Anchorage in Concrete Construction*, Ernst & Sohn, Berlin, Germany, 2006, pp. 181-210.

12. Cook, R., and Konz, R., "Factors Influencing Bond Strength of Adhesive Anchors," *ACI Structural Journal*, V. 98, No. 1, Jan.-Feb. 2001, pp. 76-86.

13. Meszaros, J., and Eligehausen, R., "Load Bearing Behavior and Design of Single Adhesive Anchors," *Proceedings of the International Symposium on Connections between Steel and Concrete*, RILEM Publications, Cachan, France, 2001, pp. 422-432.

14. Eligehausen, R.; Cook, R.; and Appl, J., "Behavior and Design of Adhesive Bonded Anchors," *ACI Structural Journal*, V. 103, No. 6, Nov.-Dec. 2006, pp. 822-831.

15. Orangun, C.; Jirsa, J.; and Breen, J., "The Strength of Anchored Bars: A Reevaluation of Test Data on Development Length and Splices," *Research Report 154-3F*, Center for Highway Research, University of Texas at Austin, Austin, TX, 1975, 94 pp.

16. Joint ACI-ASCE Committee 408, "Bond and Development of Straight Reinforcing Bars in Tension (ACI 408R-03) (Reapproved 2012)," American Concrete Institute, Farmington Hills, MI, 2003, 49 pp.

17. Fuchs, W.; Eligehausen, R.; and Breen, J., "Concrete Capacity Design (CCD) Approach for Fastening to Concrete," *ACI Structural Journal*, V. 92, No. 1, Jan.-Feb. 1995, pp. 73-94.

18. Cook, R.; Kunz, J.; Fuchs, W.; and Konz, R., "Behavior and Design of Single Adhesive Anchors under Tensile Load in Uncracked Concrete," *ACI Structural Journal*, V. 95, No. 1, Jan.-Feb. 1998, pp. 9-26.

19. Joint ACI-ASCE Committee 352, "Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures (ACI 352R-02) (Reapproved 2010)," American Concrete Institute, Farmington Hills, MI, 2002, 38 pp.

20. Hamad, B.; Hammoud, R.; and Kunz, J., "Evaluation of Bond Strength of Bonded-In or Post-Installed Reinforcement," *ACI Structural Journal*, V. 103, No. 2, Mar.-Apr. 2006, pp. 207-218.

APPENDIX A. SI EQUIVALENTS OF EQUATIONS

$$\ell_d = \left[\frac{1}{1.1} \frac{f_y}{\lambda \sqrt{f_c^{'}}} \frac{\psi_t \psi_e \psi_s}{\frac{c_b + K_{tr}}{d_b}} \right] d_b \tag{1}$$

$$\tau_{eq} = 40.1 \cdot \lambda \cdot \sqrt{f_c^{'}} \cdot \left(\frac{c_b + K_{tr}}{d_b} \right) \left(\frac{1}{\psi_t \cdot \psi_e \cdot \psi_s} \right) \tag{3}$$

$$\tau_{eq} = 50.1 \cdot \lambda \cdot \sqrt{f_c^{'}} \cdot \left(\frac{c_b + K_{tr}}{d_b} \right) \le 125.3 \cdot \sqrt{f_c^{'}} \tag{4a}$$

$$\tau_{eq} = 40.1 \cdot \lambda \cdot \sqrt{f_c^{'}} \cdot \left(\frac{c_b + K_{tr}}{d_b} \right) \le 100.3 \cdot \sqrt{f_c^{'}} \tag{4b}$$

$$N_b = 0.42 k_c \cdot \lambda \cdot \sqrt{f_c^{'}} \cdot h_{ef}^{1.5} \tag{9}$$

$$c_{Na} = 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{7.59}} \tag{11}$$

$$N_{breakout} = 0.42 \phi_{concrete} \cdot k \cdot \sqrt{f_c^{'}} \cdot h_{ef}^{\ 1.5} \tag{15}$$

$$h_{ef} = \left(\frac{\phi_s \cdot A_s \cdot f_s}{\phi_{concrete} \cdot k \cdot 0.42 \sqrt{f_c^{'}}} \right)^{2/3} = \left(\frac{\phi_s}{\phi_{concrete}} \right)^{2/3} \left(\frac{A_s \cdot f_s}{k \cdot 0.42 \sqrt{f_c^{'}}} \right)^{2/3} \tag{17}$$

$$h_{ef} = \left(\frac{1.3 \cdot A_s \cdot f_s}{k \cdot 0.42 \sqrt{f_c^{'}}} \right)^{2/3} = 1.2 \left(\frac{A_s \cdot f_s}{k \cdot 0.42 \sqrt{f_c^{'}}} \right)^{2/3} \tag{19}$$