

Bond and Development of Straight Reinforcing Bars in Tension

Reported by ACI Committee 408

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The performance of reinforced concrete structures depends on adequate bond strength between concrete and reinforcing steel. This report describes bond and development of straight reinforcing bars under tensile load. Bond behavior and the factors affecting bond are discussed, including concrete cover and bar spacing, bar size, transverse reinforcement, bar geometry, concrete properties, steel stress and yield strength, bar surface condition, bar casting position, development and splice length, distance between spliced bars, and concrete consolidation. Descriptive equations and design provisions for development and splice strength are presented and compared using a large database of test results. The contents of the database are summarized, and a protocol for bond tests is presented.

Test data and reliability analyses demonstrate that, for compressive strengths up to at least 16,000 psi (110 MPa), the contribution of concrete strength to bond is best represented by the compressive strength to the 1/4 power, while the contribution of concrete to the added bond strength provided by transverse reinforcement is best represented by compressive

strength to a power between 3/4 and 1.0. The lower value is used in proposed design equations. These values are in contrast with the square root of compressive strength, which normally is used in both descriptive and design expressions. Provisions for bond in ACI 318-02 are shown to be unconservative in some instances; specifically, the 0.8 bar size factor for smaller bars should not be used and a ϕ -factor for bond is needed to provide a consistent level of reliability against bond failure. Descriptive equations and design procedures developed by Committee 408 that provide improved levels of reliability, safety, and economy are presented. The ACI Committee 408 design procedures do not require the use of the 1.3 factor for Class B splices that is required by ACI 318.

Keywords: anchorage; bond; concrete; deformed reinforcement; development length; reinforced concrete; reinforcement; relative rib area; splice; stirrup; tie.

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The bond between reinforcing bars and concrete has been acknowledged as a key to the proper performance of reinforced concrete structures for well over 100 years (Hyatt 1877). Much research has been performed during the intervening years, providing an ever-improving understanding of this aspect of reinforced concrete behavior. ACI Committee 408 issued its first report on the subject in 1966. The report emphasized key aspects of bond that are now well understood by the design community but that, at the time, represented conceptually new ways of looking at bond strength. The report emphasized the importance of splitting cracks in governing bond strength and the fact that bond forces did not vary monotonically and could even change direction in regions subjected to constant or smoothly varying moment. Committee 408 followed up in 1979 with suggested provisions for development, splice, and hook design (ACI 408.1R-79), in 1992 with a state-of-the-art report on bond under cyclic loads (ACI 408.2R-92), and in 2001 with design

provisions for splice and development design for high relative rib area bars (bars with improved bond characteristics) (ACI 408.3-01). This report represents the next in that line, emphasizing bond behavior and design of straight reinforcing bars that are placed in tension.

For many years, bond strength was represented in terms of the shear stress at the interface between the reinforcing bar and the concrete, effectively treating bond as a material property. It is now clear that bond, anchorage, development, and splice strength are structural properties, dependent not only on the materials but also on the geometry of the reinforcing bar and the structural member itself. The knowledge base on bond remains primarily empirical, as do the descriptive equations and design provisions. An understanding of the empirical behavior, however, is critical to the eventual development of rational analysis and design techniques.

Test results for bond specimens invariably exhibit large scatter. This scatter increases as the test results from different laboratories are compared. Research since 1990 indicates that much of the scatter is the result of differences in concrete material properties, such as fracture energy and reinforcing bar geometry, factors not normally considered in design. This report provides a summary of the current state of knowledge of the factors affecting the tensile bond strength of straight reinforcing bars, as well as realistic descriptions of development and splice strength as a function of these factors. The report covers bond under the loading conditions that are addressed in Chapter 12 of ACI 318; dynamic, blast, and seismic loading are not covered.

Chapter 1 provides an overview of bond behavior, including bond forces, test specimens, and details of bond response. Chapter 2 covers the factors that affect bond, discussing the impact of structural characteristics as well as bar and concrete properties. The chapter provides insight not only into aspects that are normally considered in structural design, but into a broad range of factors that control anchorage, development, and splice strength in reinforced concrete members. Chapter 3 presents a number of widely cited descriptive equations for development and splice strength, including expressions recently developed by ACI Committee 408. The expressions are compared for accuracy using the test results in the ACI Committee 408 database. Chapter 4 summarizes the design provisions in ACI 318, ACI 408.3, the 1990 CEB-FIP Model Code, as well as design procedures recently developed by Committee 408. The design procedures are compared for accuracy, reliability, safety, and economy using the ACI Committee 408 database. The observations presented in Chapters 3 and 4 demonstrate that $f_c'^{1/4}$ provides a realistic representation of the contribution of concrete strength to bond for values up to at least 16,000 psi (110 MPa), while $f_c'^{3/4}$ does the same for the effect of concrete strength on the increase in bond strength provided by transverse reinforcement. This is in contrast to $\sqrt{f_c'}$, which is used in most design provisions. The comparisons in Chapter 4 also demonstrate the need to modify the design provisions in ACI 318 by removing the bar size g factor of 0.8 for small bars and addressing the negative impact on bond reliability of changing the load factor while maintaining

the strength reduction factor for tension in the transition from ACI 318-99 to ACI 318-02. Design procedures recommended by ACI Committee 408 that provide both additional safety and economy are presented. Chapter 5 describes the ACI Committee 408 database, while Chapter 6 presents a recommended protocol for bond tests. The expressions within the body of the report are presented in inch-pound units. Expressions in SI units are presented in Appendix A.

A few words are appropriate with respect to terminology. The term *bond force* represents the force that tends to move a reinforcing bar parallel to its length with respect to the surrounding concrete. *Bond strength* represents the maximum bond force that may be sustained by a bar. The terms *development strength* and *splice strength* are, respectively, the bond strengths of bars that are not spliced with other bars and of bars that are spliced. The terms *anchored length*, *bonded length*, and *embedded length* are used interchangeably to represent the length of a bar over which bond force acts; in most cases, this is the distance between the point of maximum force in the bar and the end of the bar. Bonded length may refer to the length of a lap splice. *Developed length* and *development length* are used interchangeably to represent the bonded length of a bar that is not spliced with another bar, while *spliced length* and *splice length* are used to represent the bonded length of bars that are lapped spliced. When used in design, development length and splice length are understood to mean the “length of embedded reinforcement required to develop the design strength of reinforcement at a critical section,” as defined in ACI 318.

CHAPTER 1—BOND BEHAVIOR

In reinforced concrete construction, efficient and reliable force transfer between reinforcement and concrete is required for optimal design. The transfer of forces from the reinforcement to the surrounding concrete occurs for a deformed bar (Fig. 1.1) by:

- Chemical adhesion between the bar and the concrete;
- Frictional forces arising from the roughness of the interface, forces transverse to the bar surface, and relative slip between the bar and the surrounding concrete; and
- Mechanical anchorage or bearing of the ribs against the concrete surface.

After initial slip of the bar, most of the force is transferred by bearing. Friction, however, especially between the concrete and the bar deformations (ribs) plays a significant role in force transfer, as demonstrated by epoxy coatings, which lower the coefficient of friction and result in lower bond capacities. Friction also plays an important role for plain bars (that is, with no deformations), with slip-induced friction resulting from transverse stresses at the bar surface caused by small variations in bar shape and minor, though significant, surface roughness. Plain bars with suitably low allowable bond stresses were used for many years for reinforced concrete in North America and are still used in some regions of the world.

When a deformed bar moves with respect to the surrounding concrete, surface adhesion is lost, while bearing

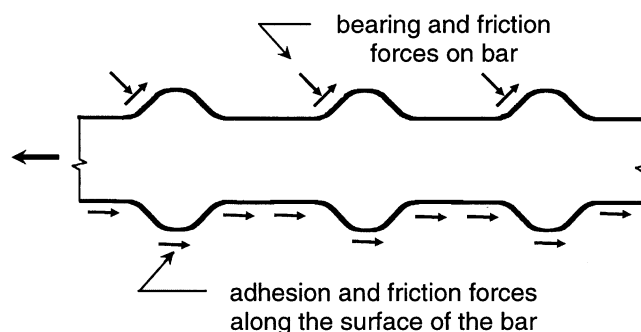


Fig. 1.1—Bond force transfer mechanisms.

forces on the ribs and friction forces on the ribs and barrel of the bar are mobilized. The compressive bearing forces on the ribs increase the value of the friction forces. As slip increases, friction on the barrel of the reinforcing bar is reduced, leaving the forces at the contact faces between the ribs and the surrounding concrete as the principal mechanism of force transfer. The forces on the bar surface are balanced by compressive and shear stresses on the concrete contact surfaces, which are resolved into tensile stresses that can result in cracking in planes that are both perpendicular and parallel to the reinforcement, as shown in Fig. 1.2(a) and 1.2(b). The cracks shown in Fig. 1.2(a), known as Goto (1971) cracks, can result in the formation of a conical failure surface for bars that project from concrete and are placed in tension. They otherwise play only a minor role in the anchorage and development of reinforcement. The transverse cracks shown in Fig. 1.2(b) form if the concrete cover or the spacing between bars is sufficiently small, leading to splitting cracks, as shown in Fig. 1.2(c). If the concrete cover, bar spacing, or transverse reinforcement is sufficient to prevent or delay a splitting failure, the system will fail by shearing along a surface at the top of the ribs around the bars, resulting in a “pullout” failure, as shown in Fig. 1.2(d). It is common, for both splitting and pullout failures, to observe crushed concrete in a region adjacent to the bearing surfaces of some of the deformations. If anchorage to the concrete is adequate, the stress in the reinforcement may become high enough to yield and even strain harden the bar. Tests have demonstrated that bond failures can occur at bar stresses up to the tensile strength of the steel.

From these simple qualitative descriptions, it is possible to say that bond resistance is governed by:

- The mechanical properties of the concrete (associated with tensile and bearing strength);
- The volume of the concrete around the bars (related to concrete cover and bar spacing parameters);
- The presence of confinement in the form of transverse reinforcement, which can delay and control crack propagation;
- The surface condition of the bar; and
- The geometry of the bar (deformation height, spacing, width, and face angle).

A useful parameter describing bar geometry is the so-called relative rib area R_r , illustrated in Fig. 3.5, which is the ratio of the bearing area of the bar deformations to the

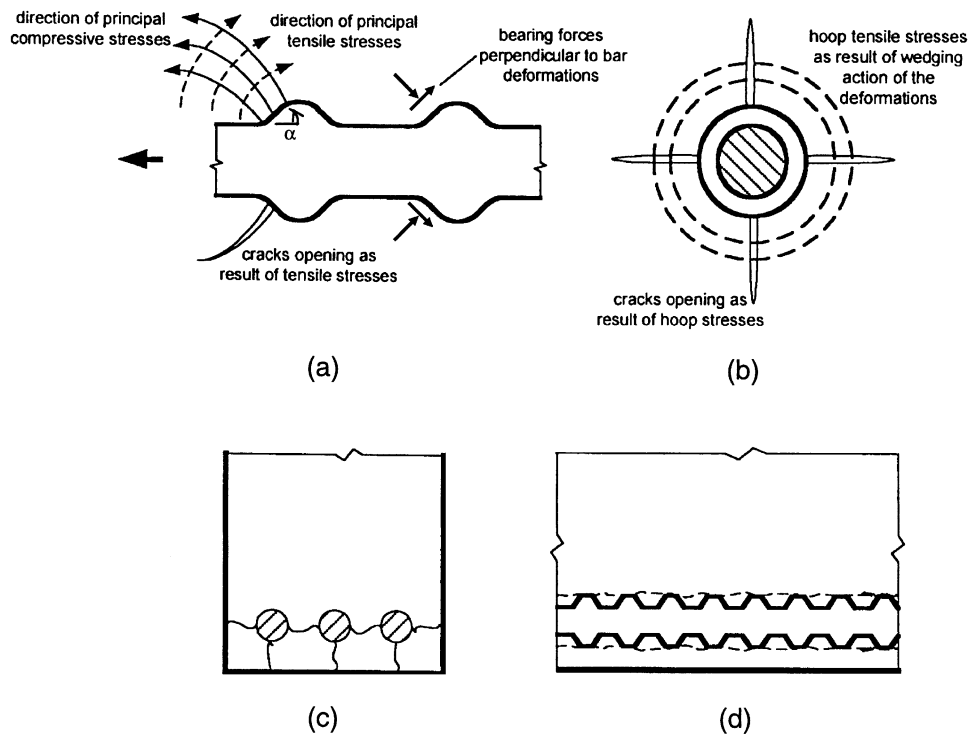


Fig. 1.2—Cracking and damage mechanisms in bond: (a) side view of a deformed bar with deformation face angle α showing formation of Goto (1971) cracks; (b) end view showing formation of splitting cracks parallel to the bar; (c) end view of a member showing splitting cracks between bars and through the concrete cover; and (d) side view of member showing shear crack and/or local concrete crushing due to bar pullout.

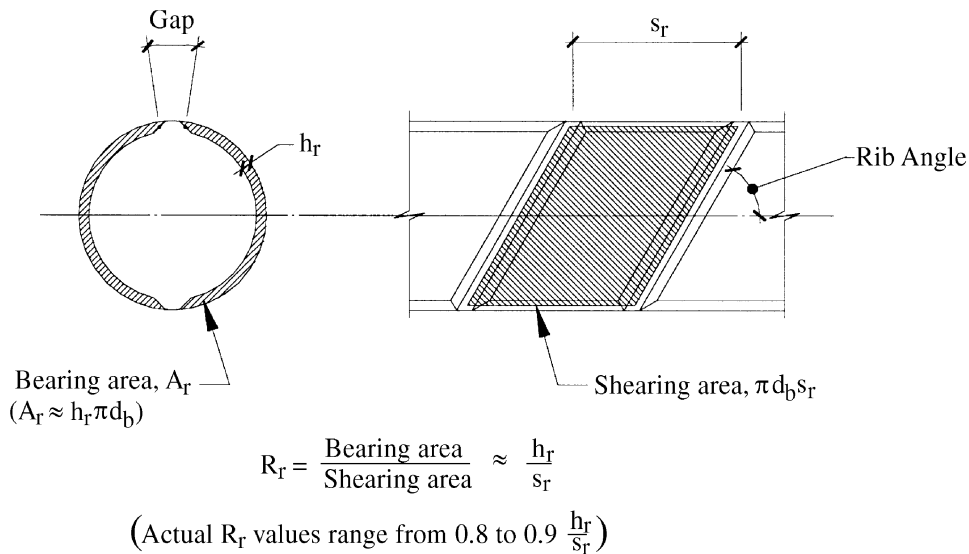


Fig. 1.3—Definition of R_r (ACI 408.3R).

shearing area between the deformations (in U.S. practice, this is taken as the ratio of the bearing area of the ribs to the product of the nominal bar perimeter and the average spacing of the ribs). Relative rib area is discussed at greater length in Section 2.2.2.

1.1—Bond forces—background

To understand the design procedures used for selecting development and splice lengths of reinforcement, it is instructive to review the nature of bond forces and stresses in

a reinforced concrete flexural member. Historically, the difference in tensile force ΔT between two sections located at flexural cracks along a member (Fig. 1.4) was calculated as

$$\Delta T = T_1 - T_2 = \frac{M_1}{jd_1} - \frac{M_2}{jd_2} \quad (1-1)$$

where T_i ($T_2 > T_1$), M_i ($M_2 > M_1$), and jd_i are the tensile force, moment, and internal moment arm at section i ($i = 1, 2$). For

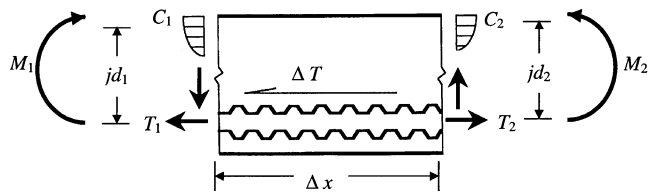


Fig. 1.4—Variation in bar force due to changes in moment in a beam.

an infinitesimally small distance between Sections 1 and 2, Eq. (1-1) becomes

$$dT = \frac{dM}{jd} \quad (1-2)$$

If the bond force per unit length U is defined as the change in tensile force per unit length, then

$$U = \frac{dT}{dl} = \frac{1}{jd} \frac{dM}{dl} \quad (1-3a)$$

$$U = \frac{1}{jd} V \quad (1-3b)$$

where V is the shear on the section.

Equation (1-3b) indicates that, away from concentrated loads, bond forces vary as a function of the applied shear along the length of reinforced concrete flexural members, and for many years, the bond force used in design U was based on this expression. Over time, however, it became apparent that the change in force in reinforcing bars dT does not vary strictly with the change in moment per unit length, as suggested in Eq. (1-3a), but simply with the force in the bar T , which varies from a relatively high value at cracks to a low value between cracks, where the concrete shares the tensile force with the reinforcing steel. Using the definition $U = dT/dl$, bond forces vary significantly along the length of a member, even varying in direction, as shown in Fig. 1.5. The real distribution of bond forces along the length of a bar, therefore, cannot be predicted because it depends on the locations of the flexural cracks and the amount of tensile load carried by the concrete—neither of which can be calculated. Given these facts and because a principal goal of design is to ensure that the bar is adequately anchored so that failure will manifest itself in some way other than in bond, it is both convenient and realistic for design purposes to treat bond forces as if they were uniform over the anchored, developed, or spliced length of the reinforcement.

Until adoption of the 1971 ACI Building Code (ACI 318-71), bond design was based on bond stress u , which is equal to bond force per unit length U divided by the sum of the perimeters of the bars developed at a section Σ_o .

$$u = \frac{U}{\Sigma_o} = \frac{\Delta T}{\Delta l \Sigma_o} = \frac{\Delta f_s A_b}{\Delta l \Sigma_o} = \frac{\Delta f_s d_b}{4 \Delta l} \quad (1-4)$$

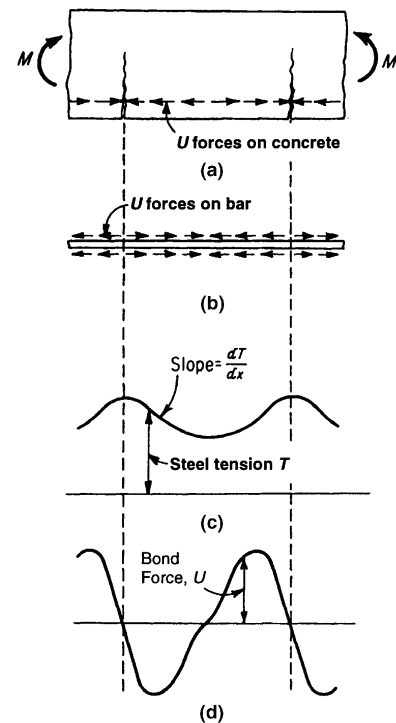


Fig. 1.5—Variation of steel and bond forces in reinforced concrete member subjected to pure bending: (a) cracked concrete segment; (b) bond stresses acting on reinforcing bar; (c) variation of tensile force in steel; and (d) variation of bond force along bar (adapted from Nilson et al. [2004]).

where A_b = area of bars; d_b = diameter of bars; and Δf_s = change in steel stress over length Δl .

For design purposes, the change in stress Δf_s equals the yield stress of the steel f_y and Δl equals the development length l_d . In ACI 318-63, the maximum bond stress was set at *

$$u = 9.5 \frac{\sqrt{f'_c}}{d_b} \leq 800 \text{ psi} \quad (1-5)$$

Substituting Eq. (1-5) into Eq. (1-4), solving for $\Delta l = l_d$, and multiplying the resultant value by 1.2 to account for the reduced bond strength of closely spaced bars (due to the interaction of splitting cracks) gives the development length

$$l_d = 0.04 A_b \frac{f_y}{\sqrt{f'_c}} \quad (1-6)$$

Equation (1-6) was used for design, beginning with ACI 318-71, until a design approach that more closely matched observed behavior was adopted in ACI 318-95.

While convenient, equations for development length [like Eq. (1-6) and some of those presented in Chapter 4] have led many designers to believe that the real force that must be developed is equal to the product of the area and yield

*SI conversions of equations that contain terms that depend on units of measure are presented in Appendix A.

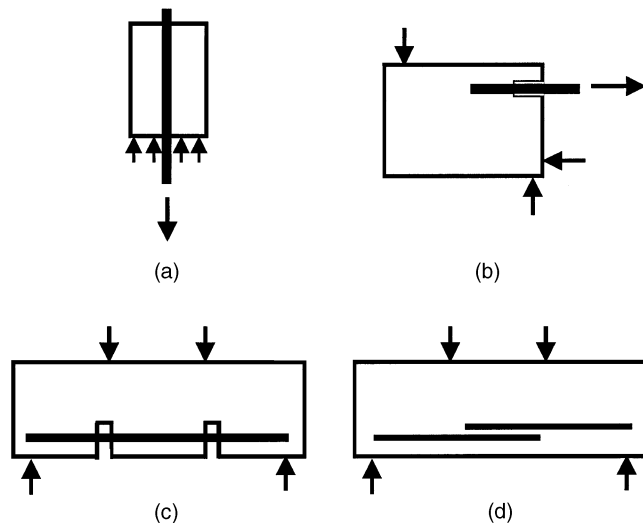


Fig. 1.6—Schematic of: (a) pullout specimen; (b) beam-end specimen; (c) beam anchorage specimen; and (d) splice specimen.

strength of a bar. In fact, the basis for the expressions for development length l_d lies in Eq. (1-3) and (1-4), which are based on the change in bar force ΔT , the result of externally applied load. The force in the bar $A_b f_y$ used in the Eq. (1-6) is the designer-selected value for ΔT . If, for example, a bar in a flexural member has a higher yield strength than specified (the usual case), a longer development length will be needed to ensure that a ductile bending failure will occur before a brittle bond failure.

1.2—Test specimens

A variety of test specimen configurations have been used to study bond between reinforcing bars and concrete. The four most common configurations are shown in Fig. 1.6. The details of the specimen affect not only the measured bond strength, but also the nature of the bond response.

The pullout specimen (Fig. 1.6(a)) is widely used because of its ease of fabrication and the simplicity of the test. The specimen often incorporates transverse reinforcement to limit splitting when the bar is placed in tension. This specimen is the least realistic of the four shown in Fig. 1.6 because the stress fields within the specimen match few cases in actual construction. As the bar is placed in tension, the concrete is placed in compression. Further, compressive struts form between the support points for the concrete and the surface of the reinforcing bar, placing the bar surface in compression. This stress state differs markedly from most reinforced concrete members, in which both the bar and the surrounding concrete are in tension, and the bearing surfaces of the bar ribs are subjected to a compressive force due to relative movement of the bar with respect to the concrete, not due to the basic load application. In cases where bar surface properties (such as epoxy coatings) or bar surface strength (such as for fiber-reinforced polymer reinforcement) are important, the compression developed at the bar surface in the pullout test reduces the applicability of the test results to structural design. Thus, the use of pullout test results as the

sole basis for determining development length is inappropriate and not recommended by Committee 408.

The specimens shown in Fig. 1.6(b) through (d) provide more realistic measures of bond strength in actual structures. The modified cantilever beam, or beam-end specimen, shown in Fig. 1.6(b), provides a relatively simple test that generally duplicates the stress state obtained in reinforced concrete members; the reinforcing steel and the surrounding concrete are simultaneously placed in tension. To achieve the desired stress state, the compressive force must be located away from the reinforcing bar by a distance approximately equal to the embedded or bonded length of the bar within the concrete. To prevent a conical failure surface from forming, a small length of bar near the surface is usually unbonded. A specimen like that shown in Fig. 1.6(b), proportioned to satisfy the spacing requirements between the bar and the compressive force and reinforced to ensure bond rather than flexural or shear failure, is specified in ASTM A 944. The shear reinforcement is placed in the specimen so as not to intercept longitudinal splitting cracks that occur at bond failure. Transverse reinforcement can be added in cases where its effect is of interest (Darwin and Graham 1993a,b). The bond strengths obtained with the test specimen closely match those obtained in other specimens designed to represent full-scale reinforced concrete members.

Beam anchorage and splice specimens shown in Fig. 1.6(c) and (d), respectively, represent larger-scale specimens designed to directly measure development and splice strengths in full-size members. The anchorage specimen simulates a member with a flexural crack and a known bonded length. Based on concern that increased normal stresses at the bar surface, caused by the reactions, may increase bond strength, some anchorage specimens have been designed so that the reactions are displaced laterally from the centerline of the beam. The splice specimen, normally fabricated with the splice in a constant moment region, is easier to fabricate and produces similar bond strengths to those obtained with the anchorage specimen. Because of both its relative simplicity of fabrication and realistic stress-state in the vicinity of the bars, the splice specimen has provided the bulk of the data used to establish the current design provisions for development length, as well as splice length, in ACI 318 (starting with ACI 318-95).

Other specimens have also been used to study bond strength. These include the wall specimen, to determine the “top-bar” effect, and the tension specimen, consisting of a bar surrounded by concrete, with tension applied to both ends of the bar, which project from the concrete. Variations on the beam-end specimen have also been used in which the compressive force is placed relatively close to the bar, resulting in higher bond strengths due to the compressive strut reaching the bar surface. These specimens generally lack the realism obtained with the specimens shown in Fig. 1.6(b) through (d).

1.3—Details of bond response

Bond force-slip and bond stress-slip curves can be used to better understand the nature of bond response. In their

simplest and most widely used form, the curves are based on known bar forces, such as obtained in the beam-end and beam anchorage specimens (Fig. 1.6(b) and (c)). Bar forces are compared with the external slip of the reinforcing bar, measured with respect to the concrete at either the loaded or unloaded end of the bar. Examples of bar force-loaded end and unloaded end slip curves are shown in Fig. 1.7(a) and (b), respectively. The loaded end bond force-slip curve shows a lower initial stiffness than the unloaded end curve. The difference represents the lengthening of the reinforcing bar between the two points of slip measurement.

More detailed information, at a smaller scale along the length of a bar, can be obtained by placing strain gages on the bar as a method to determine changes in bar force ΔT , which can be converted to bond force per unit length, $U \approx \Delta T / \Delta l$, and bar stress, $u = U / \Sigma_o$. In the most detailed studies, the strain gages are installed by splitting the bar in half, forming a small channel along the centerline, installing the strain gages and wires in the channel, and welding the bar back together. As an acceptable alternative, strain gages and wires are placed in longitudinal grooves cut at the location of longitudinal ribs. These types of installation minimize disturbances at the interface between the bar and the concrete.

A bond stress versus slip curve for a bar loaded monotonically and failing by pullout is shown in Fig. 1.8 (Eligehausen, Popov, and Bertero 1983). Bond force and bond stress-slip curves, like bond strength, are structural properties that depend on both the geometry of the bar and the details of the concrete member, including the cover, transverse reinforcement, and state of stress in the concrete surrounding the reinforcement. As shown in Fig. 1.7 and 1.8, bond force and bond stress-slip curves are initially very steep because of adhesion.

Because of concrete shrinkage, which is restrained by the reinforcing bar, small internal cracks exist immediately adjacent to the reinforcing bar. These cracks can act as stress raisers and points of crack initiation at the bar ribs at relatively low loads. Because cracks tend to form in front of the ribs, small splitting cracks may begin to propagate from the ribs, as shown in Fig. 1.2(a). If the reinforcing bar is placed in tension from a free surface, such as a beam-end specimen, it is possible for the crack to propagate to the surface, separating a roughly conical region of concrete from the rest of the specimen.

As loading continues, longitudinal splitting cracks form, as shown in Fig. 1.2(b) and (c), leading to a softening in the bond force-slip curve. In regions where these splitting cracks open and where confinement by transverse reinforcement is limited, the bar may slip with little local damage to the concrete at the contact surface with the bar ribs. In regions of greater confinement, higher rates of loading, or both, the concrete in front of the reinforcing bar ribs may crush as the bar moves, forming effective ribs with a reduced face angle (less than α in Fig. 1.2(a)). For higher values of confinement, all of the concrete between the ribs may crush or a shear crack may form along the periphery of the bar, or both, resulting in a pullout failure (Fig. 1.2(d)). Depending on the details of the member and the loading, bond failure may entail a combination of the failure modes. Transverse reinforcement, however, rarely yields during a bond failure

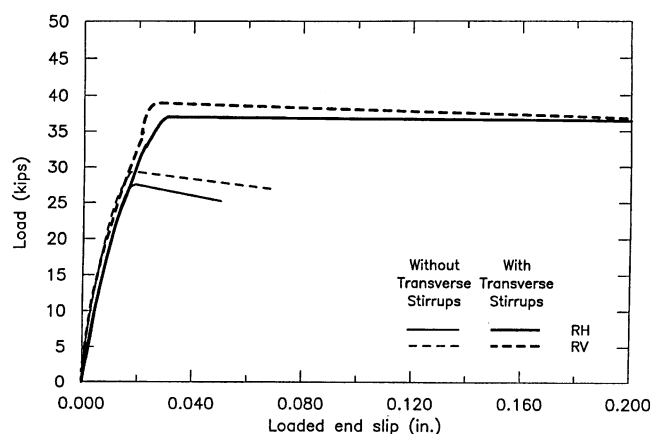


Fig. 1.7(a)—Average load-loaded end slip curves for No. 8 (No. 25) ASTM A 615 reinforcing bars in an ASTM A 944 beam-end specimen. RH and RV represent bars with longitudinal ribs oriented horizontally and vertically, respectively (Darwin and Graham 1993a). (Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN)

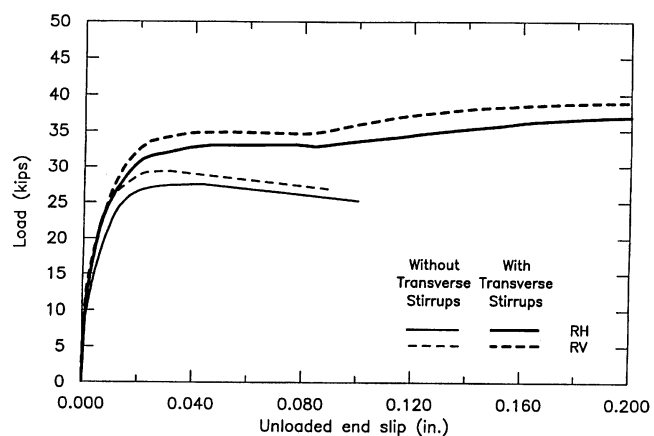


Fig. 1.7(b)—Average load-unloaded end slip curves for No. 8 (No. 25) ASTM A 615 reinforcing bars in an ASTM A 944 beam-end specimen. RH and RV represent bars with longitudinal ribs oriented horizontally and vertically, respectively (Darwin and Graham 1993a). (Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN)

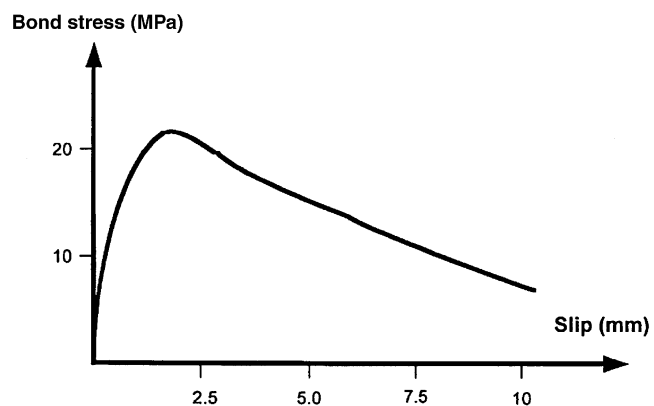


Fig. 1.8—Bond stress-slip curve for bar loaded monotonically and failing by pullout (Eligehausen, Popov, and Bertero 1983). (Note: 1 MPa = 145 psi; 1 mm = 0.0394 in.)

(Maeda, Otani, and Aoyama 1991; Sakurada, Morohashi, and Tanaka 1993; Aziznamini, Chisala, and Ghosh 1995).

Pullout failure (Fig. 1.2(d)) occurs primarily in cases of high confinement and low bonded lengths. In most structural applications, however, splitting failure (Fig. 1.2(b) and (c)) is more common (Clark 1950; Menzel 1952; Chinn, Ferguson, and Thompson 1955; Ferguson and Thompson 1962; Losberg and Olsson 1979; Soretz and Holzenbein 1979; Johnston and Zia 1982; Treece and Jirsa 1989; Choi et al. 1991). For this reason, data used for design are normally limited to members with a minimum development (or splice) length to bar diameter ratio of 15 or 16 and specified maximum values of confinement provided by the concrete and the transverse reinforcement (the latter will be discussed at greater length in Chapter 3). When extended to higher values of confinement, design expressions based on splitting give unrealistically high bond strengths (ACI 318; Orangun, Jirsa, and Breen 1977; Darwin et al. 1996a).

The role played by splitting cracks in bond failure emphasizes the importance of the tensile properties of concrete in controlling bond strength. As will be discussed in Section 2.6, the tensile properties involve more than strength, and include fracture energy; that is, the capacity of the concrete to dissipate energy as a crack opens.

1.4—Notation

A_b	= area of bar being developed or spliced
	= area of largest bar being developed or spliced (CEB-FIP 1990)
A_{tr}	= area of each stirrup or tie crossing the potential plane of splitting adjacent to the reinforcement being developed, spliced, or anchored
b_w	= width of the web of a beam
c	= spacing or cover dimension
	= $c_{\min} + d_b/2$
c_b	= bottom concrete cover for reinforcing bar being developed or spliced
$c_{\max}$	= maximum (c_b , c_s)
c_{med}	= median (c_{so} , c_b , $c_{si} + d_b/2$) [that is, middle value], (Esfahani and Rangan 1998a,b)
$c_{\min}$	= minimum cover used in expressions for the bond strength of bars not confined by transverse reinforcement
	= minimum (c_{so} , c_b , $c_{si} + d_b/2$) (Esfahani and Rangan 1998a,b)
$c_{\min}$	= smaller of minimum concrete cover or 1/2 of the clear spacing between bars
	= minimum (c_b , c_s)
c_s	= minimum [c_{so} , $c_{si} + 0.25$ in. (6.35 mm)]
c_{si}	= 1/2 of the bar clear spacing
c_{so}	= side concrete cover for reinforcing bar
C_R	= $44 + 330(R_r - 0.10)$ (ACI 408.3)
d_b	= diameter of bar
E_c	= modulus of elasticity of concrete
f_c	= stress in concrete
f'_c	= concrete compressive strength based on 6 x 12 in. (150 x 300 mm) cylinders
	= specified compressive strength of concrete

f_{ct}	= average splitting tensile strength of concrete, based on split cylinder test
f_s	= stress in reinforcing bar
f_y	= yield strength of steel being developed or spliced
f_{yt}	= yield strength of transverse reinforcement
G_f	= fracture energy
h_r	= average height of deformations on reinforcing bar
jd_i	= internal moment arm at section i
K	= constant used in CEB-FIP design expression for development length
K_1	= constant used to calculate T_s
K_{tr}	= transverse reinforcement index
	= $\frac{A_{tr}f_{yt}}{1500sn}, \left(\frac{A_{tr}f_{yt}}{10.34sn}\right)^* \text{ (ACI 318)}$
	= $35.3 t_r t_d A_{tr} / sn \text{ (ACI T2-98)}$
	= $C_R(0.72d_b + 0.28) \frac{A_{tr}}{sn}, \left(C_R(0.0283d_b + 0.28) \frac{A_{tr}}{sn}\right)^* \text{ (ACI 408.3)}$
	= $(0.52 t_r t_d A_{tr} / sn) f'_c{}^{1/2}, [(6.26 t_d A_{tr} / sn) f'_c{}^{1/2}]^* \text{ (see Section 4.3)}$
l_d	= development or splice length
$l_{d,\min}$	= minimum development length
$l_{s,\min}$	= minimum splice length
M	= constant used in expressions for the bond strength of bars not confined by transverse reinforcement
	= $\cosh(0.00092 l_d \sqrt{r f'_c / d_b}), [\cosh(0.0022 l_d \sqrt{r f'_c / d_b})]^* \text{ (Esfahani and Rangan 1998a,b)}$
M	= ratio of the average yield strength to the design yield strength of the developed bar (CEB-FIP)
M_i	= internal moment at section i
n	= number of bars being developed or spliced
N	= the number of transverse stirrups, or ties, within the development or splice length
p	= nominal perimeter of bar
p	= power of f'_c
r	= constant used in expressions for the bond strength of bars not confined by transverse reinforcement; a function of R_r
	= 3 for conventional reinforcement ($R_r \approx 0.07$) (Esfahani and Rangan 1998a,b)
R_r	= relative rib area of the reinforcement
s	= spacing of transverse reinforcement
s_r	= average spacing of deformations on reinforcing bar
t_d	= term representing the effect of bar size on T_s
	= $0.72d_b + 0.28, (0.028d_b + 0.28)^* \text{ (Darwin et al. 1996a,b)}$
	= $0.78d_b + 0.22, (0.03d_b + 0.22)^* \text{ (Zuo and Darwin 1998, 2000) (see Section 4.3)}$
t_r	= term representing the effect of relative rib area on T_s
	= $9.6R_r + 0.28 \text{ (Darwin et al. 1996a,b; Zuo and Darwin 1998, 2000) (see Section 4.3)}$
T_b	= total bond force of a developed or spliced bar
	= $T_c + T_s$

*SI expressions.

T_c	= concrete contribution to total bond force, the bond force that would be developed without transverse reinforcement
T_i	= tensile force at section i
T_s	= steel contribution to total bond force, the additional bond strength provided by the transverse steel
u	= bond stress
u_b	= bond strength of a bar confined by transverse reinforcement
	= $u_c + u_s$
u_c	= average bond strength at failure of a bar not confined by transverse reinforcement
u_s	= bond strength of a bar attributed to the confinement provided by the transverse reinforcement
U	= bond force per unit length
V	= shear on a section
α	= rib face angle
a	= reinforcement location factor
a_b	= factor used to increase the development length of a bar for lap splices (CEB-FIP 1990)
	= coating factor
b	= angle between transverse rib and longitudinal axis of the bar
b	= reliability index
Δf_s	= change in steel stress over length Δl
ΔT	= change in the bar force as a result of an externally applied load
e_c	= strain in concrete in uniaxial compression
e_o	= concrete strain at maximum concrete stress under uniaxial compression
g	= reinforcement size factor
l	= lightweight aggregate concrete factor
f	= capacity-reduction factor
f_b	= capacity-reduction factor for bond
f_d	= effective f -factor for bond
	= f_b/f_{tension}
f_{tension}	= capacity-reduction factor for section in tension
ΣA_r	= total area of ribs around bar perimeter measured on the longitudinal section of each rib using the trapezoidal method to approximate the area under the curve
ΣA_{tr}	= area of transverse reinforcement along l_d
Σgaps	= sum of the gaps between ends of transverse deformations on reinforcing bar
Σo	= perimeters of the bars anchored at a section
w	= $0.1 \frac{c_{\text{max}}}{c_{\text{min}}} + 0.9 \leq 1.25$ (ACI 408.3) (see Section 4.3)

CHAPTER 2—FACTORS AFFECTING BOND

Many factors affect the bond between reinforcing bars and concrete. The major factors are discussed in this chapter. Background research, bond behavior, and relationships between bond strength and geometric and material properties are presented under three major subject headings: structural characteristics, bar properties, and concrete properties. The structural characteristics addressed include concrete cover and bar spacing, the bonded length of the bar, the degree of transverse reinforcement, the bar casting position, and the

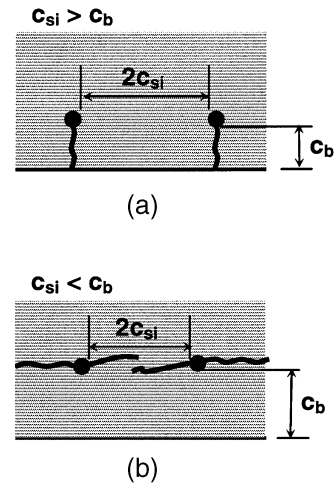


Fig. 2.1—Bond cracks: (a) $c_{si} > c_b$; and (b) $c_{si} < c_b$.

use of noncontact lap splices. The bar properties covered include bar size and geometry, steel stress and yield strength, and bar surface condition, while the concrete properties include compressive strength, tensile strength and fracture energy, aggregate type and quantity, concrete slump and workability, and the effects of admixtures, fiber reinforcement, and degree of consolidation.

2.1—Structural characteristics

2.1.1 Concrete cover and bar spacing—Bond force-slip curves become steeper and bond strength increases as cover and bar spacing increase. The mode of failure also depends on cover and bar spacing (Untrauer 1965; Tepfers 1973; Orangun, Jirsa, and Breen 1977; Eligehausen 1979; Darwin et al. 1996a). For large cover and bar spacing, it is possible to obtain a pullout failure, such as shown in Fig. 1.2(d). For smaller cover and bar spacing, a splitting tensile failure occurs (Fig. 1.2(c)), resulting in lower bond strength. The latter failure mode is the type expected to govern for most structural members. Splitting failures can occur between the bars, between the bars and the free surface, or both. Pullout-like failures can occur with some splitting if the member has significant transverse reinforcement to confine the anchored steel.

With bond failures involving splitting of the concrete for bars that are not confined by transverse reinforcement, the peak load is governed by the tensile response of the concrete, which depends on both tensile capacity and energy dissipation capacity, normally described as fracture energy G_f . As described in Section 2.3.1, concrete exhibiting higher fracture energies provide improved bond capacities, even if the concrete has similar tensile strengths.

When splitting failures occur, the nature of the splitting failure depends, in general, on whether the concrete cover c_b is smaller than either the concrete side cover c_{so} or 1/2 of the bar clear spacing c_{si} . (In this case, the symbol for bottom cover c_b is used, but the discussion applies equally to bottom, top, and side cover.) When c_b is smaller than c_{so} and c_{si} , the splitting crack occurs through the cover to the free surface (Fig. 2.1(a)). When c_{so} or c_{si} is smaller than c_b , the splitting crack forms through the side cover or between the reinforcing bars (Fig. 2.1(b)), respectively.

In ACI 318, the effective value of c_{si} for development length calculations is equal to the actual value of c_{si} . In the Canadian requirements for reinforced concrete design (CSA Standard A23.3-94), however, a greater value $[2/3 \text{ of the center-to-center spacing of the bars being developed minus } 1/2 \text{ of the bar diameter} = (2/3)(2c_{si} + d_b) - (1/2)d_b = (4/3)c_{si} + (1/6)d_b]$ is used. In an analysis of bars not confined by transverse reinforcement, Zuo and Darwin (1998) found that the best match with tests is obtained: 1) using $1.6c_{si}$ as the effective value of c_{si} when using a multiple of c_{si} ; and 2) using $c_{si} + 0.25 \text{ in. } (c_{si} + 6.4 \text{ mm})$ as the effective value of c_{si} when a constant value is added to c_{si} . Of the two procedures, $1.6c_{si}$ provides the better predictions. The fact that the effective value of c_{si} is greater than the actual value is most likely “due to the longer effective crack lengths that occur when concrete splits between bars” (Darwin et al. 1996a) (Fig. 2.1(b)), which make the bars behave as if they have an increased separation.

Although $1.6c_{si}$ gives the best match for development and splice strengths for bars not confined by transverse reinforcement, the value tends to overestimate the effective crack length between bars confined by transverse reinforcement (Zuo and Darwin 1998). In the latter case, $c_{si} + 0.25 \text{ in. } (c_{si} + 6.4 \text{ mm})$, provides a better match with tests. This observation suggests that there is a small but significant difference in the effect of cracks between bars on bond strength in the presence of confining reinforcement. In the presence of transverse reinforcement, the effective crack length between bars is still greater than the clear distance between the bars, but not as great as for similar members without confining transverse reinforcement.

Orangun, Jirsa, and Breen (1975) and Darwin et al. (1992, 1996a) observed that, although the minimum value of c_b , c_{so} , and c_{si} has the principal effect on bond strength, the relative value of c_{so} or c_{si} to c_b is also important. For bars not confined by transverse reinforcement, Darwin et al. (1996a) found that, compared to cases in which the minimum value of c_{so} or c_{si} equals c_b , the bond strength of bars for which the minimum value c_{so} or c_{si} does not equal c_b increases by the ratio

$$\left(0.1 \frac{c_{\max}}{c_{\min}} + 0.9\right) \leq 1.25 \quad (2-1)$$

where

- $c_{\max}$ = maximum (c_b , c_s);
- $c_{\min}$ = minimum (c_b , c_s);
- c_b = bottom cover;
- c_s = minimum [c_{so} , $c_{si} + 0.25 \text{ in. } (6.4 \text{ mm})$];
- c_{so} = side cover; and
- c_{si} = 1/2 of the bar clear spacing.

In addition to cover thickness, the nature of the cover plays a role in bond strength. With emphasis on methods of construction for reinforced concrete bridge decks, Donahey and Darwin (1983, 1985) evaluated the bond strength of bars with 3 in. (76 mm) of monolithic cover and bars with laminar cover, consisting of 3/4 in. (19 mm) monolithic cover topped with a 2-1/4 in. (57 mm) high-density concrete overlay. The bars with the 3 in. (76 mm) laminar cover achieved about the

same bond strengths (average = 97%) as achieved by bars with the same thickness of monolithic cover, even though greater bond strengths would have been expected based on the compressive strength of the overlay concrete, which ranged between 110 and 155% of the compressive strength of the base concrete.

2.1.2 Development and splice length—Increasing the development or splice length of a reinforcing bar will increase its bond capacity. The nature of bond failure, however, results in an increase in strength that is not proportional to the increase in bonded length. The explanation starts with the observations that bond forces are not uniform (Fig. 1.5) and that bond failures tend to be incremental, starting in the region of the highest bond force per unit length. In the case of anchored bars, longitudinal splitting of the concrete initiates at a free surface or transverse flexural crack where the bar is most highly stressed. For spliced bars, splitting starts at the ends of the splice, moving towards the center. For normal-strength concrete, splitting may also be accompanied by crushing of the concrete in front of the ribs as the bar moves (or slips) with respect to the concrete. For higher-strength concrete and for normal-strength concrete in which the bars are epoxy coated, the degree of crushing in front of the ribs is significantly decreased. For splice specimens studied after failure, it is common to see no crushed concrete at ribs near the tensioned end of a spliced bar, with the crushed concrete located at the end of the bar, indicating that failure occurred by a slow wedging action followed by rapid final slip of the bar at failure. Because of the mode of bond failure, the nonloaded end of a developed or spliced bar is less effective than the loaded end in transferring bond forces, explaining the nonproportional relationship between development or splice length and bond strength.

Although the relationship between the bond force and the bonded length is not proportional, it is nearly linear, as illustrated in Fig. 2.2 for No. 4 to 14 (No. 13 to 43) bars. Figure 2.2 indicates that bars will have measurable bond strengths even at low embedded lengths. This occurs because, in the tests, there is always at least one set of ribs that forces the concrete to split before failure. When failure occurs, a significant crack area is opened in the member due to splitting (Brown, Darwin, and McCabe 1993; Darwin et al. 1994; Tholen and Darwin 1996). As the bonded length of the bar increases, the crack surface at failure also increases in a linear but not proportional manner with respect to the bonded length. Thus, the total energy needed to form the crack and, in turn, the total bond force required to fail the member, increase at a rate that is less than the increase in bonded length. Therefore, the common design practice (ACI 318) of establishing a proportional relationship between bond force and development or splice length is conservative for short bonded lengths, but becomes progressively less conservative, and eventually unconservative, as the bonded length and stress in the developed or spliced bar increase.

2.1.3 Transverse reinforcement—Transverse reinforcement confines developed and spliced bars by limiting the progression of splitting cracks and, thus, increasing the bond force required to cause failure (Topic 1-73; Orangun, Jirsa,

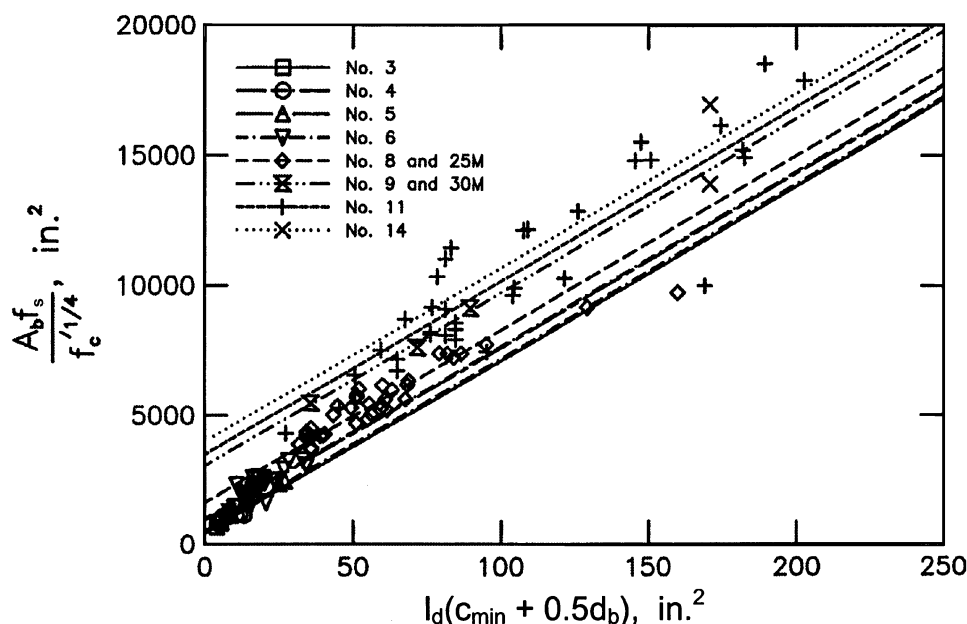


Fig. 2.2—Bond strength $A_b f_s$ normalized with respect to $f_c^{1/4}$ versus the product of the development or splice length l_d and the smaller of the minimum concrete cover to the center of the bar or $1/2$ of the center-to-center bar spacing ($c_{min} + 0.5d_b$) (Darwin et al. 1996b). (Note: $1 \text{ in.}^2 = 645 \text{ mm}^2$)

and Breen 1977; Darwin and Graham 1993a,b). An increase in transverse reinforcement results in an increase in bond force, eventually converting a splitting failure to a pullout failure. Additional transverse reinforcement, above that needed to cause the transition from a splitting to a pullout failure, becomes progressively less effective, eventually providing no increase in bond strength (Orangun, Jirsa, and Breen 1977).

The total bond force of a developed or spliced bar T_b can be represented as the sum of a concrete contribution T_c , representing the bond force that would be developed without the transverse reinforcement, plus a steel contribution T_s , representing the additional bond strength provided by the transverse steel

$$T_b = T_c + T_s \quad (2-2)$$

The value of the concrete contribution is affected somewhat by the presence of the transverse steel, as discussed in Section 2.1.1, because the effective crack length between bars is reduced as bar slip continues in the process of mobilizing the additional bond strength provided by the transverse reinforcement (Zuo and Darwin 1998, 2000). The effect of transverse reinforcement on T_c is small but measurable.

The value of the steel contribution T_s is a function of the area of reinforcing steel that crosses potential crack planes, the strength of the concrete, and both the size and deformation properties of the developed or spliced reinforcement. It can be represented in the form (Zuo and Darwin 1998, 2000)

$$T_s = K_1 t_r t_d \frac{N A_{tr}}{n} f_c'^p \quad (2-3)$$

where

- K_1 = a constant;
- t_r = a factor that depends on the relative rib area R_r of the reinforcement;
- t_d = a factor that depends on the diameter d_b of the developed or spliced bar;
- N = the number of transverse stirrups, or ties, within the development or splice length;
- A_{tr} = area of each stirrup or tie crossing the potential plane of splitting adjacent to the reinforcement being developed or spliced;
- n = number of bars being developed or spliced along the plane of splitting;
- f_c' = concrete compressive strength based on 6 x 12 in. (150 x 300 mm) cylinders; and
- p = power of f_c' between 0.75 and 1.00 (see Section 3.3 for values of K_1 and p).

In the case shown in Fig. 2.3, illustrating a two-leg stirrup confining three spliced reinforcing bars in the same plane, A_{tr} = two times the area of the stirrup A_t and $n = 3$ if c_{so} or the effective value of c_{si} is less than c_b . $A_{tr} = A_t$ and $n = 1$ if c_{so} and the effective value of c_{si} are greater than c_b .

The values of t_r and t_d can be represented as linear functions of R_r and d_b , respectively (Zuo and Darwin 1998, 2000).

$$t_r = 9.6R_r + 0.28 \quad (2-4)$$

$$t_d = 0.78d_b + 0.22 \quad (2-5)$$

with d_b in inches and $R_r \leq 0.14$.

The relationships given in Eq. (2-4) and (2-5) suggest that an increase in the wedging action of the bars, resulting from

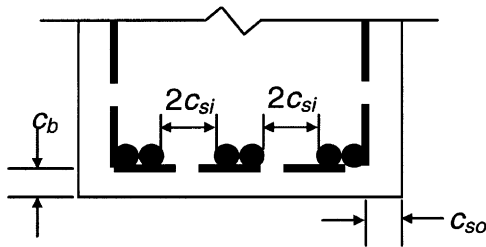


Fig. 2.3—Beam cross sections showing definition of c_b , c_{si} , and c_{so} .

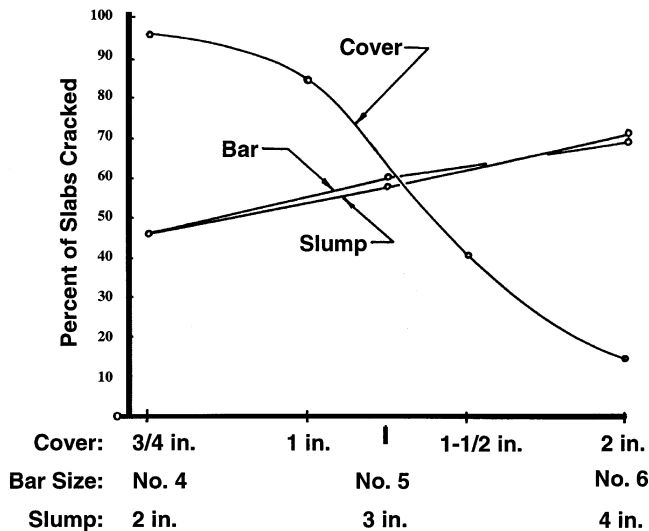


Fig. 2.4—Settlement cracking as a function of bar size, slump, and cover (Dakhil et al. 1975). (Note: 1 in. = 25.4 mm)

both an increase in R_r (a relative measure of rib size and spacing) and an increase in bar size d_b (an absolute measure of rib size), will increase the stress in the stirrups, resulting in an increase in confining force. The relationship between confinement and the degree of wedging action is in concert with the observation that stirrups rarely yield (Maeda, Otani, and Aoyama 1991; Sakurada, Morohashi, and Tanaka 1993; Azizinamini, Chisala, and Ghosh 1995), allowing an increase in lateral displacement caused by wedging to be translated into an increase in confining force. As a result, the yield strength of the transverse reinforcement f_{yt} does not play a role in the steel contribution to bond force, T_s . The effect of concrete strength on T_s is discussed further in Section 2.3.1.

2.1.4 Bar casting position—As early as 1913, Abrams observed that bar position during concrete placement plays an important role in the bond strength between concrete and reinforcing steel. Top-cast bars have lower bond strengths than bars cast lower in a member (Clark 1946, 1949; Collier 1947; Larnach 1952; Menzel 1952; Menzel and Woods 1952; Ferguson and Thompson 1962, 1965; CUR 1963; Untrauer 1965; Welch and Patten 1965; Untrauer and Warren 1977; Thompson et al. 1975; Jirsa and Breen 1981; Luke et al. 1981; Zekany et al. 1981; Donahey and Darwin 1983, 1985; Altowaiji, Darwin, and Donahey 1984, 1986; Brettmann, Darwin, and Donahey 1984, 1986; Jeanty,

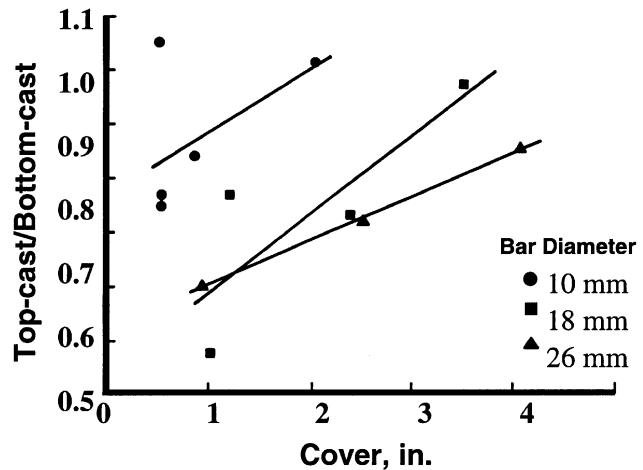


Fig. 2.5—Ratio of top-cast to bottom-cast bar bond strength versus cover (CUR 1963). (Note: 1 in. = 25.4 mm)

Mitchell, and Mirza 1988). This behavior is recognized in ACI 318 and the AASHTO Bridge Specifications (1996). Top-reinforcement, horizontal reinforcement with more than 12 in. (300 mm) of fresh concrete cast in the member below the development length or splice, requires a 30% increase in development length (ACI 318). Most research, however, indicates that while an increased depth of concrete below a bar reduces bond strength, the effect of shallow top cover is of greater significance. The impact of shallow top cover on the top-cast bar effect is emphasized by the fact that the strength reduction becomes progressively greater as cover is decreased.

The lower bond strength of top-cast bars may be explained as follows: the greater the depth of concrete below a bar, the greater will be the settlement and accumulation of bleed water at the bar, because there is more concrete beneath the bar to settle and bleed. The effects of settlement and bleeding on the concrete around a bar are aggravated by increased concrete slump and decreased cover above the bar. Dakhil, Cady, and Carrier (1975) found that longitudinal settlement cracking increased above top-cast bars with increased slump and bar size and especially with decreased top cover (Fig. 2.4).

Menzel (1952) observed settlement cracks over 1 in. (25 mm) diameter top-cast bars with a 2 in. (50 mm) cover for specimens placed with 6 in. (150 mm) slump concrete that was consolidated by hand. Cracks were not observed in lower slump concrete that was vibrated internally. Menzel (1952) also observed that the bond strength of top-cast reinforcement decreases as specimen depth increases. He observed the greatest reduction for high-slump concrete consolidated by hand rodding and the least for low-slump concrete consolidated with vibration.

The importance of cover on the reduction in bond strength for top-cast bars is demonstrated by tests in the Netherlands (CUR 1963). As shown in Fig. 2.5, the ratio of top-cast bar to bottom-cast bar bond strength decreased significantly as cover decreased.

Ferguson and Thompson (1965) conducted tests to compare the ratio of top-cast to bottom-cast bar bond

strength. The bond strength of top-cast bars decreased with increasing slump and decreasing top cover.

Zekany et al. (1981) conducted splice tests in 16 in. (406 mm) deep beams with both top- and bottom-cast bars. Splice strength decreased for both top- and bottom-cast bars with increasing slump, but the decrease was consistently greater for the top-cast bars.

In a study of casting position, Luke et al. (1981) clearly demonstrated that as the depth of concrete below a bar increases, the bond strength decreases (Fig. 2.6). They observed the greatest incremental decrease in strength for top-cast bars. As will be discussed in more detail in Section 2.3.5, they also observed that bond strength decreased with increasing slump, but that the decrease was most pronounced for top-cast bars. For bars cast below the specimen mid-depth, slump appeared to have little effect.

Brettmann, Darwin, and Donahey (1984, 1986) also found that as the amount of concrete below a bar increases, bond strength decreases. The measured decrease was least for low-slump concrete and greatest for high-slump concrete obtained without the use of a high-range, water-reducing admixture. Bond strength decreased for bars with as little as 8 in. (200 mm) of concrete below the bar, bars that would not be defined as top reinforcement under the provisions of ACI 318. In similar tests run by Zilveti et al. (1985), a "top-bar effect" was obtained for top-cast bars with as little 6 in. (150 mm) of concrete below the bars. Brettmann, Darwin, and Donahey (1984, 1986) found that bond strength was similar for bars placed 8 and 15 in. (200 and 380 mm) above the bottom of the forms. Bond strength was lowest for bars placed 36 in. (915 mm) above the bottom of the forms, with the largest reductions obtained for high slump, nonvibrated specimens, as shown in Fig. 2.7. In all cases (even for bars with only 8 in. [200 mm] of concrete below the bars), the decrease in bond strength between bottom-cast and top-cast bars was greater than between top-cast bars with 8 and 36 in. (203 and 914 mm) of concrete below the bars. These results indicate that the choice of 12 in. (300 mm) of concrete below a bar for the 30% increase in development length for top reinforcement is arbitrary. Based on concrete depth alone, there seems to be a gradual decrease in bond strength with no sharp drop-off.

2.1.5 Noncontact lap splices—A noncontact lap splice (also called a spaced splice) provides continuity of reinforcement by overlapping the ends of the steel bars without the bars touching each other. According to ACI 318, "bars spliced by noncontact lap splices in flexural members shall not be spaced transversely farther apart than one-fifth the required lap splice length, nor 6 in. (150 mm)." This provision was first incorporated in ACI 318-71. The commentary to ACI 318 argues that if individual bars in noncontact lap splices are too widely spaced, an unreinforced section is created, and that forcing a potential crack to follow a zigzag line (5 to 1 slope) is considered to be a minimum precaution. The commentary points out that the 6 in. (150 mm) maximum spacing is added because most research available on lap splices of deformed bars was conducted with reinforcement within this spacing.

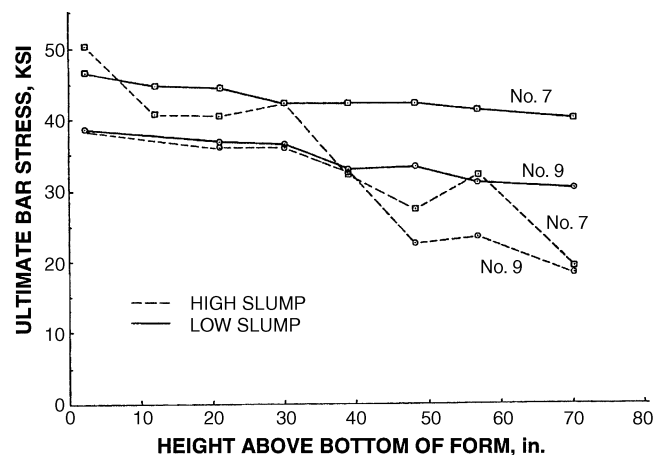


Fig. 2.6—Bond strength as a function of bar location within a wall specimen. High slump = 8-1/2 in. (215 mm). Low slump = 3 in. (75 mm) (Luke et al. 1981). (Note: 1 ksi = 6.89 MPa; 1 in. = 25.4 mm)

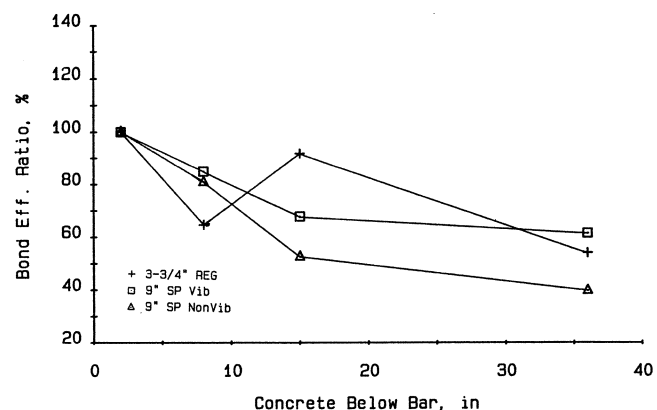


Fig. 2.7—Bond efficiency ratio versus concrete depth below the bar for lower temperature concrete (53 °F [12 °C]). REG = concrete without a superplasticizer; and SP = superplasticized concrete (Brettmann, Darwin, and Donahey 1986) (Note: 1 in. = 25.4 mm)

In earlier codes, a minimum clear spacing was actually required. ACI 318-47 specified that the minimum clear spacing between spliced bars must not be less than $1.5d_b$ for round bars or $1\frac{1}{3}$ times the maximum size of aggregate, and at least 1 in. (25 mm) in any case. ACI 318-51 retained these requirements, except that the $1.5d_b$ requirement was changed to $1.0d_b$. Engineering practice before 1950 usually required that an allowance be made for a reduction in bonded area for tied lap splices to account for the fact that concrete does not completely surround spliced bars that are in contact. This was provided by lengthening the splice. It was only in 1963 that the ACI Building Code (ACI 318-63) allowed both spaced and contact lap splices.

In a lap splice, the force in one bar is transferred to the concrete which, in turn, transfers it to the adjacent bar. This transfer of forces from one bar to another in a noncontact splice can be seen from the crack pattern, as shown in Fig. 2.8 (MacGregor 1997).

Walker (1951) reported on a series of pull-out tests to compare the performance of spaced and tied lap

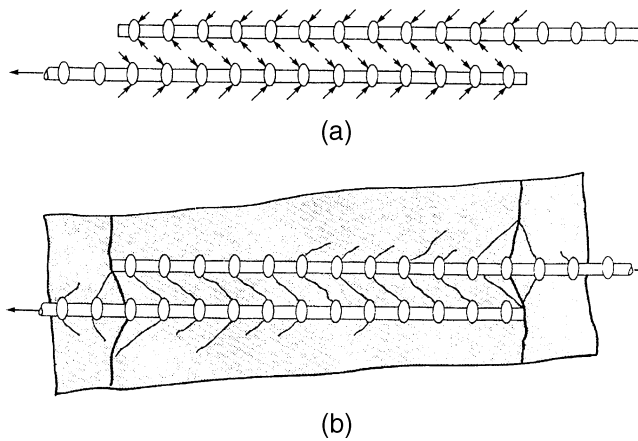


Fig. 2.8—Noncontact tension lapped splices: (a) forces on bars at splice; and (b) internal cracks at splice (MacGregor 1997).

splices. Two levels of concrete strength and three types of deformed reinforcing bars were studied. In all tests where the spliced bars were spaced, the clear spacing was $1.5d_b$. Beam tests showed no significant difference between the two splicing methods (zero spacing and $1.5d_b$ spacing), but at loads close to ultimate, there was some indication that spaced spliced bars might be slightly preferable, showing less center beam deflection and end slip at a given load level. In the pullout tests, there was no weakening of bond at the tied splice. Considering all of the data, however, Walker concluded that within the scope of his study there was no significant loss of bond when deformed bars were tied together at the splice.

Chamberlin (1952) investigated the effect of spacing of spliced bars in tension pull-out specimens. The tests were designed to provide data on the effect of spacing of lapped bars on bond and also on the effect of length of overlap in relation to effectiveness of stress transfer from one bar to another at a splice. Confinement was provided by spiral wire to prevent splitting. Based on test results, Chamberlin concluded deformed bars developed better average bond stress in adjacent tied splices (zero spacing) than in spaced splices, but that the differences in bond strength for clear spacings of one-bar diameter and three-bar diameters were not significant.

Chinn, Ferguson, and Thompson (1955) reported on research investigating the effects of many variables on the bond capacity of spliced reinforcement including the clear spacing between spliced bars, with values of 0 (contact splice), 0.75, 1.0, 1.25, and 1.88 in. (20, 25, 32, and 48 mm). Forty beam specimens were tested. Each beam contained either one or two splices, placed in a constant moment region at the center of the beam. They concluded that their tests confirmed the earlier tests by Walker (1951) and Chamberlin (1952), showing little difference in strength between contact and spaced lap splices.

Chamberlin (1958) studied the effect of spacing of lapped bars on bond strength and the effect of length of lap on the load-carrying capacity of small beams. Twenty-one beams were tested with no restraint against splitting. All beams were 6 x 6 in. (150 x 150 mm) in cross section and 36 in.

(915 mm) in length, simply supported under symmetrical two-point loading. The clear spacing between lapped bars was either 1/2 or 1 in. (12 or 25 mm). Chamberlin concluded that there was little difference in strength between adjacent and spaced splices.

Sagan, Gergely, and White (1991) studied the behavior of noncontact lap splices subjected to monotonic and repeated inelastic loading. Forty-seven full-scale flat-plate specimens were tested. Each specimen contained two splices. Variables included splice-bar spacing, concrete compressive strength, bar size, the amount and distribution of transverse reinforcement, and the lap length. The specimens were loaded slowly in direct tension. If a specimen survived the first loading to yield, it was unloaded slowly and then reloaded to yield; this process was repeated until failure. Sagan, Gergely, and White observed that:

1. A noncontact lap splice can be modeled as a plane truss, with load transferred between the two spliced bars through compressive struts in the concrete; the tension elements are provided by the transverse reinforcement and surrounding concrete;

2. An in-plane splitting crack forms between the bars of spaced bar splices. The crack results from bond-induced bursting stresses and Poisson strains generated by the compression stress field. Diagonal surface cracking of the concrete between the spliced bars becomes more prominent as the bar spacing increases. Even the large cracks that formed at the ends of the splice were diagonal;

3. The splice strength of the monotonically loaded specimens increased when transverse reinforcement was provided. Also, the number of inelastic load cycles sustained by a tension splice depends on the amount of confinement provided by transverse reinforcement; and

4. The strength of a splice is independent of the splice-bar spacing up to at least six times the bar diameter for monotonic loading. Under repeated loading up to the yield strength of the splice bars, the maximum load (equal to the yield load) is also independent of bar spacing, up to eight times the bar diameter for both No. 6 and No. 8 (No. 19 and No. 25 mm) bars.

To check the validity of the ACI 318 provisions for noncontact splices, Hamad and Mansour (1996) tested 17 slabs in positive bending. Reinforcement on the tension side consisted of three reinforcing bars spliced at the center of the span. No transverse reinforcement was provided in the splice region. The clear spacing between lapped bars varied between 0 and 50% of the splice length for slabs reinforced with 0.55 and 0.63 in. (14 and 16 mm^{*}) bars, and between 0 and 40% of the splice length for 0.80 in. (20 mm) bars. For eight specimens, the clear transverse spacing between the spliced bars was at least 20% of the splice length specified by ACI 318. The splice lengths were limited to 11.8 in. (300 mm) for slabs reinforced with 0.55 and 0.63 in. (14 and 16 mm) bars, and 13.8 in. (350 mm) for 0.8 in. (20 mm) bars. They observed that the noncontact splices developed greater

*The bars used in these tests were sized in millimeters.



bond strength than the contact splices, up to an optimum clear spacing of about $5d_b$. They concluded that the ACI limit on the transverse spacing of noncontact tensile lap splices to 20% of the lap length is conservative and that a limit of 30% could be used.

2.2—Bar properties

2.2.1 Bar size—The relationship between bar size and bond strength is not always appreciated. The reason is that, while (1) a longer development or splice length is required as bar size increases, and (2) for a given development or splice length, larger bars achieve higher total bond forces than smaller bars for the same degree of confinement.

Addressing the second point first, for a given bonded length, larger bars require larger forces to cause either a splitting or pullout failure, as illustrated in Fig. 2.2, for bars not confined by transverse reinforcement. The result is that the total force developed at bond failure is not only an increasing function of concrete cover, bar spacing, and bonded length, but also of bar area (Orangun, Jirsa, and Breen 1977; Darwin et al. 1992, 1996a). The bond force at failure, however, increases more slowly than the bar area, which means that a longer embedment length is needed for a larger bar to fully develop a given bar stress (the first point). When evaluated in terms of bond stress (Eq. (1-4) and (1-5)), smaller bars appear to have even a greater advantage; thus, conventional wisdom suggests that it is desirable to use a larger number of small bars rather than a smaller number of large bars; this is true until bar spacings are reduced to the point that bond strength is decreased (Ferguson 1977; Rehm and Eligehausen 1979).

The size of a developed bar also plays an important role in the contribution of confining transverse reinforcement to bond strength. As larger bars slip, higher strains and, thus, higher stresses, are mobilized in the transverse reinforcement, providing better confinement. As a result, the added bond strength provided by transverse reinforcement increases as the size of the developed or spliced bars increases, as shown in Fig. 2.9, which compares the relative effect of transverse reinforcement on bond strength M , normalized to the effect of the relative rib area t_r , versus bar diameter, for No. 5, No. 8, and No. 11 (No. 16, No. 25, and No. 36) bars with a wide range of relative rib area R_r . The term M is the ratio of the additional bond force provided by the transverse steel T_s , normalized with respect to $f_c'^{3/4}$ (see Section 2.3.2), to the area of transverse steel confining the bar.

2.2.2 Bar geometry—Historically, there have been widely conflicting views of the effect of bar geometry on bond strength. Some studies indicate that deformation patterns have a strong influence on bond strength. Other studies show that deformation pattern has little influence, and it is not uncommon for bars with different patterns to produce nearly identical development and splice strengths. Over time, however, the effects of bar geometry on bond behavior have become increasingly clear, as will be described in this section.

The earliest study on bond resistance of plain and deformed reinforcing bars was done by Abrams (1913) using pullout and beam specimens. The test results showed that

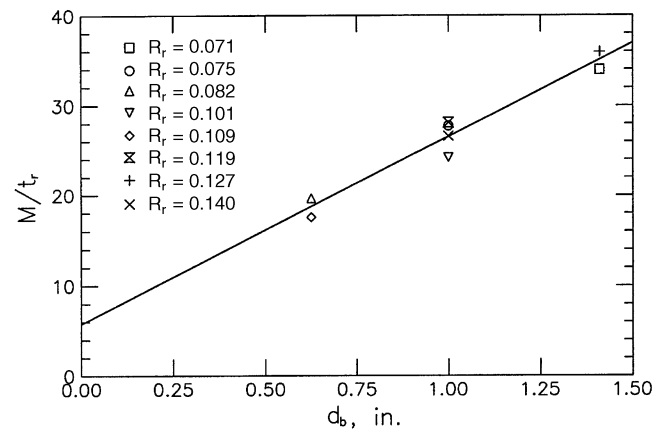


Fig. 2.9—Relative contribution of transverse reinforcement M , normalized with respect to the relative rib area ($t_r = 9.6R_r + 0.28$) versus bar diameter (Zuo and Darwin 1998). (Note: 1 in. = 25.4 mm)

deformed bars produced higher bond resistance than plain (smooth) bars. Abrams found that in pullout tests of plain bars, bond resistance reached its maximum value at a loaded end slip of about 0.01 in. (0.25 mm). For deformed bars, the load-slip performance was the same as for plain bars up to the slip corresponding to the maximum bond resistance of the plain bars. As slip continued, the ribs on deformed bars increased the bond resistance by bearing directly on the adjacent concrete. Abrams observed that the ratio of the bearing area of the projections (projected area measured perpendicular to the bar axis) to the entire surface area of the bar in the same length could be used as a criterion for evaluating the bond resistance of deformed bars. To improve bond resistance, he recommended that this ratio not be less than 0.2, resulting in closer spacings of the projections than were used in commercial deformed bars at the time.

Over 30 years later, Clark (1946, 1949) investigated 17 commercial deformation patterns using pullout and beam tests. The bond performance for each pattern was evaluated based on the bond stress developed at predetermined values of slip. Based on Clark's investigations, standard deformation requirements were introduced for the first time in the Tentative Specification ASTM A 305-47T, later appearing as ASTM A 305-49. The requirements included a maximum average spacing of deformations equal to 70% of the nominal diameter of the bar and a minimum height of deformations equal to 4% of the nominal diameter for bars with a nominal diameter of 1/2 in. (13 mm) or smaller, 4.5% of the nominal diameter for bars with a nominal diameter of 5/8 in. (16 mm), and 5% for larger bars. These requirements remain unchanged in the current ASTM specifications for reinforcing bars (ASTM A 615; ASTM A 706; ASTM A 767; ASTM A 955; ASTM A 996).

In his study, Clark (1946, 1949) found that bond performance improved for bars with lower ratios of shearing area (bar perimeter times center-to-center distance between ribs) to bearing area (projected rib area normal to the bar axis) and recommended that the ratio of shearing area to bearing area be limited to a maximum of 1.0 and, if possible, 5 or 6. In