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Report on Bond of Steel Reinforcing Bars Under Cyclic Loads

Reported by Joint ACI-ASCE Committee 408



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Report on Bond of Steel Reinforcing Bars Under Cyclic Loads

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Report on Bond of Steel Reinforcing Bars Under Cyclic Loads

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Note: The contributions of Brett Baker are gratefully acknowledged.

This report summarizes research on bond strength and behavior of steel reinforcing bars under cyclic loads. The report provides a background to bond problems, discusses the main variables affecting bond performance, and describes bond behavior under cyclic loads. Two general types of cyclic loads are addressed: high-cycle (fatigue) and low-cycle (earthquake and similar). The anchorage behaviors of straight bars, hooked bars, and lap splices are included. Analytical bond models are described, design recommendations are provided for both high- and low-cycle fatigue, and suggestions for further research are given.

Keywords: anchorage; bar slip; bond; bond models; cyclic loads; design recommendations; development length; fatigue; hooks; earthquake loads; splices.

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The transfer of forces across the interface between concrete and steel by bond stresses is of fundamental importance to many aspects of reinforced concrete behavior. Satisfactory bond performance is an essential goal in detailing reinforcement in structural components. Many detailing provisions in ACI 318 are aimed at preventing bond failures.

Bond stresses in reinforced concrete members arise from two distinct situations. The first is anchorage or development where bars are terminated. The second is flexural bond or the change of force along a bar due to a change in bending moment along the member. Bond performance under static monotonically increasing deformations—referred to as monotonic loading—has been summarized in ACI 352R, ACI 408R, and ACI Committee 408 (1966, 1970, 1979). Bond behavior under cyclic loads received little attention until design for earthquake and wave loads became important design topics (ACI Committee 408 1979). Investigations over the past 40 years have clarified some of the important parameters influencing bond behavior under cyclic loads. However, the influence of many of these parameters is understood only qualitatively.

In this report, “bar” means “reinforcing bar” and “ribs” refer to the deformations on deformed reinforcing bars. Longitudinal deformations on reinforcing bars are not classified as ribs. “Bond stress” refers to the stresses along the bar-concrete interface. The steel stresses along the length of the reinforcing bar are modified by transfer of force between the bar and the surrounding concrete along the interface (refer to Fig. 1.1).

The change in bar tensile force ΔF between two cracked sections along a flexural member ΔF is given by

$$\Delta F = T_1 - T_2 = \left(\frac{M_1}{jd_1} \right) - \left(\frac{M_2}{jd_2} \right) \quad (1.1a)$$

The average bond stress u_b is usually expressed as

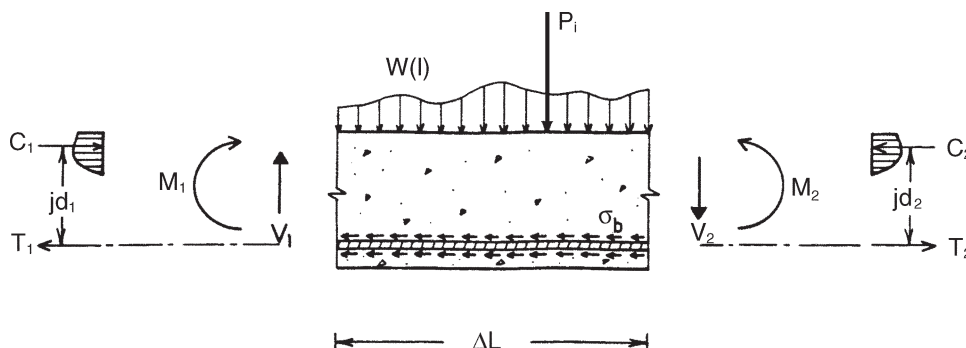


Fig. 1.1—Definition of bond stress.

$$u_b = \frac{q}{\Sigma_o} = \frac{\Delta F}{\Delta L \Sigma_o} = \frac{\Delta f_s (\pi d_b^2 / 4)}{\Delta L (\pi d_b)} = \frac{d_b \Delta f_s}{4 \Delta L} \quad (1.1b)$$

Cyclic loadings are divided into two general categories. The first is designated low-cycle or low-cycle, high-stress loading, or a load history containing less than 100 cycles and having bond stress ranges (u_r) greater than 600 psi (4 MPa). Low-cycle loadings commonly result from earthquake and high-wind loadings. The second category is designated high-cycle or fatigue loading with a load history containing thousands or millions of cycles, but at a low bond stress range (u_r) typically less than 300 psi (2 MPa). Bridge members, offshore structures, and members supporting vibrating machinery are often subjected to high-cycle or fatigue loading. High-cycle loadings can be a problem at service load levels whereas low-cycle loadings can produce problems at the ultimate limit state.

Bond behavior under cyclic loading can further be subdivided according to the type of stress applied. The first is repeated or unidirectional loading, which implies that the bar stress does not reverse from tension to compression during a load cycle, the usual situation for fatigue loading. The second is stress reversal, where the bar is subjected alternatively to tension and compression. Earthquake loading typically causes stress reversals.

1.2—Scope

This report reviews bond and anchorage of steel reinforcing bars in normal and lightweight concrete, with emphasis on bond under cyclic loading. Although the amount of information about the bond behavior of epoxy-coated reinforcing bars subjected to cyclic loading is limited, available references on this topic were reviewed and are presented in the document. Bond and anchorage of prestressing steel and headed reinforcing bars are not addressed in this report. This report serves both designers and researchers, and is organized accordingly. Chapters 3 and 4 present background information on the issue of bond under cyclic loading. Chapters 5, 6, and 7 deal with results of research and development of analytical bond models. Chapters 8 and 9 review international design guidelines dealing with bond under cyclic loads and should be of interest to designers. This report also introduces designers to the basic mechanisms involved in bond, the variables that affect those mechanisms, and differences in bond behavior under cyclic and noncyclic loads.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A_b	= cross-sectional area of reinforcing bar, in. ² (mm ²)
A_c	= area of concrete cross section, in. ² (mm ²)
A_{tr}	= area of transverse reinforcement, in. ² (mm ²)
BI	= representation of the severity of bond stresses relative to the bond strength
C	= concrete cover, in. (mm)
c_b	= smaller of: (a) the distance from center of a bar or wire to nearest concrete surface; and (b) one-half

	the center-to-center spacing of bars or wires being developed, in. (mm)
D	= development length according to AIJ (1990), in. (mm)
DCH	= ductility class high, based on the maximum behavior factor
DCM	= ductility class medium, based on the maximum behavior factor
d	= depth from extreme compression fiber to centroid of tensile reinforcement, in. (mm)
d_b	= diameter of bar, or diameter of bar being developed, in. (mm)
d_{bL}	= nominal diameter of longitudinal bars, in. (mm)
f_b	= static bond strength as used for offshore structures, psi (MPa)
f_{br}	= bond strength at 2,000,000 cycles as used for offshore structures, psi (MPa)
f'_c	= specified concrete compressive strength, psi (MPa)
f_{cd}	= design concrete compressive strength, psi (MPa)
f_{cr}	= stress range in concrete, psi (MPa)
f_{ctm}	= mean value of the tensile strength of concrete, psi (MPa)
f_{min}	= minimum stress level, ksi (MPa)
f_r	= stress range due to live loads and impact recommended by AASHTO, ksi (MPa)
f_{rup}	= static modulus of rupture, psi (MPa)
f_s	= existing reinforcement strength based on available development length, psi (MPa)
f_{sos}	= yield strength divided by a factor of safety (typically = 1.15), offshore structures, psi (MPa)
f_y	= yield strength of bar being developed, psi (MPa)
f_{yd}	= design value of the yield strength of bars, psi (MPa)
f_{yt}	= yield strength of transverse reinforcement, psi (MPa)
h	= depth of member, in. (mm)
h_c	= width of the column parallel to the bars, in. (mm)
h_j	= joint dimension parallel to the bar, in. (mm)
jd_i	= internal moment arm at section i , in. (mm)
K_{tr}	= transverse reinforcement index
k_D	= factor reflecting the ductility class equal to 1 for DCH and to 2/3 for DCM
ℓ	= total length of the member, in. (mm)
ℓ_b	= provided length of straight development, lap splice, or standard hook, in. (mm)
ℓ_d	= development length, in. (mm)
ℓ_d'	= equivalent straight bar anchorage length corresponding to a hooked bar anchorage, in. (mm)
ℓ_{dh}	= lead embedment length for a hook, in. (mm)
ℓ_e	= length of embedment of reinforcement, in. (mm)
M_i	= moment at section i , lb-in. (N-m)
N	= number of cycles to failure
N_{ed}	= design axial force, lbf (N)
q	= change of bar force per unit length of bar, lb/in. (N/mm)
r/h	= ratio of base radius to height of rolled transverse rib
S	= applied stress range, psi (MPa)
S_{bmax}	= maximum bond strength as used for offshore structures, psi (MPa)
S_{br}	= stress range as used for offshore structures, psi (MPa)

S_{max}	= maximum flexural tensile stress, psi (MPa)
S_{min}	= minimum flexural tensile stress, psi (MPa)
S_r	= stress range in straight deformed bars as used in offshore structures, psi (MPa)
s	= spacing of transverse reinforcement, in. (mm)
s_{max}	= peak slip, in. (mm)
T_i	= tensile force at section i , lb (N)
u	= bond stress range, psi (MPa)
u_b	= shear force per unit area of nominal bar surface between two sections along the bar, lb/in. (N/mm)
u_{bb}	= maximum bond stress of the beam bars over the column width, psi (MPa)
u_r	= bond stress range—the difference between bond stresses at the maximum and minimum load, considering the loading direction, psi (MPa)
α	= stress multiplier for longitudinal bars at the joint or member interface for Type 2 joints
Δf_s	= change of steel stress over length ΔL , psi (MPa)
ΔL	= length of bar over which bond stress is computed, in. (mm)
γ_{Rd}	= model uncertainty factor on the design value of resistances taken as being equal 1.2 or 1.0, respectively, for DCH or DCM (due to overstrength owing to strain-hardening of the longitudinal steel in the beam)
μ	= constant to determine development length, with suggested values ranging from 10 to 12.5
v_d	= normalized design axial force in the column, taken with its minimum value for earthquake design situation ($v_d = N_{ed}/f_{cd}A_c$)
ψ_e	= factor accounting for presence of epoxy coating on bar being developed
ψ_t	= factor accounting for casting position of bar being developed
ψ_s	= factor accounting for size of bar being developed
ρ'	= compression reinforcement ratio of the beam bars passing through the joint
ρ_{max}	= maximum allowed tension reinforcement ratio
σ_{bf}	= frictional bond resistance during cycling, psi (MPa)
$\sigma_{b max}$	= monotonic bond strength, psi (MPa)
Σ_o	= nominal perimeter of the bar, in. (mm)
τ	= average bond stress at failure measured under cyclic loading, psi (MPa)
τ_m	= safety factor = 1.25 as used for offshore structures
τ_{max}	= bond strength achieved under monotonic loading, psi (MPa)

2.2—Definitions

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

CHAPTER 3—BOND AND ANCHORAGE

3.1—Average bond stresses

Average bond stress u refers to the bond stress averaged over a length of bar embedded in concrete for at least 18 bar diameters, and not to the local stress at an individual bar

deformation or along the interface between bar and concrete. The limit of 18 bar diameters is arbitrary and constitutes a lower bound to typical anchorage lengths. However, this limit is important in discussing bond stress because most experimental data available refer to local bond stresses measured over much shorter distances (one to three bar diameters). These data indicate that local bond stresses can be four to five times higher than the average bond stress determined over longer lengths. Various techniques are used to obtain bond measurements, and the results from different research studies may not be directly comparable.

Values for maximum bond stress near ultimate load conditions that have been measured over short distances vary from approximately 1500 to 3000 psi (10 to 21 MPa). Under service load conditions, bond stresses are limited to much lower levels to avoid excessive deflections and localized damage, such as splitting cracks, and other serviceability problems. ACI 318 does not require specific checks for bond stresses. Required development lengths are based on an assumed uniformly distributed bond stress over the entire length of the bars being developed or lap spliced. The lower and upper bounds of corresponding average bond stresses at the ultimate limit state for required development lengths, using the ACI 318 approach for deformed bars in tension, are approximately 70 and 1050 psi (0.5 and 7.2 MPa), respectively. Equation (3.1a) provides the formula for development length in ACI 318.

$$\ell_d = \left(\frac{3}{40} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b \quad (\text{psi}) \quad (3.1a)$$

$$\ell_d = \left(\frac{9}{10} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b \quad (\text{MPa})$$

Equation (3.1a) was derived by rearranging and simplifying the expression for average bond stress (Eq. (3.1b)) proposed by Orangun et al. (1977).

$$u = \left[1.2 + \frac{3C}{d_b} + \frac{50d_b}{\ell_d} + \frac{A_{tr}f_{yt}}{500sd_b} \right] \sqrt{f'_c} \quad (\text{psi}) \quad (3.1b)$$

$$u = \left[0.10 + \frac{C}{4d_b} + \frac{4.2d_b}{\ell_d} + \frac{A_{tr}f_{yt}}{42sd_b} \right] \sqrt{f'_c} \quad (\text{MPa})$$

Equation (3.1b) was developed from a statistical analysis of available data and accounts for the bar stress to be developed, tensile strength of concrete, concrete cover over the bars, bar spacing and size, and amount of transverse reinforcement.

forcement crossing potential splitting planes along the bars being developed.

Equation (3.1a) retains all factors identified by Orangun et al. (1977) as relevant for determining average bond strength in a simplified and more user-friendly format. Furthermore, Eq. (3.1a) accounts for casting position of the bars, effect of epoxy coating, and whether the concrete is normalweight or lightweight. Equation (3.1a) includes an implicit relationship between development lengths and average bond stresses, and thereby provides flexibility for design with bars of different yield strengths and in cases where development of the yield strength of the bar is not required.

3.2—Components of bond resistance

Although the concept of average bond stress is convenient, the total bond force is a combination of resistance due to chemical adhesion (V_a), mechanical anchorage due to bearing of the ribs (V_b), and frictional resistance (V_f), as indicated in Fig. 3.2. Chemical adhesion between the steel and the concrete is lost at low levels of stress in the reinforcing bar. Frictional resistance is associated with micro-irregularities along the surface of the steel, wedging of granular material between the bar and the concrete, and bearing forces oriented perpendicular to the rib face that arise as the bar is loaded. Transverse components of the bearing forces can lead to a bond failure characterized by splitting of the concrete cover.

Typical adhesion values reported in the literature for test specimens subjected to monotonic loading range from 70 to 150 psi (0.5 to 1.0 MPa), whereas reported friction values range from 60 to 1450 psi (0.4 to 10.0 MPa) (Chinn et al. 1955; Eligehausen et al. 1983). It is commonly assumed that bearing forces alone are the primary load-transfer mechanism at load levels resulting in bond failure because contact between the bar and concrete necessary for friction and chemical adhesion may be lost at such load levels. Contrary to this assumption, data comparing the performance of plain and epoxy-coated reinforcing bars under monotonic loads showed that adhesion and frictional resistance may still play a significant role in anchorage failures governed by splitting of the concrete cover (Treece and Jirsa 1987).

Under cyclic loads, most of the bond stress is transferred mechanically by bearing of the bar ribs against surrounding concrete. Tensile and compressive strengths of the concrete, geometry and spacing of bar ribs, concrete cover and bar spacing, and amount of transverse reinforcement are important in controlling bond behavior for this loading case.

Although limited efforts have been made to categorize the components of bond resistance through direct measurements of bond degradation under cyclic loads, nondestructive methods (acoustic emissions and electrical resistance measurement) may further the understanding of bond properties. Acoustic emission monitoring of bond resistance

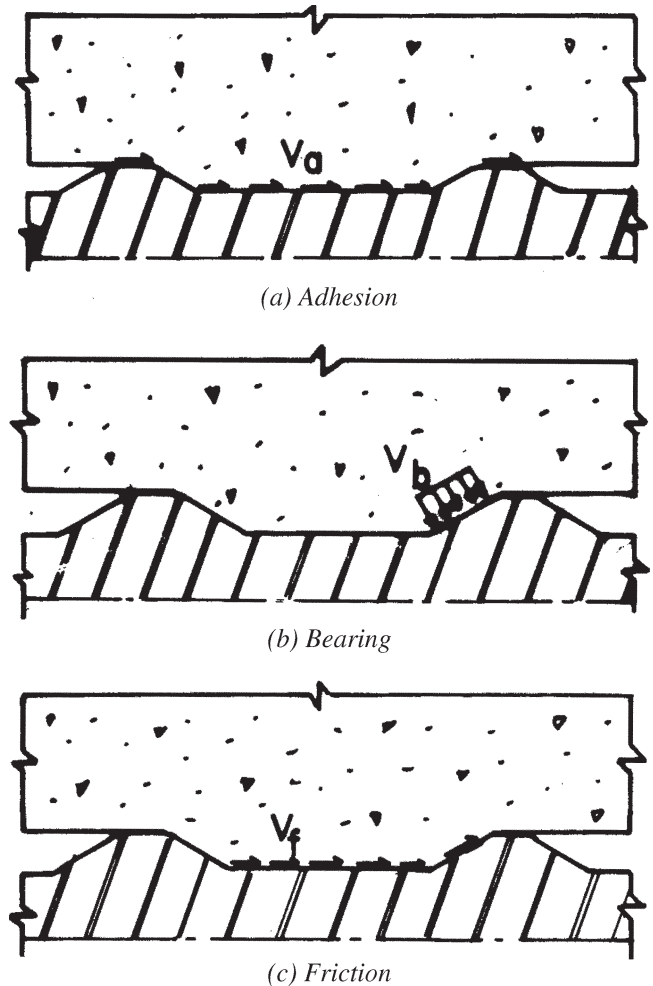


Fig. 3.2—Idealized force transfer mechanisms.

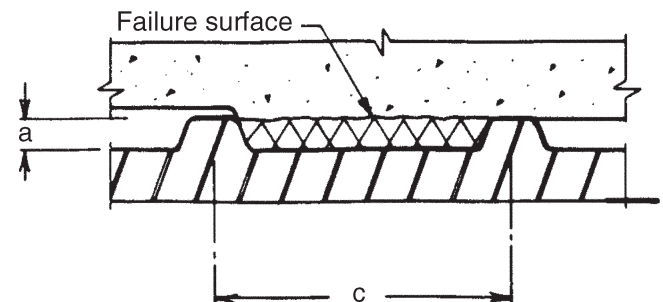


Fig. 3.3a—Pullout failure.

may permit quantification of internal damage at the steel-to-concrete interface, as damage is caused by fatigue under low- and high-cycle fatigue (Balazs 1998). Bond degradation during high-cycle (fatigue) loading has been investigated (Cao and Chung 2001) through measurement of electrical

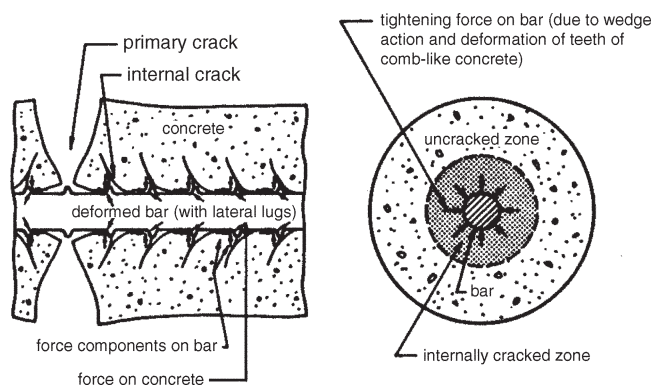


Fig. 3.3b—Splitting failure (Eligehausen et al. 1983).

resistance along the bond interface—a method that may also have application in low-cycle fatigue investigations.

3.3—Failure modes

Under monotonic loading, two types of bond failures are typical. The first is a pullout failure, as depicted in Fig 3.3a, due to the shearing of the concrete between the ribs. This occurs when sufficient concrete cover prevents splitting or where sufficient transverse reinforcement keeps splitting cracks small. Pullout resistance depends primarily on concrete strength, pattern and geometry of bar ribs, depth of concrete cover, and the size and amount of transverse reinforcement.

The second type of bond failure is splitting of the concrete cover. This occurs when cover or confinement is insufficient to attain full pullout strength. As depicted in Fig. 3.3b, splitting failure is due primarily to tensile radial stresses caused by deformation bearing forces. In this case, splitting propagates to the edges of the member, resulting in loss of concrete cover and bond (Brown 1966; Ferguson 1979; Fujii and Morita 1981; Guiriani 1981; Hawkins et al. 1982; Lutz 1970; Lutz and Gergely 1967; Morita and Kaku 1979; Robins and Standish 1982; Robinson 1965; Untrauer and Henry 1965; Watstein and Bresler 1974).

Failure modes under low-cycle loading are similar to those under monotonic loading. Under high-cycle loading, similar failure modes can occur, but fatigue failures of both the bar and concrete need to be considered.

3.4—Bond mechanisms

Bond efficiency can be quantified creating bond stress versus bar slippage curves, which depict the change in local bond stress on the bar (τ) versus total movement of the bar relative to the surrounding concrete. Because there is no direct way to experimentally measure bond stress, most researchers measure the difference in strain between closely spaced strain gauges to compute bond stress. Similarly, slip is measured as the bar movement relative to some fixed reference frame. Bar slip includes three possible components: elastic deformation of the bar, inelastic deformation of the bar, and rigid body motion of the bar relative to the concrete. Only relative bar movement, as opposed to elongation, constitutes slip, but experimentally it is difficult to separate the three components. Slip can be measured either at the end

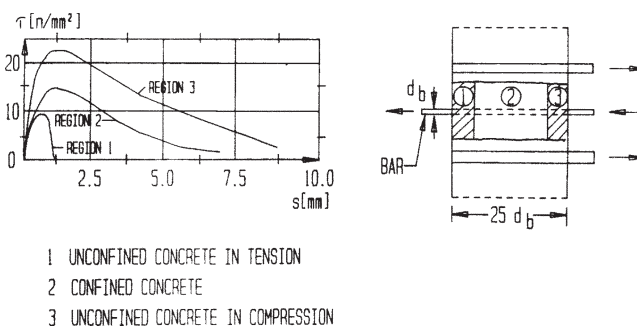


Fig. 3.4a—Monotonic bond stress-versus-slip curve for pullout failure (Eligehausen et al. 1983).

of the bar, or near the application of load. For experimental measurements, gauges should be located and applied so that movement of the bar under load is not affected.

Bond stress-versus-slip curves for a bar loaded monotonically and failing by pullout are shown in Fig. 3.4a (Eligehausen et al. 1983). The relationship shown depends on the degree of confinement and the state of concrete stress surrounding the bar, whether tension or compression. Initially, the curve is steep because of adhesion. Because of shrinkage, small internal cracks always exist immediately adjacent to the reinforcing bar. These cracks act as stress concentrations and initiate crack propagation from the bar ribs at relatively low loads. Small internal bond cracks will begin to form at bond stresses ranging from 300 to 450 psi (2.1 to 3.1 MPa) (Chinn et al. 1955).

Because stress concentrations occur at the front of the ribs, a splitting crack will propagate from the top of the ribs at approximately one-third of the maximum pullout bond stress (Fig. 3.3b). Splitting has occurred between 300 and 1000 psi (2.1 to 6.9 MPa) (Chinn et al. 1955). As loading continues, crack growth leads to a softening of the curve. The rate of crack growth is affected significantly by confining reinforcement. As the maximum bond stress is reached, shear cracks form in the concrete between ribs until the concrete is sheared and bar slip occurs. Due to friction and interface shear, some resistance is maintained even at large values of bar slip. The magnitude of the slip corresponding to maximum bond stress is highly variable and depends primarily on initial bond stiffness (defined as the slope of the bond stress/displacement curve upon initial loading). This, in turn, depends on deformation geometry and casting position of the bar.

The failure process described previously refers to bars that fail by pullout, but bond failures are often governed by splitting of the concrete rather than by pullout. Splitting failure is initiated by the wedging action of the ribs as the bar moves with respect to the concrete. Splitting failures generally occur for bars with three diameters of concrete cover or less, but may occur for bars with more cover. Orientation of the splitting cracks depends on the number and configuration of bars within the member. For layers where bars have a concrete cover less than half the bar spacing, splitting cracks occur parallel to the bar and perpendicular to the surface. For bars in which the concrete cover is greater than half the bar spacing, splitting cracks usually form in the plane of the

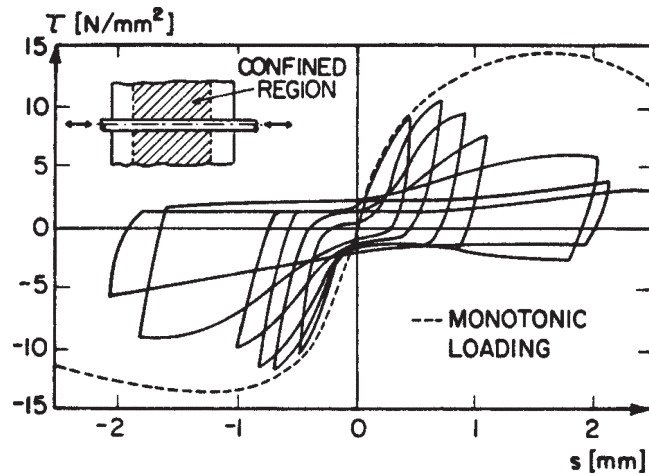


Fig. 3.4b—Cyclic bond stress-versus-slip curve with pullout failure (Eligehausen et al. 1983).

bars. In many cases, however, both types of splitting cracks occur in the same member.

The local bond stress-slip response for the pullout failure of a bar loaded cyclically is shown in Fig. 3.4b (Eligehausen et al. 1983). The initial part of the curve follows the monotonic envelope. When the load is reversed, after the bond stress exceeds approximately half of its maximum value, a significant permanent slip remains when the bar is unloaded. When loading in the opposite direction occurs, the bar must experience some rigid-body motion before bearing in the opposite direction. As cycling progresses, concrete in front of the ribs is crushed and sheared. When the load is reversed, large slip occurs before the bar ribs bear against the concrete and then bond stresses increase. The flat portion of the curve has been the source of much concern in the design of earthquake-resistant structures, especially where large bond stresses must be mobilized. In moment frame structures, where most of the joints have been subjected to inelastic load reversals, bond loss in beam-column joints can lead to large drifts. The maximum sustained bond stress under reversed cyclic loading follows a reduced monotonic bond stress-slip envelope (Balazs 1991). This reduction is a function of the load history, the number of reversed load cycles, and the maximum previous slip.

The slip increase due to constant amplitude cyclic loading tends to decrease after initial loading cycles. The Palmgren-Miner Hypothesis states that damage accumulates linearly with the number of cycles applied at a particular load level. In the context of this document, unless otherwise noted, the term “damage” refers to the degradation of bond due to the effect of cyclic loading. In the case of deformed bars, the degradation of bond is caused by crushing of the concrete in the area of the ribs (Rehm and Eligehausen 1979) or splitting of the concrete. As indicated in Fig. 3.4c, Balazs (1991, 1998) found that damage accumulates more rapidly in the initial and final phases of pullout failure. Between initial and final phases, constant-amplitude cyclic loading results in a constant rate of slip until the slip value corresponding to monotonic bond strength is reached. Under further cyclic loading, slip increases rapidly until pullout failure

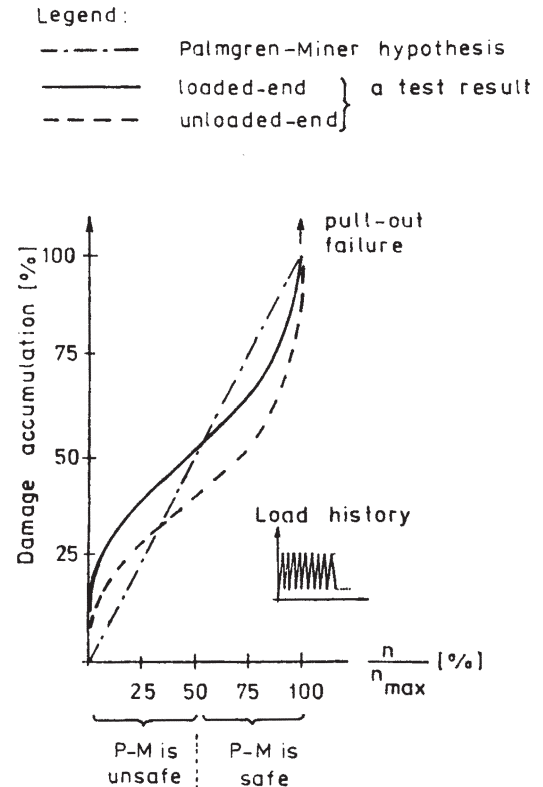


Fig. 3.4c—Bond damage accumulation with respect to the Palmgren-Miner Hypothesis (Balazs 1991).

occurs. Slip progression for reversed cyclic loading is more pronounced than for constant-amplitude cycles.

For variable or random cyclic loading, only load levels approaching or exceeding the previous highest level induce considerable slip increase, and the slip contribution of decreasing load levels is minimal (Balazs 1998).

3.5—Factors affecting bond strength under cyclic loads

The main factors influencing bond strength under cyclic loads are as follows.

3.5.1 Concrete compressive strength—The tensile and shear strength of concrete are seldom measured directly but have a strong correlation to the square root of the compressive strength (f'_c) for concretes having f'_c less than 10,000 psi (70 MPa) (Carino and Lew 1982; ACI 363R). Because force is transferred by bearing and bond, and failure can occur by tensile splitting and shearing of the concrete, compressive strength is a key parameter in bond behavior (Orangun et al. 1977).

Bond strength increases with an increase in compressive strength (Alavi-Fard and Marzouk 2002). Additionally, they report that for concrete with compressive strengths of 7000 to 14,000 psi (50 to 95 MPa), the reinforcement bond is proportional to the cubic root of the compressive strength, and the square root of the compressive strength is not the ideal representation.

3.5.2 Concrete cover—The type of bond failure observed under monotonic loading in laboratory tests is strongly dependent on the concrete cover and bar spacing (Eligehausen et al. 1983).

hausen 1979; Orangun et al. 1977; Tepfers 1973). When concrete cover is small, a splitting tensile failure occurs at a shorter anchorage length than that required for a pullout, or bond shear, failure. This effect is probably more important for cyclic than for monotonic loading because more severe cracking will probably occur and load repetitions will increase the energy available for crack propagation.

3.5.3 Bar size—To develop the same force in two bars with different sizes and the same development length, the large bar develops a lower bond stress than the small bar. The opposite is true if the aim is to develop the same bond stress in bars of two different sizes within a given development length. For improved bond performance, it is more effective to use many small bars rather than to use few large bars when reasonable clear distances between bars can be maintained. Although evidence shows that the apparent maximum bond strength decreases as bar size increases (Rehm and Eligehausen 1979), analysis of data in the ACI 408 Database indicates that accounting for an increased bond strength for smaller-diameter bars is not justified (Zuo and Darwin 2000). Because the tests in this database were performed under monotonic loading, the effect of bar size on bond performance of reinforcing bars subjected to reversed cyclic loads needs further study.

3.5.4 Anchorage length—Under monotonic loading, increases in bond force at failure are not proportional to increases in anchorage length. The maximum bond stress decreases with increasing anchorage length. Under cyclic loading, however, the number of cycles to a pullout failure should increase as anchorage length increases. There is no conclusive data available to develop guidelines in this area.

3.5.5 Deformation (transverse rib) geometry—Relative rib area, or ratio of transverse rib bearing area (area of ribs perpendicular to the bar axis) to transverse rib shearing area (perimeter of the ribs times rib spacing), has had a significant effect on initial bond stiffness and, thus, on bond performance. Bond strength and stiffness increase as the relative rib area increases (Balazs 1986; Eligehausen 1979; Rehm and Eligehausen 1977, 1979; Soretz and Holzenbein 1979). Typical values of this ratio vary from 0.05 to 0.08. However, high-relative rib area bars, defined as bars with relative rib area ratios between 0.1 and 0.14, inclusive, are produced and used more today than in the past. Reverse cyclic testing indicates that increasing relative rib area from 0.085 to 0.119 reduces unloaded end slip by 50 to 70 percent and loaded end slip by 30 to 40 percent (Zuo and Darwin 2000).

In addition, bar fatigue performance (high-cycle loading) is related to the radii at the bases of transverse reinforcement ribs due to stress concentrations at those radii. Transverse ribs normal to the bar axis reduce fatigue life in comparison with bars where ribs are angled to the bar axis (Zheng and Abel 1998).

3.5.6 Steel yield strength—Because bond stress is directly related to the force in the bar, using lower-strength steel may reduce the bond stress demand under monotonic loading. Most bars used today have nominal yield strengths from 60 to 70 ksi (410 to 480 MPa). Under monotonic loading, bond stress demand can be reduced by using bars with 40

ksi (280 MPa) yield strength. This is especially true under low-cycle loading where several reversed yielding cycles may be experienced. However, using lower-yield-strength bars may not improve bond strength because using lower-yield-strength steel requires using more or larger bars to achieve the required tensile force for a section. Using more bars results in closer spacing and reduces developed bond stress. For structures detailed for large load reversals, severe reinforcement congestion may cause difficulties in casting and achieving development. Strain hardening of the reinforcement under high-stress cycles should be considered, as it may result in larger bond stress demands than anticipated in the original design.

3.5.7 Amount and distribution of transverse steel—Splitting tensile failures are strongly dependent on amount and distribution of transverse reinforcement (Eligehausen 1979; Orangun et al. 1977; Tepfers 1973). Properly detailed transverse reinforcement confines the concrete after cracking and significantly increases the resistance to splitting failure, especially under cyclic loads. The upper bound to this improvement is the pullout failure of the bar.

3.5.8 Casting position, vibration, and revibration—Casting position and the effects of vibration and revibration can significantly influence bond strength under monotonic loading (Abrams 1913; Altowaiji et al. 1986). Proper consolidation of the concrete around the reinforcing bar is required for optimum bond strength. Consolidation is difficult to ensure in the field, particularly when bars are placed near the top of a deep member. In this case, voids and water pockets will form under the bar as the concrete flows during placement, and bond strength will decrease. In addition, tensile strength of the concrete near the top of a placement lift may be reduced if bleedwater significantly increases the local water-cement ratio (w/c).

3.5.9 Strain (or stress) range—Under cyclic loading, the amount of cumulative damage and the number of cycles at pullout are influenced primarily by the stress range (Bertero and Bresler 1968; Holmen 1982). For bond, both the stress range and strain (or slip) range are important (RILEM 1986), and evidence shows that the minimum stress level influences fatigue failure of bars.

3.5.10 Type and rate of loading—Research data indicate that the type of cyclic loading, whether fully reversed or one-directional, may have a large influence on bond performance (Rehm and Eligehausen 1977). Test data also show that a fast loading rate can result in significantly higher bond strengths and less damage in anchorages than a slow loading rate (Shah and Chung 1986). Concrete compressive, tensile, and shear strengths increase with the rate of loading. Loading frequency also affects the cumulative concrete damage under cyclic loading.

3.5.11 Temperature—Low temperatures can significantly increase bond strength. The strength gain and deterioration mechanisms at cryogenic temperatures can be explained by fracture mechanics of concrete (Shih et al. 1986, 1988). When a bar is subjected to multiple temperature cycle reversals, the capacity to store and dissipate energy through the interface is reduced under cyclic loading (Shih et al. 1986, 1988).

This is reflected in the shape of the bond stress-slip relationships. Conversely, a loss of bond strength can be observed at elevated temperatures or after the concrete member has been exposed to fire (Diederichs and Schneider 1981; Morley and Royles 1983).

3.5.12 Surface condition and coatings—Evidence indicates that the bond strength of bars decreases when bars are coated with epoxy or similar substances. Monotonic tests show that pullout strength decreases approximately 15 percent, and splitting strength decreases by as much as 35 percent (Johnson and Jirsa 1981; Johnston and Zia 1982; Treece and Jirsa 1987). Tests indicate that epoxy coatings on bars reduce adhesion and friction, and lead to large splitting forces.

High-cycle testing of splices indicates that epoxy-coated bars have a smaller average bond stress at failure than the average bond stress of uncoated bars. This reduction ranges between 0.67 and 0.88, depending on concrete cover, bar size, deformation pattern, coating thickness, and confinement (Cleary and Ramirez 1993). A cover-to-bar-diameter ratio over 3.0 requires no additional precautions beyond those suggested by ACI 215R and ACI 408R in designing members with epoxy-coated bars for high-cycle fatigue or repeated loading in the service load range usually encountered in bridge decks (Cleary and Ramirez 1993). Accordingly, the larger cover-to-bar-diameter ratios recommended in harsh environments should not be reduced with the expectation that the epoxy coating will protect the reinforcing bars (Cleary and Ramirez 1993).

Epoxy-coated reinforcing bars under repeated loading does not result in significantly increased deflection despite the increased bar slip that is indicated by larger flexural crack widths (Cleary and Ramirez 1993; Hasan et al. 1996). Fewer cracks negate increased rotation and, thus, deflection, due to additional initial slip. The loss of deformation area due to extra thick epoxy coating can be critical during repeated load application if stresses are close to the upper bond limit (Cleary and Ramirez 1993).

3.5.13 Lightweight aggregate concrete—Research indicates that the fatigue strength of lightweight-aggregate concrete is similar to or better than that of normalweight-aggregate concrete of similar strength properties (Mor et al. 1992). Adding silica fume to lightweight concrete improved bond strength by 100 percent and improved fatigue life by over 60 percent. No significant improvement was observed when silica fume was added to normalweight concrete.

Research indicates that bond strength for high-strength lightweight concrete is not significantly affected by cyclic loading, provided the maximum cyclic displacement is less than the peak load slip in monotonic tests (Mitchell and Marzouk 2007). Once displacement exceeds this peak load slip, however, bond strength rapidly deteriorates.

3.5.14 Corrosion—Research by Fang (2006) indicates that the maximum corrosion level, in terms of mass loss, that improves cyclic bond is approximately 5 to 7 percent. Significant corrosion provides the most bond improvement in the first five cycles. Subsequent bond deterioration occurs as load cycles increase.

3.5.15 Alkali-silica reaction—The fatigue life of tension reinforcement within a lap splice is reduced when alkali-silica reactions are present due to a bond reduction resulting from expansive forces within the concrete (Ahmed et al. 1999).

3.5.16 Fiber-reinforced concrete—Experimental evidence suggests that introducing steel or polymer fiber reinforcement into concrete improves both maximum and residual bond strength (Krstulovic-Opara et al. 1994; Hota and Naaman 1997; Parra-Montesinos et al. 2005; Harajli 2005). Recent research on evaluating the bond of reinforcing bars embedded in fiber-reinforced cement composites (FRCC) showed that FRCC, compared with concrete with conventional transverse reinforcement, significantly enhances the bond performance in terms of bond strength, stiffness, and ductility under cyclic loading. This is particularly true for FRCC, which exhibits strain-hardening behavior in tension (Chao 2005). In concrete members confined by spiral or rectangular hoops, a minimum lateral expansion, which leads to cracking, is required in the cementitious matrix before transverse reinforcement effectively provides confinement. These initial cracks may reduce the bond contribution from friction and mechanical interlocking, especially under reversed cyclic loading. Alternatively, the confinement and bridging effects provided by fibers in FRCC after cracking can increase the tensile stress and strain capacity of the concrete matrix and limit crack width, which leads to increased bond strength of reinforcing bars embedded in such composites.

Parra-Montesinos et al. (2005) tested large-scale beam-column subassemblies constructed with strain-hardening FRCC in the connection and adjacent beam regions. No confinement (transverse) reinforcement was used in the connection region. The researchers reported that no signs of deterioration of bond between longitudinal beam bars and surrounding strain-hardening FRCC material were observed in the test specimens, even though the column depth represented 18.7 beam bar diameters and the beam bars were subjected to large inelastic strain reversals. A peak average bond stress of approximately 1500 psi (10 MPa) was developed in beam longitudinal bars with no noticeable reinforcement slip.

3.5.17 Bond in grouted ducts used in precast structures—Raynor et al. (2002) addressed the bond behavior of bars grouted in ducts used to connect precast concrete frames subjected to cyclic loading. They proposed a relationship between local bond stress and slip, and this relationship was used to develop a design equation for the equivalent debonded region used in the connection of precast concrete frame elements.

3.6—Summary

A single, simple model cannot represent the complex problem of bond strength under cyclic loading. Many factors affect bond strength under cyclic loading, and their influence on bond strength and failure mechanisms is understood only qualitatively in many cases. The following chapters summarize the research available for bond strength under high- and low-cycle fatigue and present models for bond strength under cyclic loading. This report describes the research

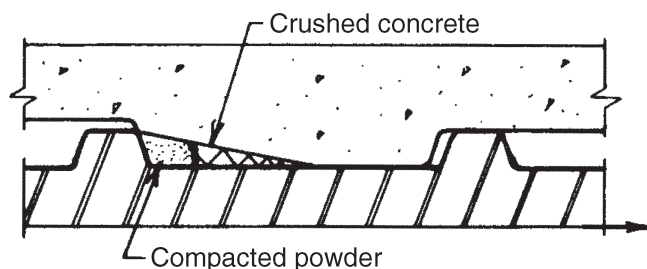


Fig. 4.1a—Crushing of concrete in front of ribs.

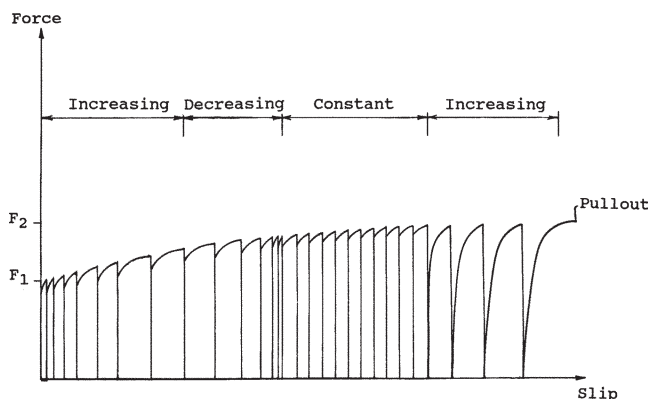


Fig. 4.1b—Typical bond fatigue failure (Balazs 1986).

efforts that are the most complete and accessible. Existing design recommendations and possible additional guidelines for design are discussed.

CHAPTER 4—BOND BEHAVIOR UNDER CYCLIC LOADS

Repeated loads can reduce bond strength at failure and accelerate the rate of bond deterioration as the stress range or the number of cycles increase. The following sections discuss the mechanisms that influence bond deterioration under cyclic loads.

4.1—High-cycle fatigue

Fatigue is a progressive permanent internal structural weakening of a material subjected to repetitive stresses (ACI 215R) and can lead to failure of a material at a stress lower than the maximum stress under monotonic loading. Fatigue loading on a specimen or member of a structure may be thought of as a load sine wave that varies periodically, such as a superimposed on a static load. The number of cycles to failure is a function of both the static load and the varying superimposed load. The most severe fatigue loading occurs when the mean static load is zero and the varying load causes alternating tension and compression. When the magnitude of reversed loading exceeds the yield strength of the material, then alternating plasticity produces failures after a few load cycles.

Analysis of systems under fatigue loading is complicated by the changes in material properties that occur over time. Conditions in a concrete member change over time due to the interactive static and dynamic influences of creep, shrinkage, temperature, and the applied stress range.

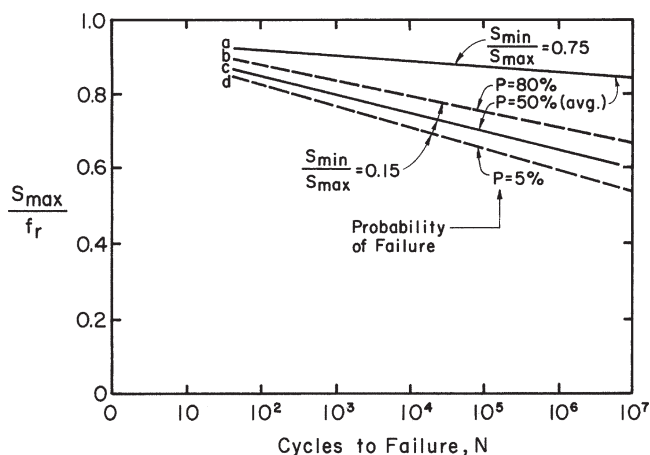


Fig. 4.1c—S-N curve for plain concrete (ACI 215R).

Therefore, comprehensive design guidelines for fatigue loading are difficult to derive. Recent data indicate that the maximum bond strength under fatigue loading relates directly to internal concrete damage. Thus, repeated loads have a similar influence on the bond strength and slip as on deformation and failure of unreinforced concrete (Rehm and Eligehausen 1979). The main mechanism of bond deterioration seems to be progressive crushing of concrete in front of the ribs, which is depicted in Fig. 4.1a. Figure 4.1b indicates four behavioral stages apparent in most bond fatigue tests.

1. A fast increase in slip due to initial crushing of the concrete
2. A rapid reduction in the slip rate due to the stabilization of the process
3. A long portion with a constant slip rate
4. A fast increase in the slip rate as the failure approaches.

Such a response is typical of a pullout failure. A splitting failure would result in a sudden drop in load-carrying capacity.

Fatigue test data are commonly analyzed using stress-versus-fatigue life diagrams, commonly called *S-N* diagrams. In these diagrams, applied stress or stress range (*S*) is plotted against the number of cycles to failure (*N*); *N* is usually plotted on a logarithmic scale. Figure 4.1c indicates the fatigue resistance of plain concrete beams subjected to flexural loading. The ordinate is the ratio of maximum flexural tensile stress (S_{max}) to the static modulus of rupture (f_{rup}). The abscissa represents *N* plotted on a logarithmic scale. Solid lines represent the fatigue life of the concrete when subjected to two different stress ranges relative to S_{max} . For a given number of cycles, a smaller stress range, indicated by a larger value of S_{min}/S_{max} in the figure, corresponds to increased fatigue strength. Each solid line represents a 50 percent probability of failure for the given stress range. Dashed lines represent different probabilities of failure for $S_{min}/S_{max} = 0.15$. For reinforcing steel, the *S-N* curve in Fig. 4.1d indicates that, regardless of the number of cycles, fatigue does not appear to be a problem below a certain stress range. This is the ideal working stress range for high-cycle fatigue applications, and design guidelines can be developed for this area of the *S-N* curves.