# Evaluation of Bond Performance in Reinforced Concrete Structures

by S. L. McCabe and S. J. Pantazopoulou

**Synopsis:** The increasing significance of performance-based criteria in modern structural design has motivated new considerations in bond design of conventional reinforcing steels, relating to more reliable assessment of both the demand and the supply sides of the anchorage/development design problem. Accurate identification of the required anchorage lengths needed to ensure strain compatibility, by proper consideration of the conditions affecting bond, is necessary to limit slippage of the steel relative to the concrete. While minimum development lengths calculated by designers imply that the bar is fully anchored, it is well established by experimental observation that in practice there is always some bar slip.

Recent research results from around the world provide the basis for improved understanding of the effects on bond performance of critical parameters such as confinement, spacing, and material properties. Much of this work has been empirical in nature and the applicability of empirical design expressions in calculations is limited. Nonlinear finite element calculations and other sophisticated analysis requires more information as to how the bond failure proceeds than simply an upper limit.

This paper will summarize the available information that exists both within North America through ACI and within the CEB as to the viable approaches and philosophies that can be applied to the bond problem. The range of application of the various techniques will be identified as will limitations and needs for more research.

Keywords: Anchorage; bond; bond models; design; detailing; development; finite element method; plasticity; reinforcement

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## INTRODUCTION

In the traditional capacity design approach used in North America for reinforced concrete structures, designing for bond was considered to be a question of providing adequate strength along the bar anchorage for development of the bar yield force. Therefore, bond, as well as all other tension-failure controlled problems such as shear, were dealt with in design by establishing allowable stress limits for concrete, with stresses calculated from an admissible state of equilibrium without consideration of deformations. When needed, calculations of ductility capacity were performed after dimensioning and were associated with purely flexural response. Hence, of the two coordinates in the load-deformation response recorded in anchorage tests, the ACI code writing bodies have focused only on the load capacity side (with particular reference to yielding of reinforcement). The usual approach taken in the past was to synthesize a summary of experimental data for bond strength using empirical relations that subsequently formed the basis of the design requirements. These empirical relations attempt to describe the parametric dependence of bond strength on an array of important design variables. Although this process is fraught with debate, within ACI the main point of concern has really only been in selecting the form of the design expressions. The array of significant variables used, as well as the underlying philosophy of the approach, is generally accepted as a necessary accessory of the capacity design framework. Note that whereas deformation (slip) was universally acknowledged in the reported tests which formed the basis of current design requirements for bond, no systematic attempt had been made to process the available information in a manner analogous to the efforts placed on the side of strength.

The emergence of new performance based design approaches have refocused the problem of bond, since at the heart of these approaches lie considerations about deformation demand and supply, which inevitably bring forth questions about compatibility of deformations between concrete and the anchored bar throughout the various stages of response. For the design to meet certain predetermined performance requirements, it may no longer be adequate to simply assess the strength of a given anchorage without at the same time knowing the levels of deformation that would have to develop in order for the given strength to be realized, and whether these deformations can be realized in the structure without completely altering the assumed load path (redistribution). For this reason, code writing bodies which have already explicitly adopted performance-based design approaches, such as the CEB, require modeling of the complete load deformation envelope of the bond-slip response of a given anchorage as reference for design purposes. Complete bond-slip relationships have been obtained either by empirical modeling of tests, or by analytical (mechanistic, numerical, etc.) methods. These approaches will be critically reviewed in the following sections with the objective to identify the salient features, range of applicability and common points between alternative methods. The scope of the review is limited to straight bar anchorages and will concentrate on monotonic loading, although many of the observations that can be made are common to both monotonic and cyclic loading.

## REVIEW OF AVAILABLE MODELING APPROACHES

The array of important variables controlling the mechanics of bond both directly or indirectly appears from familiar experimental evidence to be quite extensive and has not been completely defined as of yet. The parametric complexity of the problem discourages formulation of straightforward mathematical descriptions. Even when restricting the scope to straight bar anchorages under monotonically increasing loads, the complexity of the problem persists: Position of the bar at casting, bar profile (rib geometry), transfer length, material properties, direct or incidental confinement of the surrounding concrete, and geometric boundary conditions all seem to not only have a profound influence on the shape and coordinates of the load deformation response, but also determine the mode of failure (splitting, pullout, or combination of the two modes).

Depending on the type of idealization and simplifying assumptions used in modeling the load deformation envelope for bond, published models can be classified as follows:

(a) mechanistic vs. empirical models
(b) global versus localized models
(c) splitting vs. pullout models
(d) numerical or hand-calculation models

## CODE APPROACHES AND MODELS

The CEB design model and all ACI-related models are global, hand-calculation approaches: the influence of bond is evaluated by considering the free body equilibrium of a segment of the anchored reinforcing bar (Fig. 1). The critical section is at one end of the segment, at which point the applied bar stress is known. A distribution of bond stresses along the segment is postulated based on experimental results. The magnitude of bond stress is obtained from local bond stress - slip relationships that are often referred to as constitutive laws for bond. Note that this term is necessarily used loosely in this context, as bond is not an intrinsic physical property but the mechanical response of the bar - concrete assembly under certain specific boundary conditions. The force in the other end of the bar segment is obtained directly from equilibrium considerations. If the characteristic stress-strain relationships of steel have a positive definite tangent stiffness throughout the strain range, then it also is possible to obtain the deformations and the distribution of stresses along the anchored bars.

(1) Models Adopted by ACI Committees 318 and 408:

Various ACI committees (408, 318) have adopted several simplified versions of the above model over the years. In all cases the length of the segment considered involves the entire anchorage length, $L_d$, whereas the known stress input at the critical section is usually the yield or ultimate stress capacity of the bar ($L_d$ corresponds to the length required for the stress in the bar to become zero at the unloaded end). The intensity of bond stresses $u_b$ is taken as constant along the anchored length, set equal to an allowable maximum value multiple of the square root of $f'_c$ in order to acknowledge the fact that bond failure is associated with tension failure of concrete in the principal directions. Note that by definition, bond stress is proportional to the rate of change of bar stress along the anchored length, therefore as a variable it has only local physical significance. Therefore the assumption of constant bond stress distribution is a simplifying idealization adopted for the sake of mathematical convenience. Experimental evidence suggests that whereas this assumption leads to acceptable results for normal weight concrete and uncoated bars, it cannot adequately model the behavior of epoxy coated bars, or of bars anchored in high strength concrete. (In these cases the bond stress distribution appears to be highly nonlinear with peak intensity at the loaded end of the bar.)

The apparent advantage of the ACI approaches is two fold. First, despite the overall empiricism inherent in the assumptions made, there is an underlying physical model, which explicitly establishes equilibrium in order to define the required $L_d$ (in design) or in order to define allowable values for $u_b$ (from experiments, where $L_d$ is known). Second, the resulting expressions are simple enough for routine hand calculations. The obvious disadvantages of this approach are, first in the underlying assumptions of constant bond stress, which, as was mentioned in the proceeding, are unrealistic in the case of high strength concrete or epoxy coated steel. Secondly, the model provides no detailed information about the behavior of the bar under load; rather, the result is a binary answer as to the adequacy of the anchorage length for the development of the yield strength of the reinforcement. In a performance driven design framework it is necessary to predict structural behavior at specified load demands. Here it is not sufficient to know whether the anchorage length can or cannot develop the bar force capacity. For example, if the member is stressed up to the yield point or slightly beyond yield, then the bar slip relative to concrete will be limited and perhaps not important. In contrast, if the system is subjected to large inelastic deformations, then the demands on the bar and surrounding concrete will be such that significant bar slip may develop. This slippage will result in reduced apparent stiffness of the member and reduced energy dissipation capacity in the overall system. The assumption of perfect bond leads to higher estimates of the structural stiffness and to lower displacements than the actual values, the more so for systems undergoing cyclic load reversals. Sozen (1974) observed that the plastic rotation of a concrete joint may be doubled due to bond slip. Because this amount of increase in rotation affects significantly the story drift, it is important in design to limit bar slip to small values. In the ACI 408 report (1992), it is recommended that bond stresses be limited to about 80% of the ultimate values when designing for cyclic loads. The objective of this recommendation is to limit the demand on the bars – and consequently the intensity of bond stress – to values below the levels that would cause deterioration of the surrounding concrete.

<u>The ACI Design Equations for Bond</u>: Over the recent years, ACI design expressions have progressively evolved from simple formulae to the more complex but also more accurate expressions of today. The process was primarily driven by an ever-expanding database of published experiments. Typically, best-fit lines have been used to describe the relationship between test parameters and performance. However the procedure was packaged, the resulting design expressions were intended to ensure (based on the experimental evidence) that the anchorage length was sufficiently long to fully develop the bar nominal yield force, while maintaining the bond stresses below allowable limits. The reason why converging to the design expressions of today has taken several iterations is that it has been shown over time, that the experimental values are sensitive to specimen shape and support conditions during testing. The perceived significance of other conditions, such as position at casting, has never been properly linked to physical properties of concrete such as porosity and composition. With such vital information missing from the database, and with the ongoing emergence of new qualities of concrete materials, the relative significance of

this parameter on the anchorage length is continually changing. An added layer of complication in the process of deriving design expressions from a collection of tests has been related to the mode of failure of the tests considered; note that whereas the most challenging aspect of bond design is to protect against splitting failures, until recently the majority of available tests in the literature had ended as pullout failures. Uniform specifications for designing and testing of bond specimens have been proposed only recently by the research community.

Early expressions for bond simply divided the yield force by the bond area surrounding the bar:

$$u_b = \frac{A_b f_s}{\pi d_b L_d} = \frac{d_b f_s}{4L_d} \tag{1}$$

In the ACI 318-56 Building Code, the limiting bond stress was taken as 0.10 $f'_c$ for bottom deformed bars and 0.07 $f'_c$ for top bars, with an upper limit on cylinder strength of 3500 psi. Thus an average bond stress of 350 psi (2.41 MPa) was assumed. Ferguson (1965) recommended the use of even lower bond stresses than permitted by ACI 318 in applications of low cover (i.e., a limit of 0.05 $f'_c$ for smaller sized bars and 0.04 $f'_c$ for No. 9, 10 and 11 bars). For Grade 60 (420 MPa) No. 8 bar embedded in 4000 psi (30 MPa) concrete, a development length of 30 inches (762 mm) would be required by this formula at a bar stress of 24 ksi (170 MPa) – the typical value employed in allowable stress design in 1956. Thus, here was the source of the 30 bar diameter rule of thumb used for many years in concrete design and construction in North America.

This method for computing development lengths was used until the early seventies when strength design replaced the allowable stress design. A primary study that drove the development of improved equations was the work of Orangun, Jirsa and Breen (1975, 1977). Design was based on the following empirical relationship:

$$u_b = \left[1.2 + \frac{3C}{d_b} + \frac{50d_b}{L_d} + \frac{A_{tr} f_{yt}}{500s\, d_b}\right]\sqrt{f'_c} \qquad \text{U.S. Customary Units} \tag{2}$$

$$u_b = \left[0.1 + 0.25\frac{C}{d_b} + 4.15\frac{d_b}{L_d} + \frac{A_{tr} f_{yt}}{41.52s d_b}\right]\sqrt{f'_c} \qquad \text{S.I. Units} \tag{3}$$

The above equation models bond strength as a linear function of bar diameter. The term $3C/d_b$ reflects the confining influence of cover and the negative effects of close bar spacing on bond. The square root power of $f'_c$ is used in order to indicate that bond strength is a function of the tensile capacity of concrete and increases at a slower rate than cylinder compressive strength. Passive confinement is accounted for in a separate term that is proportional to the confining steel area. After simplification of terms, and by ignoring the confinement term, the above equation may be written in terms of the required development length:

$$L_d = \frac{0.04A_b f_y}{\sqrt{f'_c}} \qquad \text{U.S. Customary Units} \tag{4}$$

$$L_d = \frac{0.02A_b f_y}{\sqrt{f'_c}} \quad \text{S.I. Units} \tag{5}$$

Rearranging this equation provides a bond stress relationship of

$$u_b = \frac{8\sqrt{f'_c}}{d_b} \tag{6}$$

with an upper bound on bond stress of 625 psi (4.31 MPa) using ACI 318 limits.

Similar work by ACI Committee 408 has led to the following equation for the required development length of Grade 420 (60 ksi) reinforcement:

$$L_{db} = \frac{5500A_b}{\phi K\sqrt{f'_c}} \quad \text{U.S. Customary Units} \tag{7}$$

where K is the smaller of

(a) $0.5d_b + C_c + K_{tr}$ or

(b) $0.5d_b + C_c + \left[\frac{\Sigma K_{tr}}{n}\right]$; but no larger than $3d_b$

In the ACI 318-95 Building Code, the bond expressions have been reevaluated and restated in terms of bar diameter. The basic equation is,

$$L_d = \left(\frac{3d_b f_y}{40\sqrt{f'_c}}\right)\left(\frac{\frac{\alpha\beta}{(c+K_{tr})}}{d_b}\right) \quad \text{U.S. Customary Units} \tag{8}$$

For standard configuration, this equation reduces to

$$L_d = \left(\frac{d_b f_y}{20\sqrt{f'_c}}\right)(\alpha\beta) \quad \text{U.S. Customary Units} \tag{9}$$

for No. 7 bars and larger with at least one bar diameter of cover, c, and minimum stirrups provided, $K_{tr}$. The coefficient in the denominator is 25 for smaller bars. Using an upper bound of 10,000 psi (69 MPa) for the compressive strength of concrete, the larger bar series has a required development length of 30 bar diameters. This corresponds to a bond stress of 500 psi (3.45 MPa).

From the preceding review it is evident that the ACI philosophy over the years has been consistent. The objective has been to limit bond stresses to values in the range of 500 - 600 psi (3.45 – 4.14 MPa), which are considered conservative based on experience from experimental studies. For different configurations in terms of bar spacing, cover or confining steel, adjustment factors are used to represent the influence of these parameters on the required development length. For example, the

$K_{tr}$ term in the ACI 318-95 equation and the associated cover terms represent these factors. This approach was calibrated with research results and was originally developed by the ACI Committee 408 (1990).

In curve-fitting the available database of bond tests, the ACI 408 approach has been to include a $\phi$ factor in the analysis to account for localized uncertainties in the material properties and configuration. A $\phi$ value of 0.80 in the denominator is used to increase the required development length. On the other hand, the ACI 318 approach has been to avoid compounding the $\phi$ factors that appear in the bond expression and in flexure, shear or other applications. For this reason, the 318 expressions have a factor close to 1.0. It was mentioned in the introduction that an explicit relationship for bond stress vs. slip is not explicitly addressed in the ACI documents relating to bond. This is justified, since the overall approach of the ACI philosophy is to provide rules to preclude significant amounts of plasticity; in this range of response, detailed mechanistic models are not essential. Elaborate analytical models for bond have been developed by the research community, but are mostly used in the study of problems involving a significant degree of nonlinearity, and fall beyond the scope of the ACI framework of design.

(2) The CEB Approach in Modeling the Problem of Bond:

The CEB Model Code (1990) has taken a somewhat different approach to design: it is required that performance be checked within defined limit states. Two primary limit states are considered, the Service Limit State (SLS) and the Ultimate Limit State (ULS). The SLS represents a comparable state to that implied by the serviceability checks within the ACI 318 Building Code. Straightforward design rules are provided; the design objective is to ensure minimum levels of performance for crack control, deflections and other items.

The ULS represents a plastic limit state where the structure and its components are assumed to undergo significant inelastic deformations. Additional checks are needed to ensure the design satisfies pertinent performance and capacity requirements. The use of computer-based analysis, and in particular the Finite Element Method, is a hallmark of this approach.

A significant example of the differences between the SLS and ULS is that of moment redistribution. When considering the ULS, the Model Code allows up to 50% of the negative moment to be redistributed to the positive part of the moment diagram. Such levels of redistribution are not allowed by the ACI 318 code, where the maximum redistribution is 10%.

The moment redistribution provisions have an important corollary and that is the basic performance of the reinforcing steel. European reinforcing steels are generally comparable to ASTM A 706 steel. They are weldable, low-alloy steels of good quality and material traceability. The significant difference is the relative lack of strain hardening in these Euro-steels as compared to U.S. steels. For example, a Class A steel with a 500 MPa nominal yield has a required ratio of tensile strength to yield strength of 1.08. Within ASTM-A 706 it is required that the ratio of ultimate to yield strength be a minimum of 1.25. From a design perspective this means that a member forming a plastic hinge will have a relatively constant amount of moment capacity as the hinge is loaded and rotation occurs. In a similar situation but using U.S. steel the moment capacity will increase substantially as the steel hardens. The implication is that the increase in capacity will provide additional strength when needed under severe demands. In addition, considerations prevail relating to increased need for confinement as compared to the first onset of yielding as the internal loads are changing within the member. Therefore, the different amounts of moment redistribution permitted by the two codes complement the relative characteristics of the reinforcing steels considered in the two cases.

In practical applications forces and moments within the structure may be computed either by nonlinear analysis, linear analysis, linear analysis with redistribution, or by plastic analysis. Each of these options has its own set of requirements and models that must be followed. It is not the intent of this paper to restate the entire Model Code; however it is evident that the Model Code is compatible with many alternative approaches of analysis and design.

Compared to the ACI approaches, another source of difference is the application of factors both to increase the level of loads on the structure for design (load factors) but also the reduction of material strengths by other factors. The design strength of reinforcing steel is defined as

$$f_{ycd} = \frac{f_{ytk}}{\gamma_s} \tag{10}$$

where the value of the partial factor $\gamma_s$ is 1.15 for sustained loads. In the specific case of bond and development, similar comments can be made. The CEB requires that the designer check the bond stress to ensure that levels are not excessive.

<u>CEB Design Rules</u>: The CEB defines the basic length to transfer the yield force of a bar as

$$L_d = \frac{d_b f_{yd}}{4 f_{bd}} \tag{11}$$

where $d_b$ is the bar diameter in mm, and $f_{yd}$ is the bar design yield force. Term $f_{bd}$ represents the design bond stress of concrete, given by

$$f_{bd} = \eta_1 \eta_2 \eta_3 f_{t'd} \tag{12}$$

where $\eta_1$ is a geometry factor taken as 2.25 for ribbed bars, $\eta_2$ is an orientation factor for bond (a value of 1.0 is used in most cases). $\eta_3$ is a bar size factor set at 1.0 for 32 mm and smaller bars and taken as $(132 - d_b)/100$ for larger bars. The term $f_{td}'$ is the design tensile strength of concrete, defined as the characteristic tensile strength divided by 1.50.

The design anchorage length is determined as

$$L_{d,net} = \frac{\alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 L_d A_{s,cal}}{A_{s,eff}} \geq L_{d,min} \tag{13}$$

In the above, factors $\alpha_i$ adjust the length based on:

i = 1: Accounts for the effect of bar form, for straight bars = 1.0
i = 2: Accounts for the presence of welded transverse bars = 0.7
i = 3: Accounts for confinement provided by the concrete cover:

$$\text{For straight bars} = 0.7 \leq \left\{ \frac{1 - 0.15[c_d - d_b]}{d_b} \right\} \leq 1.0$$

i = 4: Accounts for confinement provided by traverse steel:

$$\left[1 = \frac{K\,(\Sigma A_{st} - 0.25\,A_s)}{A_s}\right]$$

where K = 0.1 for bars in stirrup corners and 0.05 for bars away from stirrup corner

i = 5: Effect of pressure perpendicular to the splitting plane

For high bond bars, the product $\alpha_3\,\alpha_4\,\alpha_5 = 0.7$ as a limit. For a typical 25 mm diameter bar in a 30 MPa concrete, analogous to a No. 8 U.S. bar in a 4000 psi concrete, the required development length would be about 780 mm or 31 inches.

An attractive feature of the CEB approach is that it supplies the designer with a simplified bond model that may be used in practical applications. The model is based on research results and uses a nonlinear law to relate the applied bond stress, $\tau$, to bar slippage, s. The model is illustrated in Fig. 2. The characteristic values $s_1$, $s_2$ and $s_3$ are based on the state of confinement of the concrete and the quality of bond that is thought to be present.

The four segments of the relationship are defined as follows:

$$\tau = \tau_{max}\left(\frac{s}{s_1}\right)^{0.4} \quad \text{for } 0 \le s \le s_1 \qquad (14)$$

$$\tau = \tau_{max} \quad \text{for } s_1 \le s \le s_2 \qquad (15)$$

$$\tau = \tau_{max} - \frac{(\tau_{max} - \tau_f)(s - s_2)}{(s_3 - s_2)} \quad \text{for } s_2 \le s \le s_3 \qquad (16)$$

$$\tau = \tau_f \quad \text{for } s_3 \le s \qquad (17)$$

Terms for these equations are defined in Table 1. The parameter $\tau_{max}$ represents the bond strength. Once the strength is attained, the bar slips at constant stress up to a value of slip, $s_2$, of 0.6 mm for unconfined concrete and 3.0 mm for confined concrete. Beyond that point the bond resistance decays with increasing bar slip and it reaches a residual value of $\tau_f$ at a slip value $s_3$ of approximately 2.5 mm.

## ANALYTICAL BOND MODELS

The heart of the issue in bond modeling is the bar slippage that occurs while the system is being loaded. If compatibility of strain between bar and concrete is maintained, then no relative movement occurs (i.e., no bar slip). This behavior is not found in actual members where there is always some bar slip as the reinforcement is loaded. Moreover, the assumption of perfect bond can be unconservative in analysis. Some shear failures are in fact bond failures that cause a reduction of shear capacity as a secondary effect (McCabe 1997). From a practical viewpoint, it is of interest to determine, for a given reinforcing arrangement, the amount of bond-slip likely to develop and its effect on performance. Note that if the slippage is too great then stiffness is reduced causing large member displacements. This behavior has been well-documented in experimental studies of monotonic and cyclically loaded structures and members. In the case of monotonic loading, local bond-slip relationships are difficult to measure experimentally. Results are generally reported as force at the loaded end vs. slip at either the loaded or unloaded end. Darwin et al. (1992) reported that the amount of actual slip measured under load was significant, even at low bar stress levels. Adhesion of the bar to concrete is seen to be destroyed nearly immediately. Following loss of adhesion, the amount of slip increases with increasing bar force in a nonlinear manner leading to complete loss of resistance to slippage at about 0.002 inches (0.05 mm) for a 25 mm diameter bar. The behavior noted is specimen and configuration dependent with changes in confinement, stirrups and cover, and bar size having large effect on the amount of slippage and the slope of the curve. Thus, the actual bar slip vs. bar stress relationship is complex and one that is quite difficult to characterize experimentally in a manner that can be readily used in design. Rather the information is of more interest to researchers.

A traditional approach taken in modeling the behavior is to use experimental data and represent the experimental trends by rules. These empirical approaches are very practical and may be the only workable model for large scale computations where complete structural systems must be modeled. However, more robust analytical models of the bar-concrete interface behavior have been proposed; these are derived from basic principles and to a large extent they avoid the limitations implicit in empirical models. Studies have included three primary approaches (ACI 408 1992): (1) solve for bond performance based on an assumed set of equilibrium and compatibility equations; (2) assume a function to model bond-stress distribution along the bar; and (3) use non-associated plasticity theory or fracture mechanics to model the interface between the steel and concrete where the bond develops.

Models that fall in categories (1) and (2) represent the background of the existing code methods and as such, have been implicitly described in the preceding sections. Alternative modeling approaches that attempt to mathematically define the mechanics of the bond are reviewed below:

<u>(a) Mechanics-Based Models for Bond</u>--Recently, a number of experimental and analytical studies have been presented, seeking a more fundamental definition of the local bond slip relationships (Gambarova et al. 1989, Giuriani et al. 1991, Cox and Herrmann 1992, Rosati and Schumm 1992, Rostasy et al. 1987). Giuriani et al. (1991) considered the development of an axially loaded bar, confined by transverse stirrups and anchored in concrete which has already developed splitting cracks. Provided that some reinforcement is crossing the splitting plane, bond action is still possible past the development of splitting. Based on the work of Gambarova et al. (1989), the local bond stress-slip relationship is expressed as a function of the crack opening, w: