Title No. 119-S132

Bond Strength and Development Length Models for Straight Plain Longitudinal Reinforcement in Tension

by Eyitayo A. Opabola

Previous experimental and analytical studies have shown that state-of-practice provisions for evaluating the bond strength and development length of plain bars are not appropriate. Existing provisions tend to provide overconservative estimates of the required development length of plain bars. Using a database of 518 development and 35 splice test specimens, this study proposes simple models for evaluating the bond strength and development length of plain round and square bars. The study demonstrates that the bond strength of a plain bar depends on the casting position, concrete strength, and the ratio of concrete cover to bar diameter. The study also shows that the influence of the loading rate and stirrup confinement level on the bond strength of plain bars may be insignificant. Furthermore, it is shown that the bar size factor in ACI 318 for deformed bars is not justified for plain bars. Using the proposed model, it is concluded that the required development length of a bottom-cast plain bar is 1.33 times that of a bottom-cast deformed bar. Also, the required development length of a top-cast plain bar is two times that of a top-cast deformed bar. The proposed model in this paper is recommended for incorporation into assessment standards.

Keywords: bond strength; development length; existing structures; plain bars; splice length.

INTRODUCTION

A significant number of existing multi-story reinforced concrete (RC) frame and wall structures in seismically active regions of the world were designed prior to the 1960s when the use of plain longitudinal reinforcement was allowed by building standards.[1,2] By the start of the 1960s, recognizing the better concrete-reinforcing bar bond-slip behavior of deformed bars, deformed bars became widely adopted as longitudinal reinforcement in concrete structures. Since then, most research studies have focused on the behavior of concrete structures with deformed bars.

In recent years, however, there has been renewed interest in improving the understanding of the behavior of concrete structures with plain longitudinal reinforcement. The renewed interest is a result of the increased societal demands on expected performance objectives of older-type structures under seismic demands and the socioeconomic implications of evaluating these structures using inadequate and conservative seismic assessment provisions. It is well-known that improved knowledge of structural behavior is critical to developing refined assessment provisions that may help avoid unwarranted intervention strategies in certain concrete structures with plain longitudinal reinforcement. With the aim of enhancing the understanding of the behavior of older-type structures, recent analytical and experimental studies have focused on assessing the behavior and performance of concrete structures with plain longitudinal reinforcement on the global level,[3] component level,[4,5] and material level[6,7] under various loading conditions.

Of interest to the current study are the existing assessment provisions for evaluating the development and splice lengths of straight plain bars. Two existing assessment provisions are considered in this study: ASCE/SEI 41-17[8] and ACI 562-19.[9] ASCE/SEI 41-17 provisions for the seismic assessment of existing buildings recommend that the development length of a plain bar should be taken as twice the development length of a deformed bar determined in accordance with ACI 318-19.[10] It is noteworthy that $l_{d,plain} \approx 2 \times l_{d,deformed}$ has been in place since ACI 318-51.[11] Experimental and analytical studies[12,13] have suggested that the rule may be too conservative for evaluating the lateral strength of RC columns with longitudinal plain bars.

ACI 562-19[9] makes provisions for the assessment, repair, and rehabilitation of existing concrete structures. ACI 562-19 recommends that a preliminary assessment of existing concrete structures should be carried out using the code provisions that were in effect when the structure was constructed. Aside from the expected high level of conservatism in these older design codes, as will be discussed later, ACI 562-19 recommendations can potentially result in a considerable nonuniformity in preliminary assessment results for similar structures built in different design eras. This is because the understanding and codification of the concrete-plain bar bond provisions evolved significantly between 1910 and 1963 (that is, from the 1910 Standard Building Regulations for the Use of Reinforced Concrete[14] to ACI 318-63[15]) (refer to Table 1).

Although experimental results as early as 1913[16] highlighted the significance of casting position effects on the concrete-reinforcing bar bond behavior, it was not until 1951 that the ACI Building Code started considering the influence of casting position on bond provisions. Prior to 1951, the bond stress in straight plain bars was limited to $0.04f_c'$, where f_c' is the concrete compressive strength (Table 1). Assuming a concrete compressive strength of 14 MPa (2000 psi) (the typical compressive strength of 1920s

ACI Structural Journal, V. 119, No. 6, November 2022.
MS No. S-2021-337.R1, doi: 10.14359/51736111, received March 24, 2022, and reviewed under Institute publication policies. Pertinent discussion including author's closure, if any, will be published ten months from this journal's date if the discussion is received within four months of the paper's print publication.

Table 1—Bond provisions for plain bars in United States from 1910 to 1963

Design code		Bond provisions for plain bars
1910 Standard Building Regulations for the Use of Reinforced Concrete[14]		0.55 MPa (80 psi) 1.03 MPa (150 psi) in cases where adequate mechanical anchorage is provided
1920 Standard Building Regulations for the Use of Reinforced Concrete[17]		$0.04f_c'$
ACI 501-36-T[18]		$0.04f_c'$ for beams, slabs, and one-way footings
ACI 318-41[19]		$0.04f_c' \leq 1.1$ MPa (160 psi) for beams, slabs, and one-way footings
ACI 318-47[20]		$0.04f_c' \leq 1.1$ MPa (160 psi) for beams, slabs, and one-way footings $0.06f_c' \leq 1.38$ MPa (200 psi) when end hooks are provided
ACI 318-51[11]		$0.03f_c' \leq 0.724$ MPa (105 psi) for top bars $0.045f_c' \leq 1.09$ MPa (158 psi) for bars other than top bars; plain bars must be hooked
ACI 318-63[15]	Working stress design	$u = 1.7\sqrt{f_c'}/D \leq 1.1$ MPa (160 psi) for top bars $u = 2.4\sqrt{f_c'}/D \leq 1.1$ MPa (160 psi) for bars other than top bars
	Ultimate strength design	$u = 3.35\sqrt{f_c'}/D \leq 1.72$ MPa (250 psi) for top bars. $u = 4.75\sqrt{f_c'}/D \leq 1.72$ MPa (250 psi) for bars other than top bars

concrete components[8]), the 1920 provisions are similar to the 1910 Standard Building Regulations[14] bond stress limit of 0.55 MPa.

The ACI 318-51 provisions reduced the bond stress limit for top-cast plain bars to $0.03f_c'$ from the bond stress of $0.04f_c'$ provided for all casting positions in pre-ACI 318-51 provisions. On the other hand, a bond stress limit of $0.045f_c'$ was provided for bottom-cast plain bars in ACI 318-51. Hence, according to ACI 318-51, the required development length of a top-cast plain bar is 1.5 times that of a bottom-cast plain bar. ACI 318-63, on the other hand, provided that the required development length of a top-cast plain bar is 1.4 times that of a bottom-cast plain bar. Also, the upper limit for the bond stress in top-cast bars was increased to 1.1 MPa (160 psi) from 0.724 MPa (105 psi) in ACI 318-51.

From ACI 562-19's point of view, based on the previously discussed variations in bond provisions in pre-1960s ACI Codes, there is bound to be a significant nonuniformity in preliminary assessment results for structures with similar reinforcement detailing but built in different design eras. Likewise, as previously mentioned, experimental studies have suggested that the ASCE/SEI 41 bond provisions may result in the overconservative estimation of the lateral strength of RC columns. To address all these, it is important to develop refined formulations for predicting the development and splice length of straight plain bars.

This paper proposes a formulation for predicting the development and splice length of plain bars. Using a collated database of 518 development test specimens subjected to monotonic loading, simple formulations are proposed for evaluating the bond strength and development length of plain bars. The adequacy of the proposed development length formulation for evaluating the splice length of plain bars was subsequently validated using a database of 35 splice test specimens with square and round plain longitudinal reinforcement.

RESEARCH SIGNIFICANCE

Practicing engineers are required to assess the performance of RC structures from various design eras and with various vulnerabilities using codified assessment provisions. To tackle the socioeconomic impact of inadequate and overconservative provisions in state-of-practice standards, it is important to develop refined assessment procedures. In this paper, a simple formulation has been developed for evaluating the bond strength and development length of plain bars. The proposed formulation is recommended for adoption by state-of-practice provisions.

Table 2—Range of properties for development test database (refer to Data set I in Appendix)

Parameter	Minimum	Maximum	Mean	Median
f_c', MPa (ksi)	9.7 (1.4)	61.6 (8.9)	30.6 (4.4)	27.4 (4.0)
d_b, mm (in.)	6.35 (0.25)	31.75 (1.25)	17.5 (0.7)	16 (0.63)
l_b/d_b	2.4	30	13.5	14.4
c_b/d_b	1.5	15.7	3.7	2.4

Note: f_c' is concrete compressive strength; d_b is bar diameter; l_b/d_b is ratio of provided development length to bar diameter; c_b/d_b is ratio of concrete cover to bar diameter.

EXPERIMENTAL DATABASE

Data set I

For this study, two groups of data sets were collated. The first data set consists of development tests on plain longitudinal bars with or without transverse reinforcement. Only monotonic tests were considered in Data set I. Data set I consists of 518 straight plain bar specimens collated from test programs published in the last 110 years. All the specimens in Data set I experienced a pullout failure. Due to the fact that beam-end tests provide more realistic measures of bond strength,[21] the initial database was segregated into pullout and beam-end test data subsets. However, a comparison of the test data from these subsets (that is, pullout and beam-end tests) did not suggest a significant influence of test configuration on the measured bond strength of plain bars. Hence, both data sets were combined for this study. As subsequently discussed in this paper, the transverse reinforcement confinement effect is also not significant on measured bond strength. The database is presented in the Appendix.*

The range of key parameters in Data set I is presented in Table 2. The majority of the test specimens in the database have compressive strength between 10 and 30 MPa (1.45 and 4.35 ksi). This range covers the lower-bound compressive strength in pre-1960s concrete components (refer to ASCE/SEI 41-17[8]). Information on measured steel properties was not typically reported in the older tests. In the modern tests where steel material properties were provided, the measured yield strength ranged from 247 to 380 MPa (36 to 55 ksi).

In terms of casting position, the proportion of bottom-cast, top-cast, and vertical-cast test specimens is 34%, 36%, and 30%, respectively. Eighty-two percent of all the test specimens had no transverse reinforcement, while the remaining 18% had transverse reinforcement. Information on transverse reinforcement spacing is provided in the database.

Data set II

Data set II consists of splice specimens. Only tests with straight splices were considered for this data set, and only 35 test data were sourced. It is noteworthy that all 35 test data have similar beam dimensions (305 x 410 mm [12 x 16 in.]) and shear span (1370 mm [54 in.]). The key variables in the data set are bar size (ranging from No. 6 to 10), bar shape (circular and square), provided splice length (ranging from $13d_b$ to $64d_b$), and casting position. The concrete compressive strength in Data set II ranged from 17 to 36 MPa (2.5 to 5.2 ksi). The yield strength of the longitudinal reinforcement ranged from 310 to 350 MPa (45 to 50.8 ksi). Key test data on specimens in Data set II are presented in the Appendix. It is noted that all the specimens in Data set II, except specimens 19-1010_C_B and 19-1210_C_B, experienced bond failure.

SUMMARY OF EXPERIMENTAL FINDINGS

The overall behavior and serviceability of RC structures rely on the stress transfer between the longitudinal reinforcement and surrounding concrete. This stress transfer depends on the anchorage and bond mechanism. The bond mechanism comprises three components: chemical adhesion, frictional resistance, and mechanical interlock.[21,22] Unlike deformed bars, the absence of ribs in plain bars means the mechanical interlock contribution to the concrete-plain reinforcing bar is negligible.[21,23] The frictional resistance is induced when the reinforcement slips relative to the surrounding concrete.[24]

Various experimental programs in the last century have explored the influence of the mechanical properties of the surrounding concrete, the volume of concrete around the bar (related to the bar-to-bar spacing and concrete cover), casting position, surface condition of the bar (corrosion level and coating), bar geometry (presence of ribs, rib geometry, and bar shape), and presence of confinement from transverse reinforcement on the bond resistance of plain bars (refer to the database in the Appendix). The results from these

*The Appendix is available at **www.concrete.org/publications** in PDF format, appended to the online version of the published paper. It is also available in hard copy from ACI headquarters for a fee equal to the cost of reproduction plus handling at the time of the request.

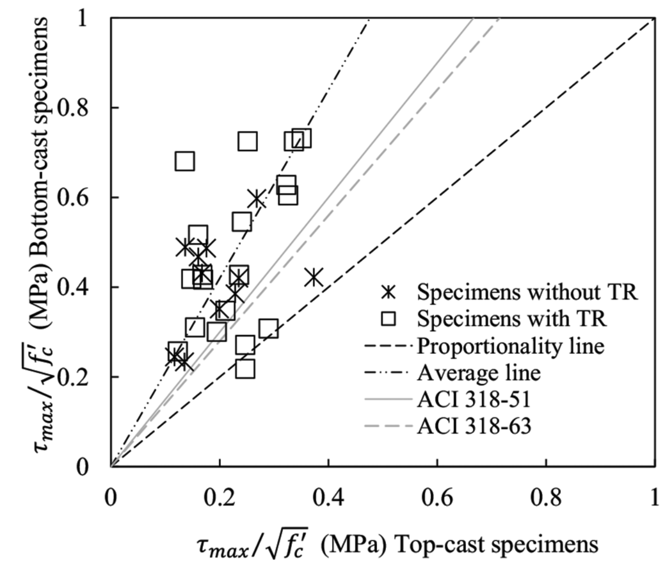


Fig. 1—Influence of casting position on bond strength of plain bars with and without transverse reinforcement (TR). (Note: ACI 318-19 provisions are intended for deformed bars only.)

experimental programs will be adopted to achieve the aims of this study. Prior to that, the subsequent subsections focus on discussing the influence of the aforementioned parameters on the concrete-plain bar bond resistance using experimental observations from Data set I.

Influence of casting position

As far back as 1913, studies have looked at the influence of casting position on the bond strength of plain bars. Abrams[16] observed that the bond strength of vertical-cast plain bar specimens could be up to 75% larger than the bond strength of horizontal-cast plain bar specimens cast from the same concrete batch. Abrams concluded that the difference is associated with the pullout direction relative to the direction of the concrete settlement.

Several studies[6,25-27] on plain bars have also concluded that the bond strength of horizontal-cast specimens decreases as the concrete depth below a bar increases. The influence of casting position on horizontal-cast specimens has generally been observed in experimental programs and recognized by ACI 318 since 1951.[11] The development length of top-cast plain bars was proposed to be 1.5 and 1.4 times the development length of bottom-cast plain bars in ACI 318-51 and ACI 318-63, respectively. It is noted that top-cast bars have always been defined in ACI 318 as horizontal-cast bars with more than 12 in. (300 mm) of concrete below the bar.

A subset of top-cast and bottom-cast plain bar specimens were selected from the collated database in this study. Only top-cast plain bar specimens with corresponding bottom-cast plain bar specimens molded from the same concrete batch were considered in this subset. A comparison of the normalized measured bond strength of the top-cast and bottom-cast specimens in the subset is presented in Fig. 1. As shown in Fig. 1, both ACI 318-51[11] (factor of 1.5) and ACI 318-63[15] (factor of 1.4) underestimate the magnitude by which the bond strength of bottom-cast specimens is larger

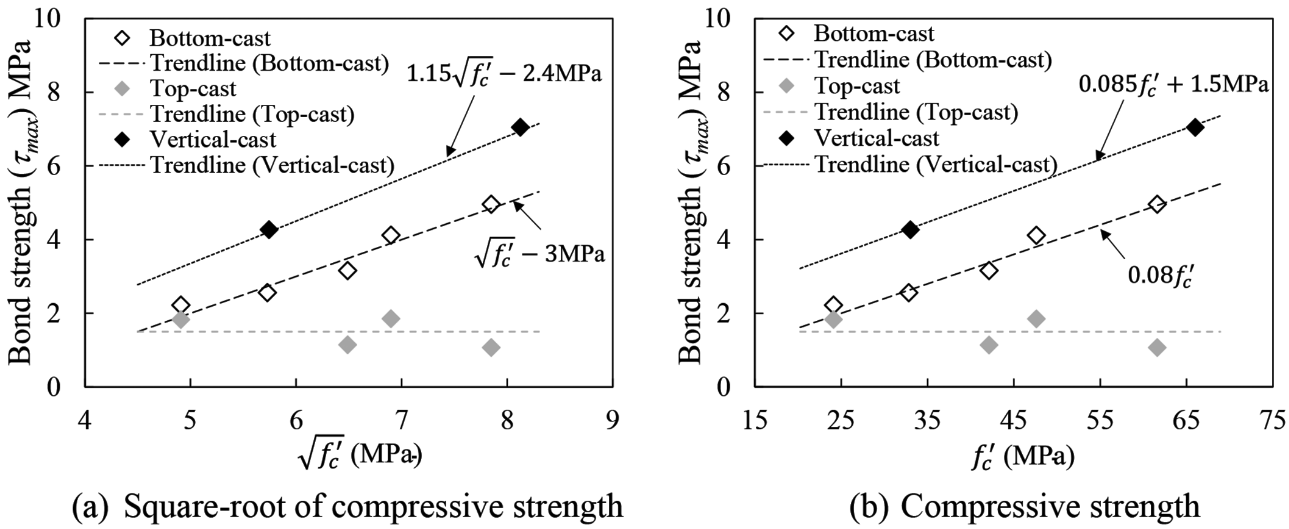


Fig. 2—Relationship between bond strength and concrete compressive strength adopting power coefficient (p) of: (a) p = 1/4; (b) p = 1/2; and (c) p = 1.

than the bond strength of corresponding top-cast specimens. Likewise, the ASCE/SEI 41-17 approach of adopting ACI 318-19's casting position coefficient of 1.3 for top-cast bars is also inadequate (note that the ACI 318-19 casting position coefficients were calibrated to tests on deformed bars). For the considered subset in this study, as shown in Fig. 1, the ratio of the bond strength of a bottom-cast specimen to the bond strength of a corresponding top-cast specimen has a mean of 2.1 and a coefficient of variation of 38%.

Influence of concrete strength

Concrete-reinforcing bar bond behavior is influenced by concrete properties—tensile strength, compressive strength, concrete mixture (that is, the mixture ratio of constituents, aggregate size, and aggregate type), fracture energy, and so on.[16,21,22,26,28] The effect of concrete properties on bond strength is typically represented using concrete compressive strength using a power function (that is, $\tau_{max} \propto f_c'^p$). Past studies[22,29,30] and design codes[11,15,17] have adopted the power coefficient p ranging from 1/4 to 1.0 for plain and deformed bars. As mentioned earlier, pre-1960s ACI design provisions adopted a power coefficient of 1.0, while ACI 318-63 provisions adopted a power coefficient of 1/2 for evaluating the bond strength of plain bars.

Figure 2 presents the relationship between bond strength and compressive strength with power coefficients of 1/4, 1/2, and 1 using only test data from experimental programs that have considered concrete strength as a test variable.[26,31] The bottom-cast and top-cast test bins are from Chana[26] with a c_b/d_b of approximately 2. On the other hand, the vertical-cast test data are from Mo and Chan[31] with a c_b/d_b of 6. It is noteworthy that other power coefficients were also considered aside from the power coefficients represented in Fig. 2. Based on the available data for plain bars, there is no justification to consider any power coefficient superior to the other. Additional test data may be needed to explore further the justification of a power coefficient for the relationship between concrete strength and bond strength of concrete-embedded plain bars. As adopted in past studies and current ACI 318 provisions, however, a power coefficient of 1/2 is adopted in this study to evaluate the bond strength of plain bars. It is noted that the discussions in this subsection are based on limited test data. Additional test data on specimens with concrete strength as a variable are needed to further validate the discussions presented in this subsection.

Figure 2 shows that the relationship between concrete strength and bond strength is dependent on the casting position. Figure 2 also suggests that concrete strength has a lesser significance on top-cast plain bars. However, there is not sufficient test data to conclude the influence of concrete strength on the bond strength of top-cast plain bars.

Influence of concrete cover

The failure mode of a concrete-reinforcing bar bond is typically related to the concrete cover.[21] It is well-known that there is a failure mode transition from a splitting failure mechanism to a pullout failure as concrete cover increases.[32] However, due to experimental evidence that bond failure of plain bars is typically through pullout failure, the influence of concrete cover on the bond strength of plain bars was typically considered insignificant.

In this section, the influence of concrete cover on the bond strength of plain bars is discussed using only test data from experimental programs that have considered concrete cover as a test variable.[6,16,27] Figure 3(a) looks at the influence of the ratio of concrete cover to bar diameter (c_b/d_b) on the bond strength (normalized by the square root of the compressive strength) for bottom-cast specimens with and without transverse reinforcement. The influence of the ratio of concrete cover to bar diameter (c_b/d_b) on the bond strength (normalized by the square root of the compressive strength) for top-cast specimens with and without transverse reinforcement is presented in Fig. 3(b). Lastly, Fig. 3(c) depicts the influence of the ratio of concrete cover to bar diameter (c_b/d_b) on the bond strength (normalized by the square root of the compressive strength) for vertical-cast specimens without transverse reinforcement based on test data from Abrams.[16] As shown in Fig. 3, irrespective of the casting

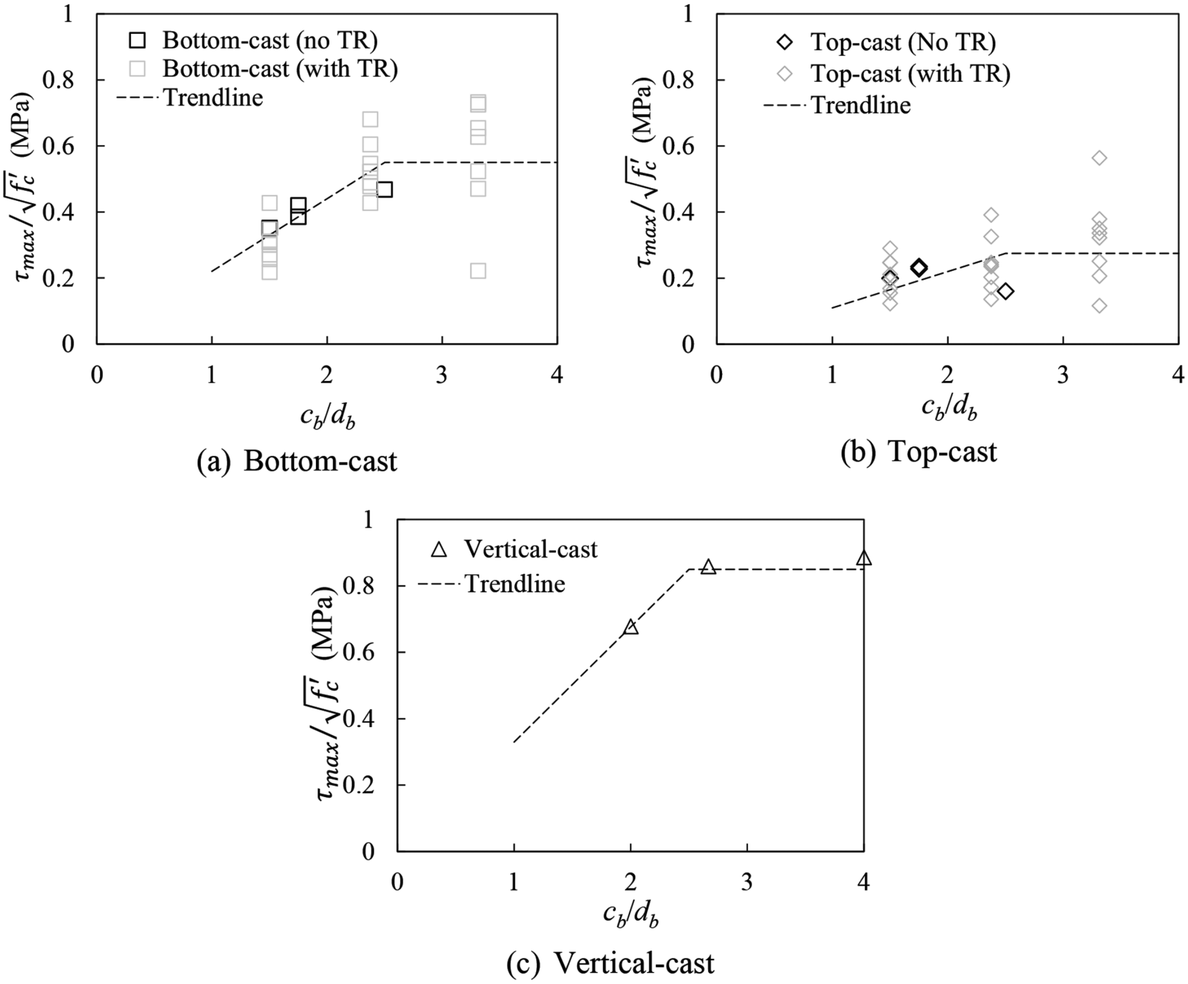


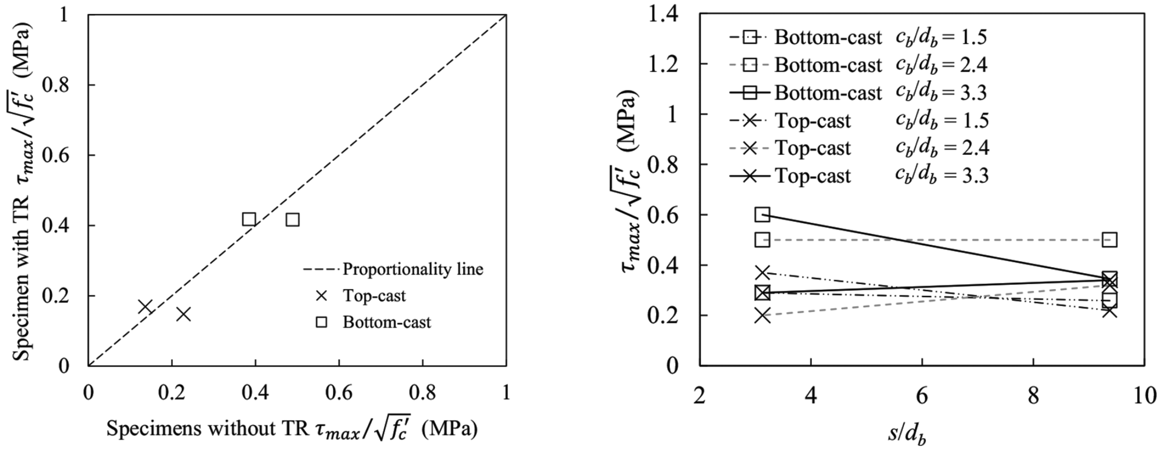
Fig. 3—Influence of concrete cover on bond strength of plain bars (TR is transverse reinforcement). (Note: Only test data from experimental programs that have considered concrete cover as test variable are considered herein.)

position, the bond strength of plain bars increases with an increase in provided concrete cover. However, for plain bars with a c_b/d_b greater than 2.5, the mean bond strength was not influenced by the c_b/d_b. Interestingly, a c_b/d_b of 2.5 is typically considered the transition point between splitting failure and pullout failure in deformed bars.[32] In comparison with the data presented in Fig. 3(a) for bottom-cast specimens, Fig. 3(b) shows that a larger scatter is observed in top-cast specimens. This scatter suggests that the bond behavior of top-cast specimens is susceptible to a high level of variability. A critical review of experimental results[26] showed that test-to-test variability in nominally identical bottom-cast specimens ranged from 9 to 17%, while test-to-test variability in nominally identical top-cast specimens ranged from 17 to 33% (that is, approximately twice that of the bottom-cast specimens), thereby suggesting that the scatter in Fig. 3 is representative. Additional test data are needed to draw conclusions on the scatter for vertical-cast specimens. It is noteworthy that pre-1963 ACI Codes do not consider the influence of the c_b/d_b on the bond strength of plain bars.

Influence of confinement from transverse reinforcement

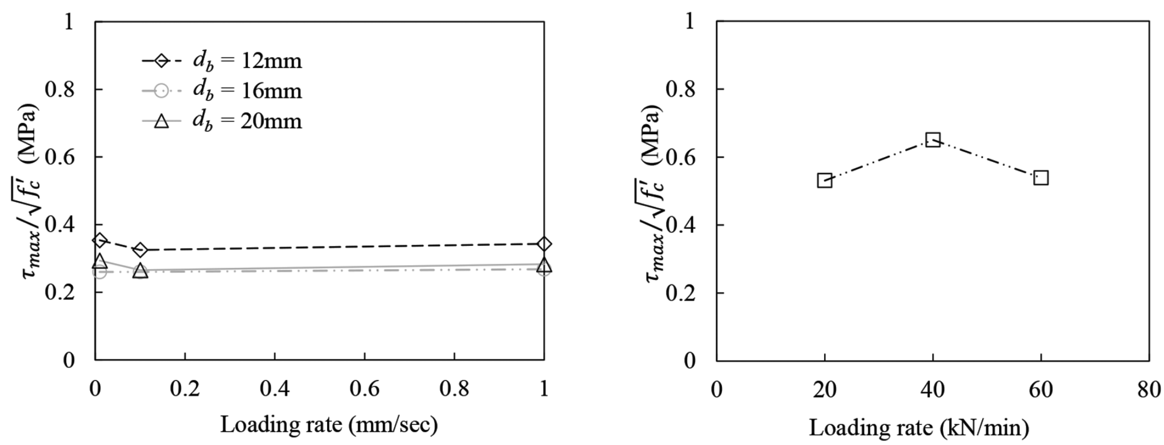
As discussed in past studies,[30,32] confinement from transverse reinforcement inhibits the progression of splitting cracks, and a sufficient amount of transverse reinforcement can cause a failure mode switch from splitting to a pullout failure mechanism. However, in plain bars with and without transverse reinforcement, pullout failure is typically observed,[26,27] thereby suggesting that the bond failure in plain bars does not rely on the presence and quantity of transverse reinforcement.

Figure 4(a) presents experimental results[25,26] on tests where the presence of transverse reinforcement was a variable. As shown in Fig. 4(a), irrespective of casting positions, the presence of transverse reinforcement did not result in a significant increase in bond strength. Figure 4(b) presents experimental results from Metzinger[27] where transverse reinforcement spacing was a variable. The results presented in Fig. 4(b) show that reducing the ratio of transverse reinforcement spacing to longitudinal bar diameter (s/d_b) from 9.4 to 3.1 did not result in a significant increase in bond strength. Based on these experimental results, it is concluded that the presence and spacing of transverse reinforcement do not significantly influence the bond strength of plain bars.



(a) (b)

Fig. 4—Influence of transverse reinforcement on bond strength of plain bars: (a) bond strength of specimens with transverse reinforcement versus bond strength of nominally identical specimens without transverse reinforcement; and (b) influence of ratio of transverse reinforcement spacing to longitudinal bar diameter (s/d_b) on bond strength.



(a) Tests by Li et al.[33] (b) Tests by Mo and Chan[31]

Fig. 5—Influence of loading rate on bond strength of plain bars.

It is noteworthy that pre-1963 ACI Codes do not consider the influence of transverse reinforcement on bond strength. Additional tests are, however, needed to draw solid conclusions on the influence of transverse reinforcement confinement on the bond strength of plain bars.

Influence of loading rate

In recognition of the fact that the loading rate influences concrete behavior under tension and compression, a few experimental tests[31,33] have been carried out to understand the influence of loading rate on the bond strength of plain bars. While it has been generally concluded that the loading rate is influential on the bond strength of deformed bars, experimental tests on plain bars show that the influence of the loading rate on the bond strength of plain bars may be negligible.

Figure 5 presents results from the experimental programs[31,33] that have looked at the influence of loading rate on the bond strength of plain bars. As shown in Fig. 5, for bar sizes ranging from 12 to 20 mm, the influence of the loading rate may be negligible. Based on the conclusions from these experimental programs, it can be assumed that there may be no need to account for the loading rate effects on the bond strength of plain bars. Additional tests are, however, needed to draw solid conclusions on the influence of loading rate on the bond strength of plain bars.

PROPOSED MODEL

Based on discussions presented in the previous section, it is concluded that the bond stress of plain bars is sensitive to concrete compressive strength, casting position, and the ratio of concrete cover to bar diameter (c_b/d_b). However, the bond stress of straight plain bars is not sensitive to the presence of transverse reinforcement and loading rate. Figure 6 presents measured data on the influence of the casting position and c_b/d_b on the ratio of measured bond stress to the square root of concrete compressive strength for all the specimens in the database.

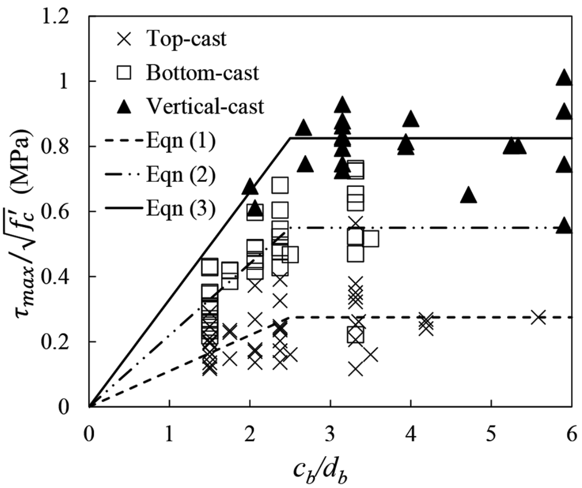


Fig. 6—Influence of casting position and c_b/d_b *on ratio of measured bond stress to square root of concrete compressive strength.*

As previously mentioned and shown in Fig. 6, experimental data suggest that the influence of concrete cover confinement becomes less significant as the c_b/d_b becomes greater than 2.5. There is, however, a large scatter in the top-cast data set (refer to Fig. 6). As mentioned earlier, this scatter is attributed to the significant test-to-test variability in top-cast specimens.

Neglecting the contribution of transverse reinforcement, the strength of a concrete-plain bar bond can be computed using Eq. (1) to (3).

For top-cast

$$\frac{\tau_{max}}{\sqrt{f'_c}} = 0.11\left(\frac{c_b}{d_b}\right) \text{ (in MPa)} \tag{1}$$

For bottom-cast

$$\frac{\tau_{max}}{\sqrt{f'_c}} = 0.22\left(\frac{c_b}{d_b}\right) \text{ (in MPa)} \tag{2}$$

For vertical-cast

$$\frac{\tau_{max}}{\sqrt{f'_c}} = 0.33\left(\frac{c_b}{d_b}\right) \text{ (in MPa)} \tag{3}$$

where c_b/d_b should not be taken greater than 2.5.

From Eq. (1) to (3), it is concluded that for the collated data set, the bond strength of a bottom-cast plain bar is twice that of a top-cast, and that of a vertical-cast is three times that of a top-cast. Combining Eq. (1) to (3), the bond strength of a concrete-embedded plain bar is computed as

$$\frac{\tau_{max}}{\sqrt{f'_c}} = \frac{0.22}{\psi_{cp}}\left(\frac{c_b}{d_b}\right) \text{ (in MPa)} \tag{4}$$

where ψ_{cp} equals 2.0 for top-cast, 1.0 for bottom-cast, and 0.67 for vertical-cast.

Given that ACI 318 provisions consider the influence of bar diameter on the bond strength of deformed bars (that is, bar diameter coefficient ψ_s), it was decided to explore the possibility of incorporating the influence of bar diameter into Eq. (4) for plain bars. Figure 7 looks at the influence of bar diameter on the model error of Eq. (4), defined as the ratio of measured bond strength to the computed bond strength using Eq. (4). As shown in Fig. 7, a strong justification cannot be provided for the incorporation of the bar diameter coefficient ψ_s for plain bars. It is also noted that ACI Committee 408[21] also concluded that the ACI 318 ψ_s factor of 0.8 for No. 6 (19 mm) and smaller deformed bars is not justified. Figure 7 shows that the ACI Committee 408[21] conclusion for deformed bars may be valid for plain bars as well.

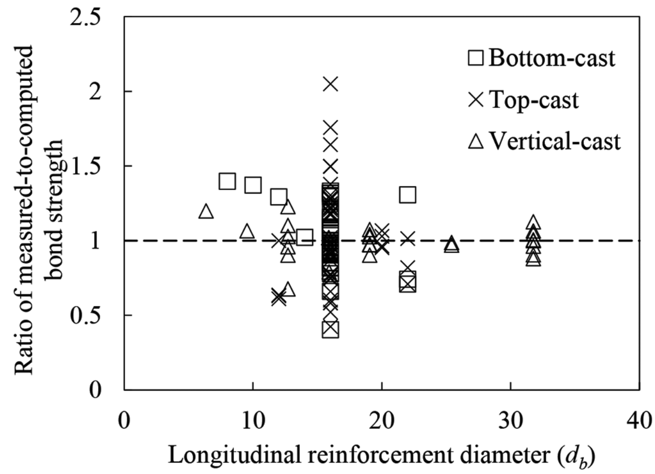


Fig. 7—Model error of Eq. (4) versus longitudinal reinforcement diameter.

Table 3—Statistics of ratio of measured-to-computed bond strength using Eq. (4)

	Mean	Median	Coefficient of variation, %
Bottom-cast	1.0	1.0	23
Top-cast	1.0	0.97	33
Vertical-cast	1.0	1.0	11
All specimens	1.0	1.0	25

Table 3 provides the statistics of the ratio of measured-to-computed bond strength of plain bars using Eq. (4). As shown in Table 3, the highest coefficient of variation is associated with the top-cast data set. For all the columns in Data set I, Eq. (4) provides a ratio of measured-to-computed estimate with a mean of 1.0 and a coefficient of variation of 25%.

A lognormal cumulative distribution function (CDF) is fitted to the distribution of measured-to-calculated bond strength values. The median estimates for the CDFs are presented in Table 3. Table 4 presents multipliers to Eq. (4) to achieve various probabilities of exceedance for different casting positions.

Assuming a uniform bond stress, Eq. (4) can be expressed as

$$l_{d,plain} = \frac{6}{5}\psi_{cp}\frac{f_y}{\sqrt{f'_c}\left(\frac{c_b}{d_b}\right)}d_b \text{ (in MPa)} \tag{5}$$

Table 4—Multipliers to Eq. (4) to achieve specific probabilities of exceedance

	Multiplier to achieve probability of exceedance		
	35%	20%	5%
Bottom-cast	0.9	0.8	0.67
Top-cast	0.86	0.74	0.57
Vertical-cast	0.95	0.9	0.83
All specimens	0.88	0.79	0.65

where ψ_{cp} equals 2.0 for top-cast, 1.0 for bottom-cast, and 0.67 for vertical-cast.

In imperial units, Eq. (5) can be rewritten as

$$l_{d,plain} = \frac{1}{10}\psi_{cp}\frac{f_y}{\sqrt{f_c'}\left(\frac{c_b}{d_b}\right)}d_b \text{ (in psi)} \tag{6}$$

COMPARISON WITH EXISTING MODELS

A number of formulations have been proposed to evaluate the bond strength of concrete-embedded plain bars under uniaxial tensile stresses. Other studies have also proposed formulations for predicting the bond strength of concrete-embedded plain bars under lateral tensile-compressive stresses.[34] This section focuses on the bond strength of concrete-embedded plain bars under uniaxial tensile stresses.

Feldman et al.[35] developed a formulation for predicting the bond strength of concrete-embedded plain bars, accounting for the influence of casting position, bar shape, and the ratio of concrete cover to bar diameter. In comparison with ACI 318 provisions for deformed bars, the Feldman et al.[35] formulation assumes that the bond strength is proportional to the square root of c_b/d_b.

$$\frac{\tau_{max}}{\sqrt{f_c'}} = 0.35k_1\sqrt{\frac{c_b}{d_b}} \text{ (in MPa)} \tag{7}$$

where k_1 is a top-cast ratio equal to 1.0 for all bottom-cast bars, 0.4 for round bars in the top-cast position, and 0.6 for square bars in the top-cast position.

Model Code 1990[36] proposed a bond strength of $0.3\sqrt{f_c'}$ for concrete-embedded plain bars with good bond conditions (that is, bottom-cast) and $0.15\sqrt{f_c'}$ for concrete-embedded plain bars with bad bond conditions (that is, top-cast).

ASCE/SEI 41-17[8] recommends that the development length of plain bars is twice that of deformed bars evaluated using the ACI 318 provisions. Assuming a uniform bond stress, this would correspond to a bond strength of

$$\frac{\tau_{max}}{\sqrt{f_c'}} = \frac{0.14}{\psi_t\psi_s}\left(\frac{c_b + k_{tr}}{d_b}\right) \text{ (in MPa)} \tag{8}$$

where ψ_s equals 1.0 for No. 7 (22 mm) bars and larger or 0.8 for No. 6 (19 mm) bars and smaller; and ψ_t equals 1.3 for top-cast and 1.0 for other casts.

According to the ultimate strength design (USD) approach of ACI 318-63,[15] the maximum bond strength would be

$$\frac{\tau_{max}}{\sqrt{f_c'}} = k_{63}\frac{7.07}{d_b} \leq \frac{1.72\,\text{MPa}}{\sqrt{f_c'}} \text{ (in MPa and mm)} \tag{9}$$

where k_{63} is equal to 1.0 for top-cast and 1.4 for other casts.

As mentioned earlier, a maximum bond strength of $0.04f_c' \leq 1.1$ MPa is provided by pre-1950s ACI Codes, while a value of $0.045f_c' \leq 1.09$ MPa and $0.03f_c' \leq 0.724$ MPa are provided for bottom-cast and top-cast plain bars, respectively, by ACI 318-51.[11]

The adequacy of all the formulations presented earlier is presented in Fig. 8 and Table 5. For the purpose of figure clarity, the formulations are binned into three groups. Figure 8(a) compares the proposed, Feldman et al.,[35] and ASCE/SEI 41-17[8] formulations; 8(b) compares the proposed, ACI 318-63,[15] and ACI 318-51[11] formulations; and 8(c) compares the proposed, Model Code 1990,[36] and pre-1950s ACI Code formulations.[14,17-20]

As shown in Table 5, the proposed model provides the best estimate of the bond strength of plain bars. On average, the Feldman et al.[35] model is 20% and 50% more conservative than the proposed model for top-cast and vertical-cast plain bars, respectively. The ASCE/SEI 41-17[8] approach provides a good mean measured-to-predicted ratio, but the model scatters are quite significant.

Comparing the pre-1970s ACI Building Codes, in terms of the mean measured-to-predicted ratio, the ACI 318-63[15] model performs better than its predecessors (that is, ACI 318-51,[11] ACI 318-47,[20] and so on). The higher level of conservatism in the pre-1960s Codes is associated with the inherent conservatism of the working stress design approach adopted by these Codes. The model scatters for all pre-1970s ACI Building Codes are relatively larger than those of the proposed model.

DEVELOPMENT LENGTH: PLAIN BARS VERSUS DEFORMED BARS

As earlier mentioned, ASCE/SEI 41-17[8] specifies that the development length of a plain bar is twice that of deformed bars evaluated using the ACI 318 provisions. This section compares the proposed development length of plain bars (Eq. (5)) to the ACI 318 development length for deformed bars (Eq. (10))

$$l_{d,deformed} = \frac{9}{10}\psi_t\psi_s\frac{f_y}{\sqrt{f_c'}\left(\frac{c_b + K_{tr}}{d_b}\right)}d_b \text{ (in MPa)} \tag{10}$$

where ψ_s equals 1.0 for No. 7 (22 mm) bars and larger, or 0.8 for No. 6 (19 mm) bars and smaller; and ψ_t equals 1.3 for top-cast and 1.0 for bottom-cast.

The comparison is carried out by computing the ratio of computed development length of plain bars using Eq. (5) and that computed using Eq. (10). Given that ACI 318 combines bottom-cast and top-cast (that is, refers to both cases as "others"), for comparison purposes, it was decided to adopt a ψ_{cp} value of 1.0 for both bottom-cast and vertical-cast when adopting Eq. (5). Given the previous conclusion that the stirrup confinement effect is negligible, K_{tr} is taken as zero.

Table 5—Comparison of existing models for predicting the bond strength of plain bars

	Top-cast		Bottom-cast		Vertical-cast		All specimens	
	Mean	CoV	Mean	CoV	Mean	CoV	Mean	CoV
Proposed (Eq. (4))	1.0	33	1.0	22	1.0	11	1.0	25
Feldman et al.[35]	1.2	35	0.9	27	1.5	12	1.2	32
ASCE/SEI 41-17[8]	0.8	39	1.0	43	1.3	46	1.3	43
Model Code 1990[36]	1.6	38	1.6	33	2.7	13	1.8	38
ACI 318-63[15] (USD)	0.7	40	1.3	33	2.1	30	1.3	54
ACI 318-51[11]	1.9	37	2.7	40	5.2	19	3.0	54
Pre-1950 ACI[14,17-20]	1.3	38	2.5	37	5.8	21	3.0	67

Note: CoV is coefficient of variation.

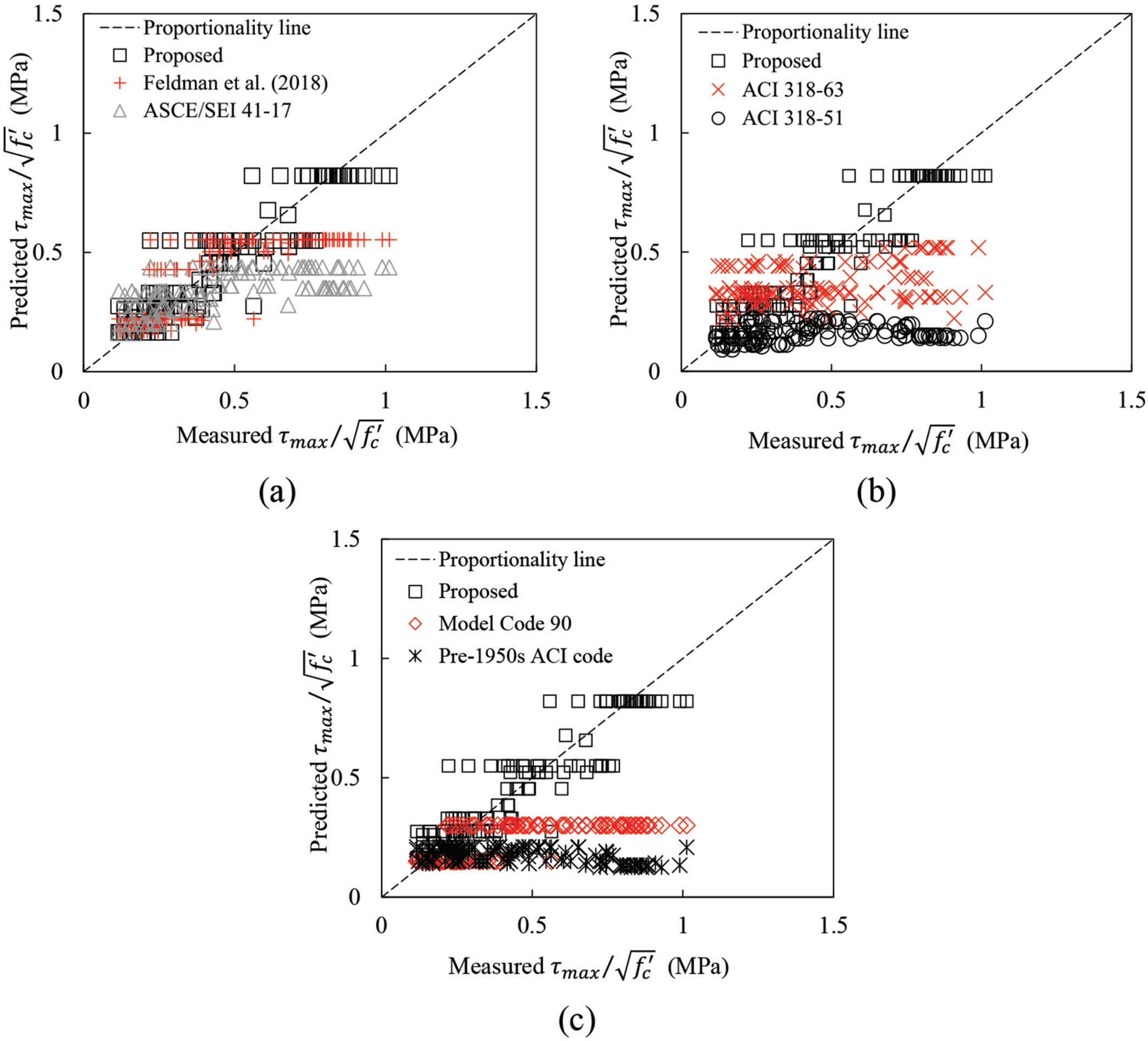


Fig. 8—Adequacy of considered models for predicting bond strength of plain bars.

Table 6 compares the computed development lengths for plain bars using Eq. (5) to the computed development lengths for deformed bars using Eq. (10). As shown in Table 6, the ratio of the computed development length for plain bars using Eq. (6) to the computed development length for deformed bars using Eq. (10) ($l_{d,plain}/l_{d,deformed}$) is dependent on the casting position and bar size. However, it is important to note the influence of bar size on the calculated $l_{d,plain}/l_{d,deformed}$ is attributed to the ψ_s factor in ACI 318 provisions. If the ACI 318 ψ_s factor is neglected in Eq. (10), as

Table 6—Ratio of computed development length for plain bars using Eq. (5) to computed development length for deformed bars using Eq. (10)

Bar size	$l_{d,plain}/l_{d,deformed}$	
	Top casting	Other casting
$d_b \geq$ No. 7	2.05	1.33
$d_b <$ No. 6	2.56	1.67

Table 7—Ratio of computed development length for plain bars using Eq. (5) to computed development length for deformed bars using Eq. (10) (neglecting ACI 318 ψ_s)

Bar size	$l_{d,plain}/l_{d,deformed}$	
	Top casting	Other casting
$d_b \geq$ No. 7	2.05	1.33
$d_b <$ No. 6	2.05	1.33

previously discussed in this paper, the development length of a top-cast plain bar is twice that of a top-cast deformed bar and 1.33 times that of other casts (Table 7).

Therefore, the current study recommends that the development length of a top-cast plain bar be taken as twice that of a top-cast deformed bar evaluated using the ACI 318 provisions, and that the development length of a plain bar with other casting positions be taken as 1.33 times that of a deformed bar with other casting positions evaluated using the ACI 318 provisions. For all casting positions, the ψ_s bar size factor and K_{tr} should be ignored for estimating the development length of plain bars.

SPLICE LENGTH OF PLAIN BARS

In older-type concrete components with splices within the region of maximum moment demand, it is important to evaluate the maximum developable tensile stress in the splices. It is well-known that force transfer between bars in a lap splice is through the surrounding concrete; hence, the bond strength of the lapped plain bars and the provided splice length influence the maximum developable tensile stress in the splices.

This section explores the adequacy of Eq. (5) in evaluating the maximum developable tensile stress in the splices of 35 beam specimens with spliced straight plain bars in Data set II.

For each specimen, the maximum developable tensile stress (f_s) is computed using Eq. (11). For the square bars, the bar diameter ($d_{b,s}$) is converted to an equivalent circular diameter $d_{b,s\text{-}c} = 1.13d_{b,s}$. The predicted tensile strength of the splice is evaluated from the predicted f_s.

$$f_s = \frac{l_b}{l_{d,plain}} f_y \leq f_y \tag{11}$$

where l_b is the provided splice length; and $l_{d,plain}$ is the required development length evaluated using Eq. (5).

A comparison of the measured peak tensile force and predicted tensile strength of the splices in the test specimens is presented in Fig. 9. As shown in Fig. 9, Eq. (11) provides a good estimate of the measured peak tensile force in the splices with a mean measured-to-predicted ratio of 1.1 and a coefficient of variation of 18%. It is also noteworthy that Eq. (11) provides a good estimate irrespective of bar shape, with a mean measured-to-predicted ratio of 0.9 and 1.1 for the bottom-cast and top-cast square bars; hence, it was concluded that, as adopted in ASCE/SEI 41-17,[8] no bar shape factor is necessary. It is also noteworthy that the experimental data do not suggest that the bar size influences the bond strength of spliced plain bars.

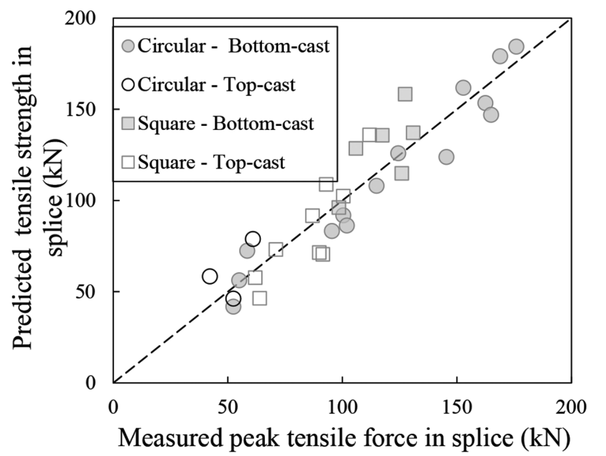


Fig. 9—Comparison of measured peak tensile force and predicted tensile strength in splices for Data set II.

Based on the databases adopted in this study, the model error of Eq. (5) for evaluating the development length (that is, a mean measured-to-computed ratio of 1.0 and a coefficient of variation of 25%) and splice length (that is, a mean measured-to-computed ratio of 1.0 and a coefficient of variation of 18%) are quite similar. Hence, it is concluded that, on average, developed plain bars and lap-spliced plain bars develop the same strength given the same length.

CONCLUSIONS

Using experimental data from 518 development tests and 35 splice tests on plain bars, this paper proposed models for predicting the bond strength and development length of plain round and square bars. As a first step, using the experimental results from the collated test data, it was demonstrated that the bond strength of plain bars is sensitive to concrete compressive strength, casting position, and concrete cover. However, available data suggest that the bond strength of straight plain bars is not sensitive to the presence of transverse reinforcement and loading rate.

Subsequently, simple models were proposed for evaluating the bond strength and development length of plain bars. The proposed models account for the influence of concrete compressive strength, casting position, and concrete cover. The study shows that the bar size factor in ACI 318 for deformed bars is not justified for plain bars. For the development tests, the proposed bond strength model provides a ratio of measured-to-computed estimate with a mean of 1.0 and a coefficient of variation of 25%. For the splice tests, the proposed development length model provides a ratio of measured-to-computed estimate with a mean of 1.0 and a coefficient of variation of 18%.

By comparing the proposed development length for plain bars to the ACI 318-19 development length for deformed bars, the current study recommends that the development length of a top-cast plain bar be taken as twice that of a top-cast deformed bar evaluated using the ACI 318 provisions. It also recommends that the development length of a plain bar with other casting positions be taken as 1.33 times that of a deformed bar with other casting positions evaluated using the ACI 318 provisions. For all casting positions, the ACI 318 bar size and confinement factors should be ignored for estimating the development length of plain bars.

It is noted that the proposed models have been developed based on limited test data. Additional tests are needed to further validate the developed models.

AUTHOR BIOS

ACI member **Eyitayo A. Opabola** *is a Senior Research Fellow in Resilience Engineering at University College London, London, UK. He received his BS and MS from the Voronezh State University of Architecture and Civil Engineering, Voronezh, Russia, and his PhD from the University of Auckland, Auckland, New Zealand. He is a member of ACI Subcommittees 369-A, General Provision, and 369-C, Frames. His research interests include seismic assessment and vulnerability modeling of concrete structures.*

REFERENCES

1. Fardis, M. N., *Seismic Design, Assessment and Retrofitting of Concrete Buildings: based on EN-Eurocode 8*, Springer, Dordrecht, the Netherlands, 2009, 744 pp.

2. Blaikie, E. L., and Spurr, D. D., "Earthquake Risk Associated with 1935-1975 Reinforced Concrete Buildings in New Zealand," Earthquake and War Damage Commission, Wellington, New Zealand, 1990, 142 pp.

3. Comert, M.; Demir, C.; Ates, A. O.; Orakcal, K.; and Ilki, A., "Seismic Performance of Three-Storey Full-Scale Sub-Standard Reinforced Concrete Buildings," *Bulletin of Earthquake Engineering*, V. 15, No. 8, Aug. 2017, pp. 3293-3320. doi: 10.1007/s10518-016-0023-4

4. Arani, K. K.; Di Ludovico, M.; Marefat, M. S.; Prota, A.; and Manfredi, G., "Lateral Response Evaluation of Old Type Reinforced Concrete Columns with Smooth Bars," *ACI Structural Journal*, V. 111, No. 4, July-Aug. 2014, pp. 827-838. doi: 10.14359/51686734

5. Bousias, S.; Spathis, A.-L.; and Fardis, M. N., "Seismic Retrofitting of Columns with Lap Spliced Smooth Bars Through FRP or Concrete Jackets," *Journal of Earthquake Engineering*, V. 11, No. 5, 2007, pp. 653-674. doi: 10.1080/13632460601125714

6. Cairns, J.; Du, Y.; and Law, D., "Residual Bond Strength of Corroded Plain Round Bars," *Magazine of Concrete Research*, V. 58, No. 4, May 2006, pp. 221-231. doi: 10.1680/macr.2006.58.4.221

7. Feldman, L. R., and Bartlett, F. M., "Bond Stresses Along Plain Steel Reinforcing Bars in Pullout Specimens," *ACI Structural Journal*, V. 104, No. 6, Nov.-Dec. 2007, pp. 685-692.

8. ASCE/SEI 41-17, "Seismic Evaluation and Retrofit of Existing Buildings," American Society of Civil Engineers, Reston, VA, 2017, 550 pp.

9. ACI Committee 562, "Code Requirements for Assessment, Repair, and Rehabilitation of Existing Concrete Structures (ACI 562-19) and Commentary," American Concrete Institute, Farmington Hills, MI, 2019, 94 pp.

10. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19)," American Concrete Institute, Farmington Hills, MI, 2019, 624 pp.

11. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-51)," American Concrete Institute, Farmington Hills, MI, 1951.

12. Goksu, C.; Yilmaz, H.; Chowdhury, S. R.; Orakcal, K.; and Ilki, A., "The Effect of Lap Splice Length on the Cyclic Lateral Load Behavior of RC Members with Low-Strength Concrete and Plain Bars," *Advances in Structural Engineering*, V. 17, No. 5, May 2014, pp. 639-658. doi: 10.1260/1369-4332.17.5.639

13. Opabola, E. A.; Elwood, K. J.; and Oliver, S., "Deformation Capacity of Reinforced Concrete Columns with Smooth Reinforcement," *Bulletin of Earthquake Engineering*, V. 17, No. 5, May 2019, pp. 2509-2532. doi: 10.1007/s10518-018-00540-w

14. National Association of Cement Users (NACU), "Standard Building Regulations for the Use of Reinforced Concrete (Standard No. 4)," American Concrete Institute, Farmington Hills, MI, 1910, 14 pp.

15. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-63) and Commentary," American Concrete Institute, Farmington Hills, MI, 1963, 144 pp.

16. Abrams, D. A., "Tests of Bond between Concrete and Steel," University of Illinois Bulletin, V. 11, Bulletin No. 71, University of Illinois Engineering Experiment Station, University of Illinois at Urbana-Champaign, Urbana, IL, Dec. 1913, 240 pp.

17. ACI Committee on Standard Building Regulations for the Use of Reinforced Concrete, "Standard Building Regulations for the Use of Reinforced Concrete (ACI Standard Specifications No. 23)," *ACI Journal Proceedings*, V. 16, No. 2, Feb. 1920, pp. 283-302.

18. ACI Committee 501, "Building Regulations for Reinforced Concrete (ACI 501-36-T)," American Concrete Institute, Farmington Hills, MI, 1936, 41 pp.

19. ACI Committee 318, "Building Regulations for Reinforced Concrete (ACI 318-41)," American Concrete Institute, Farmington Hills, MI, 1941, 67 pp.

20. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-47)," American Concrete Institute, Farmington Hills, MI, 1947, 64 pp.

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

22. *fib*, "Bond of Reinforcement in Concrete," *fib* Bulletin No. 10, International Federation for Structural Concrete, Lausanne, Switzerland, Aug. 2000, 434 pp.

23. Stookey, M. C., "Tests of Bond between Concrete and Steel," Bachelor's thesis, University of Illinois at Urbana-Champaign, Urbana, IL, 1907, 51 pp.

24. Stocker, M. F., and Sozen, M. A., "Investigation of Prestressed Reinforced Concrete for Highway Bridges: Part VI: Bond Characteristics of Prestressing Strand," Civil Engineering Studies, Structural Research Series No. 344, University of Illinois at Urbana-Champaign, Urbana, IL, June 1969, 403 pp.

25. Clark, A. P., "Bond of Concrete Reinforcing Bars," *Journal of Research of the National Bureau of Standards*, V. 43, No. 6, Dec. 1949, pp. 565-579. doi: 10.6028/jres.043.051

26. Chana, P. S., "A Test Method to Establish Realistic Bond Stresses," *Magazine of Concrete Research*, V. 42, No. 151, June 1990, pp. 83-90. doi: 10.1680/macr.1990.42.151.83

27. Metzinger, H., "Bond of Plain Round Bars," Edinburgh, UK, 2002.

28. Peattie, K. R., and Pope, J. A., "Effect of Age of Concrete on Bond Resistance," *ACI Journal Proceedings*, V. 52, No. 2, Feb. 1956, pp. 661-672.

29. Zsutty, T., "Empirical Study of Bar Development Behavior," *Journal of Structural Engineering*, ASCE, V. 111, No. 1, Jan. 1985, pp. 205-219. doi: 10.1061/(ASCE)0733-9445(1985)111:1(205)

30. Tepfers, R., "A Theory of Bond Applied to Overlapped Tensile Reinforcement Splices for Deformed Bars," Publication 73:2, Division of Concrete Structures, Chalmers University of Technology, Gothenburg, Sweden, 1973, 330 pp.

31. Mo, Y. L., and Chan, J., "Bond and Slip of Plain Rebars in Concrete," *Journal of Materials in Civil Engineering*, ASCE, V. 8, No. 4, Nov. 1996, pp. 208-211. doi: 10.1061/(ASCE)0899-1561(1996)8:4(208)

32. Orangun, C. O.; Jirsa, J. O.; and Breen, J. E., "A Reevaluation of Test Data on Development Length and Splices," *ACI Journal Proceedings*, V. 74, No. 3, Mar. 1977, pp. 114-122.

33. Li, X.; Wu, Z.; Zheng, J.; and Dong, W., "Effect of Loading Rate on the Bond Behavior of Plain Round Bars in Concrete under Lateral Pressure," *Construction and Building Materials*, V. 94, Sept. 2015, pp. 826-836. doi: 10.1016/j.conbuildmat.2015.07.085

34. Wu, Z.; Zhang, X.; Zheng, J.; Hu, Y.; and Li, Q., "Bond Behavior of Plain Round Bars Embedded in Concrete Subjected to Biaxial Lateral Tensile-Compressive Stresses," *Journal of Structural Engineering*, ASCE, V. 140, No. 4, Apr. 2014, p. 04013089. doi: 10.1061/(ASCE)ST.1943-541X.0000872

35. Feldman, L. R.; Poudyal, U.; and Cairns, J., "Proposed Development Length Equation for Plain Bars," *ACI Structural Journal*, V. 115, No. 6, Nov. 2018, pp. 1615-1623. doi: 10.14359/51702230

36. *fib*, "CEB-FIP Model Code 1990," International Federation for Structural Concrete, Lausanne, Switzerland, 1993, 460 pp.