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Bond Strength and Cyclic Bond Deterioration of Large-Diameter Bars

by Juan Murcia-Delso, Andreas Stavridis, and P. Benson Shing

A study on the bond strength and cyclic bond deterioration of large-diameter reinforcing bars embedded in well-confined concrete is presented. The study included monotonic pullout tests and cyclic pull-pull tests conducted on No. 11, No. 14, and No. 18 (36, 43, and 57 mm) reinforcing bars. The bond stress-slip relations obtained from the tests are presented, and the effects of the compressive strength of concrete, bar size, pull direction (for a vertically cast bar), and slip history on the bond strength are examined. Moreover, a phenomenological bond stress-slip law for monotonic and cyclic loading is proposed for bars embedded in well-confined concrete. This law has been validated with experimental results obtained in this study and in previous research.

Keywords: bond; bond-slip; cyclic; development length; large-diameter bars; pullout tests; reinforced concrete; reinforcing bars.

INTRODUCTION

The bond strength and bond-slip behavior between reinforcing steel and concrete have been extensively studied over the last few decades, and monographs have been prepared by *fib*¹ and ACI 408R-03² on this topic. Previous studies have shown that the bond strength depends on many factors, including the properties of the concrete and steel, the surface deformation of the bars, the loading history, the casting position of the bar, and the confinement level.^{1,2} Among these, a notable study was conducted by Eligehausen et al.³ to investigate the bond-slip behavior of bars subjected to monotonic and fully reversed cyclic loads. In their study, an extensive experimental program was carried out on No. 8 (25 mm) diameter bars embedded in concrete with a confinement level comparable to that found in typical beam-column joints of reinforced concrete (RC) frame structures. Based on this work, a number of analytical models have been proposed³⁻⁶ to predict the bond stress-slip relation under severe cyclic slip demands.

For large RC components, such as large bridge columns and piles, the use of reinforcing bars with diameters greater than No. 8 (25 mm) is common. However, data on the bond strength and bond stress-slip relation of large-diameter bars are scarce. The development length requirements in ACI 318-08⁷ and AASHTO LRFD Specifications⁸ are largely based on experimental data obtained from No. 11 (36 mm) and smaller bars and do not allow lap splicing of bars larger than No. 11 (36 mm) because of the lack of experimental data. Recently, some pullout and lap-splice tests were conducted on large-diameter bars.⁹⁻¹¹ Ichinose et al.⁹ carried out such tests on bars up to 2 in. (52 mm) in diameter and observed that the influence of bar size on the bond strength depends on the level of confinement. In their pullout and lap-splice tests conducted on specimens with no stirrups, bond failure was governed by concrete splitting and the bond strength decreased significantly with the increase of the bar size. Even though lap splices in specimens confined by stir-

rups also failed by the splitting of concrete, the effect of the bar size was not so significant. However, in the pullout tests conducted on specimens confined by stirrups, bond failure was caused by the localized crushing of concrete in front of the bar ribs and the effect of the bar size on the bond strength was not noticeable. Plizzari and Mettelli¹⁰ carried out pullout tests on 1.6 and 2 in. (40 and 50 mm) bars under low confinement conditions. Bond failure in all these tests was caused by the splitting of concrete and the resulting bond strengths were significantly lower than those obtained for smaller bars. Steuck et al.¹¹ carried out pullout tests on No. 10, No. 14, and No. 18 (32, 43, and 57 mm) bars embedded in well-confined high-strength grout. All specimens failed by the pullout of the bars and no significant variation in the bond strength was observed for the different bar sizes. All the tests on large-diameter bars mentioned in these studies were carried out under monotonically increasing slip. No information has been reported on the cyclic bond-slip behavior of large-diameter bars.

This paper presents an experimental and analytical study on the bond-slip behavior of large-diameter reinforcing bars embedded in well-confined concrete under monotonic and cyclic loads. The confinement level considered is representative of that for vertical reinforcing bars extending from a bridge column into a pile shaft, whose diameter is normally 2 ft (610 mm) larger than that of the column. Similar confinement conditions can be found for reinforcing bars anchored in other types of foundations. Monotonic pullout and cyclic pull-pull tests were conducted on No. 11, No. 14, and No. 18 (32, 43, and 57 mm) bars to obtain the bond strength and cyclic bond stress-slip relation. In addition to the influence of the bar diameter, the effect of the concrete strength, pull direction (for a vertically cast bar), and loading history on the bond strength have also been studied. Based on the obtained results and data from other studies, an analytical model is proposed herein to predict the bond stress-slip relation of bars embedded in well-confined concrete. This model is based on concepts originally proposed by Eligehausen et al.³ and further extended by others.⁴⁻⁶ However, it is distinct from those in that it requires the calibration of only three parameters and can be applied to bars of any size and concrete of different strengths.

RESEARCH SIGNIFICANCE

This study has generated much-needed experimental data on the bond-slip behavior of large-diameter bars embedded

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in well-confined concrete, providing information on the influence of the bar size, concrete strength, and slip history on the bond strength. A bond stress-slip law that can be easily calibrated based on the pertinent properties of the concrete and reinforcing bar has been developed to capture the cyclic bond-slip behavior of deformed bars of any size embedded in well-confined concrete. The model can be used in finite element analysis to understand the bond stress variation along the development length of a deformed bar.

PULLOUT TESTS ON LARGE-DIAMETER BARS

Four series of pullout tests were conducted on large-diameter reinforcing bars embedded in well-confined concrete.

Three were to study the bond-slip behavior of No. 11, No. 14, and No. 18 (32, 43, and 57 mm) bars under different loading histories, and the fourth was to study the influence of the compressive strength of concrete on the bond strength. A total of 22 specimens were tested; eight of these specimens were subjected to a monotonically increasing slip and 14 were subjected to cyclic loading. The specimen properties and test results are summarized in Table 1. These tests were conducted to identify the fundamental bond stress-versus-slip relation of a bar. In all the tests, bond failure was governed by the pullout of the bars from the concrete.

Specimen design, test setup, instrumentation, and loading protocols

A typical test specimen is shown in Fig. 1(a). Each specimen consisted of a reinforcing bar embedded in a 3 ft (914 mm) diameter concrete cylinder that had a height of 15 times the nominal bar diameter D_b . The bar was bonded only in the midheight region of the concrete cylinder over a length of $5D_b$, and polyvinyl chloride (PVC) tubes were used to create unbonded regions of $5D_b$ in length on each side of the bonded zone to minimize any local disturbance to the bond stress due to the experimental setup. This short embedment length was intended to provide a fairly uniform bond-stress distribution and prevent the yielding of the steel so that the fundamental bond stress-versus-slip relation could be obtained.

Grade 60 bars were used. The No. 11, No. 14, and No. 18 (32, 43, and 57 mm) bars had relative rib areas (the

Table 1—Test matrix and specimen properties

Series No.	Test No.	Bar size	Concrete compressive strength f'_c , ksi (MPa)	Tensile splitting strength f_{cs} , ksi (MPa)	Loading history	Peak bond strength τ_{max} , ksi (MPa)
1	1	No. 11 (36 mm)	4.9 to 5.3 (33.8 to 36.5)	0.45 (3.1)	Monotonic up	2.2 (15.2)
	2				Monotonic down	1.8 (12.4)
	3				Half cycles	2.0 (13.8)
	4				Half cycles	2.1 (14.5)
	5				Half cycles	1.7 (11.7)
	6				Full cycles	1.8 (12.4)
2	1	No. 14 (43 mm)	4.9 to 5.4 (33.8 to 37.2)	0.40 (2.8)	Monotonic up	2.8 (19.3)*
	2				Half cycles	2.2 (15.2)
	3				Full cycles	2.2 (15.2)
	4				Monotonic up	2.4 (16.5)
	5				Half cycles	2.2 (15.2)
	6				Double half cycles	2.2 (15.2)
3	1	No. 18 (57 mm)	5.0 to 5.9 (34.5 to 40.7)	0.45 to 0.50 (3.1 to 3.5)	Monotonic up	2.5 (17.2)
	2				Full cycles	1.9 (13.1)
	3				Full cycles	2.0 (13.8)
	4				Monotonic up	2.6 (17.9)
	5				Half cycles	2.1 (14.5)
	6				Half cycles	2.2 (15.2)
4	1	No. 14 (43 mm)	7.9 to 8.2 (54.5 to 56.5)	0.55 (3.8)	Monotonic up	3.5 (24.1)
	2				Monotonic up	3.3 (22.8)
	3				Double full cycles	2.8 (19.3)
	4				Full cycles	2.9 (20.0)

*Sealing in polyvinyl chloride (PVC) tube failed during construction, resulting in small amount of concrete accumulated at end of tube and, thereby, an increase of bonded length. Because actual embedment length is unknown, bond strength has been calculated with specified embedment length of $5D_b$.

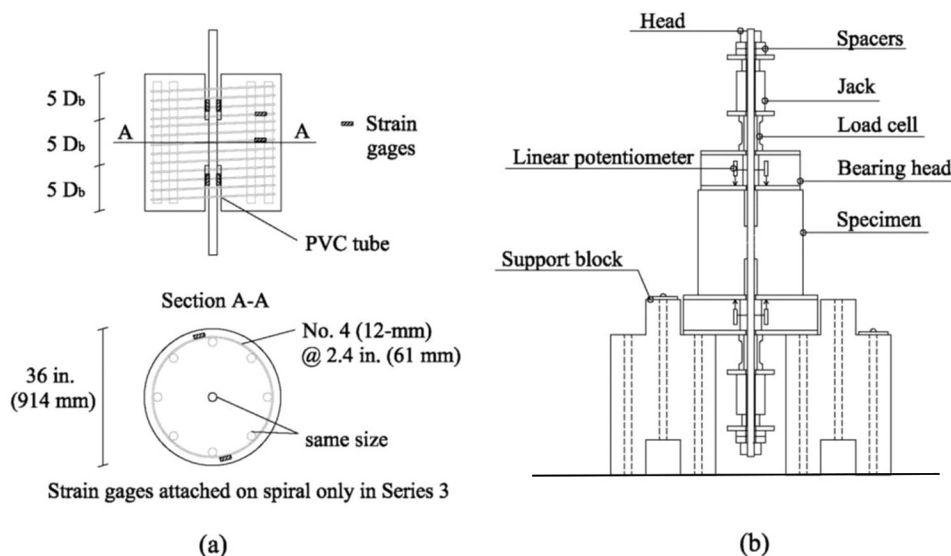


Fig. 1—(a) Typical test specimen and strain gauge locations; and (b) test setup and linear potentiometer locations.

ratio of the projected rib area normal to the bar axis to the bar surface area between the ribs) ranging from 0.068 to 0.095. The properties of the bars are summarized in Table 2. Each end of a bar had a T-head anchor, which provided a reaction for the application of the pulling force during a test.

The diameter of the cylinder and the quantity of the spiral reinforcement were selected to mimic the concrete cover and confinement level for the vertical reinforcing bars extending from a bridge column into a well-confined enlarged pile shaft designed according to the Bridge Design Specifications of the California Department of Transportation (Caltrans)¹² or AASHTO LRFD Specifications.⁸ The concrete was confined with No. 4 (13 mm) diameter spiral reinforcement with a pitch of 2.4 in. (61 mm) on center and an outer diameter of 32 in. (813 mm). This resulted in a confinement volumetric ratio of 1%.

Two concrete mixtures with different compressive strengths were used. The Series 1 through 3 tests had a concrete with a targeted compressive strength of 5 ksi (34.5 MPa), a maximum aggregate size of 3/8 in. (9.5 mm), a water-cement ratio (w/c) of 0.45, and a specified slump of 7 in. (178 mm). Series 4 had a concrete with a targeted compressive strength of 8 ksi (55 MPa), a maximum aggregate size of 0.5 in. (12.7 mm), a w/c of 0.32, and a specified slump of 8 in. (203 mm). The aggregate size and high slump used in these two mixtures represent what is typically used for cast-in drilled-hole (CIDH) foundation piles. All specimens in each series were cast in an upright position at the same time and were tested consecutively following the order shown in Table 1. The tests started on the day when the concrete strength was close to the targeted value. Setting up a test, testing, and dismantling took 1 to 2 days per specimen. The compressive and tensile splitting strengths of the concrete on the first and last days of testing for each test series are shown in Table 1.

The test setup is shown in Fig. 1(b). This setup was designed to allow the bar to be pulled upward or downward using center-hole hydraulic jacks that were positioned at each end of the bar. The bar was pulled out from the concrete cylinder when one of the hydraulic jacks pushed against the adjacent anchor head; the other jack was depressurized to allow the opposite end of the bar to move freely. To reverse

Table 2—Bar properties

Bar size No.	D_b , in. (mm)	Rib area ratio	Clear rib spacing, in. (mm)
11 (36 mm)	1.41 (36)	0.070	0.75 (19.1)
14 (43 mm)	1.69 (43)	0.068	0.98 (24.9)
18 (57 mm)	2.26 (57)	0.095	0.96 (24.4)

the pull direction, the jack initially pushing against the anchor head was depressurized before the other jack started to push against the anchor head at the opposite end. This was a self-reacting system; thus, the concrete was subjected to compression when the bar was being pulled out.

A load cell was placed between a hydraulic jack and the adjacent bearing head to measure the pullout force during the test. Two strain gauges were attached on two opposite sides of the bar right outside the bonded region at each end to measure the bar deformation, as shown in Fig. 1(a). In Series 3, four strain gauges were also attached on two opposite sides of the spiral at two elevations to monitor the spiral strain that could be introduced by the dilatation of the concrete during bar slip. Bar slip was measured with two linear potentiometers mounted at each end on the opposite sides of the bar, as shown in Fig. 1(b). Each pair of potentiometers measured the displacement of the attachment point on the bar with respect to the bearing head. A picture of one of the specimens and the test setup is shown in Fig. 2.

For all specimens that were tested with a monotonically increasing slip except one, the bar was pulled upward. Several load histories were used for the cyclic tests, with variables including the increment size of the slip amplitude in each loading cycle, the number of cycles per amplitude, and the type of cyclic reversals. Two types of cyclic reversals were considered: 1) full cycles with the same slip amplitudes in both directions for each cycle; and 2) half cycles with slips mainly in one direction and slightly passing the origin in the other. In most of the tests, only a single cycle was applied for each slip amplitude; however, in two tests, two cycles were applied for each amplitude. The type of loading protocol used for each specimen is given in Table 1.

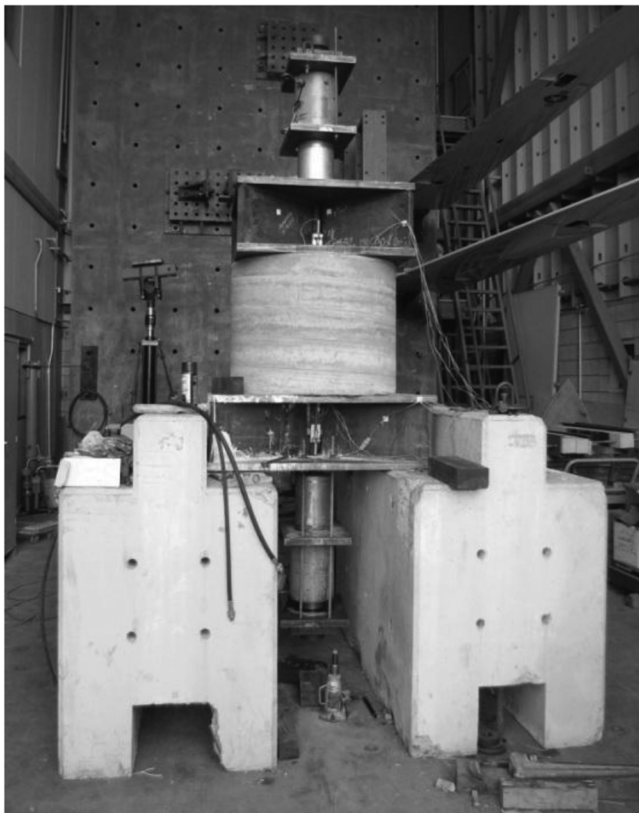


Fig. 2—Test setup.

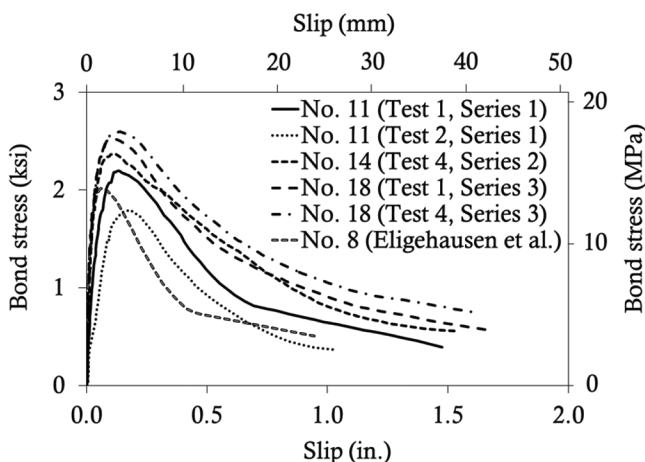


Fig. 3—Monotonic load tests.

Monotonic test results

The local bond stress (τ)-slip (s) relations were obtained as the average bond stress versus the average of the slips at the two ends of the bonded zone. The average bond stress was calculated by dividing the pullout force F by the nominal contact area between the bar and the concrete, as shown in the following equation

$$\tau = \frac{F}{\pi D_b L_e} \quad (1)$$

where L_e is the bonded length of the bar.

The slip at each end was calculated as the average of the slips measured by the pair of linear potentiometers. At the loaded end, the bar elongation between the attachment point of the linear potentiometers and the end of the bonded zone was subtracted from the potentiometer reading to get the actual slip. The bar elongation was calculated from strain gauge readings.

The bond stress-slip relations obtained from monotonic pullout tests in Series 1 to 3, which had concrete strengths of approximately 5 ksi (34.5 MPa), are plotted in Fig. 3. For comparison, the curve obtained by Eligehausen et al.³ for a No. 8 (25 mm) bar and 4.35 ksi (30 MPa) concrete is also included in Fig. 3. All the bond stress-slip curves show similar patterns. The slip at the peak strength was approximately 0.07 in. (1.8 mm) for the No. 8 (25 mm) bar and approximately 0.12 in. (3.0 mm) for the No. 11, No. 14, and No. 18 (32, 43, and 57 mm) bars. With increasing slip, the bond resistance dropped and tended to stabilize at a residual value that was approximately 20 to 30% of the peak resistance. Eligehausen et al.³ pointed out that a practically constant residual resistance was achieved when the value of the slip was approximately equal to the clear rib spacing of the bar, s_R . This can be explained by the total damage of the concrete between the ribs. Beyond this point, the resistance to slip was provided solely by friction. Figure 3 and the s_R values given in Table 2 confirm this observation for the large-diameter bars. However, the transition between the peak and the residual resistance seems to be more gradual for large-diameter bars as compared to the No. 8 (25 mm) bars tested by Eligehausen et al.³

The bond strengths τ_{max} obtained from the tests are summarized in Table 1. The results from Series 1 to 3 show that τ_{max} slightly increases with the bar diameter from 2.2 ksi (15.2 MPa) for No. 11 (36 mm) bars to an average of 2.55 ksi (17.6 MPa) for No. 18 (57 mm) bars. The tests conducted by Eligehausen et al.³ for No. 8 (25 mm) bars obtained an average bond strength of 2.0 ksi (13.8 MPa), but a direct comparison cannot be established because a concrete with a lower compressive strength and a lower degree of confinement was used by them. As Fig. 3 shows, the bond strength obtained for a No. 11 (36 mm) bar that was pulled downward (Test 2 from Series 1) was 20% lower than that for a bar pulled upward (Test 1 from Series 1). The initial stiffness was also reduced and the peak strength was reached at a slip of 0.18 in. (4.6 mm).

The results from Series 4 tests on No. 14 (43 mm) bars with 8 ksi (55 MPa) concrete showed a 45% increase of the average bond strength as compared to that obtained from Series 2, which had the same bar size but 5 ksi (34.5 MPa) concrete. Owing to the high bond strength developed in Series 4, the bars subjected to a monotonically increasing load (Tests 1 and 2) yielded at the pulled end. The bond stress-slip curves were not obtained for these two tests because the bond stress would not be uniform along the bonded length, as yielding might have penetrated into the bonded region. Studies^{13,14} have shown that bond resistance could be reduced in regions where a bar yielded. Hence, the bond strength for a bar without yielding would be higher than the average strength calculated from the results of this series of tests. However, this influence does not appear to be significant because the ratio of the average bond strength obtained from these two bars to that from the cyclic tests (Tests 3 and 4 of Series 4), in which the bars did not yield,

is comparable to the corresponding strength ratios obtained for the other series.

The strain gauges placed in the confining spirals registered small tensile strains. The values of the strains were so small (smaller than $50 \mu\epsilon$) that the resolution of the readings was only adequate to provide qualitative information. The tension in the spirals indicates that the concrete expanded slightly in the lateral direction as the bar was being pulled out. This can be explained in part by the dilatation caused by the wedging action of the bar ribs when the bar slipped. Another cause is the Poisson effect induced by the vertical compressive force exerted on the concrete cylinder as the bar was pulled.

Cyclic test results

The bond stress-slip relations for selected cyclic tests of Series 2 are compared to the monotonic test results in Fig. 4 and 5. The hysteretic response in all the tests shows a consistent behavior. Upon the reversal of the slip direction, a small resistance developed immediately in the other direction. This resistance started to increase when the slip approached the previously attained maximum slip. After this point, the resistance followed a curve similar in shape to the monotonic bond stress-slip curve due to bond deterioration induced by slip reversals. In addition, the absolute value of the slip at which the peak stress developed increased as the cumulative slip increased.

The maximum bond resistance obtained from a cyclic test is between 75 and 95% of that obtained from a monotonic load test, as shown in Table 1. The residual bond resistance diminishes to almost zero after severe cyclic slip reversals, as shown in Fig. 4 and 5. Moreover, the results presented in Fig. 4 indicate that full cycles induced a more severe deterioration of the bond resistance than half cycles given the same slip amplitude in one direction. Likewise, Fig. 5 shows that a second cycle between the same levels of slip produced an additional reduction of the bond stress. Overall, the observed hysteretic bond stress-slip relation for large-diameter bars is similar to that obtained by Eligehausen et al.³ for No. 8 (25 mm) bars.

FACTORS AFFECTING BOND STRENGTH

The tests presented herein provide useful information on the influence of the compressive strength of concrete, bar size, pull direction (for a vertically cast bar), and slip history on the bond strength. With additional data available in the literature, these effects are further analyzed and quantified in this section.

Effect of compressive strength of concrete

The tests presented previously show that the compressive strength of concrete f'_c has an important effect on the bond strength. A number of studies^{2,3} have suggested that the bond strength can be assumed to be proportional to $f'_c^{1/2}$, while others have found that it should be proportional to f'_c . However, the former relation has been adopted in ACI 318-08⁷ and AASHTO LRFD Specifications.⁸ The tests presented herein show that the bond strength was increased by approximately 45% when 8 ksi (55 MPa) concrete was used instead of 5 ksi (34.5 MPa) concrete. This implies that the bond strength is more or less proportional to $f'_c^{3/4}$. It is possible that this effect could be slightly underestimated

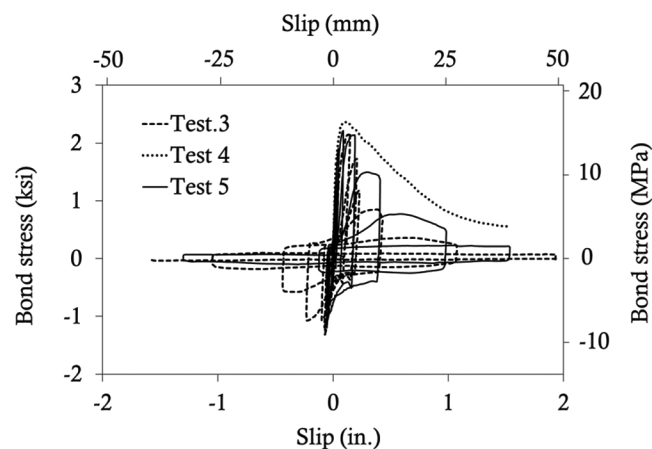


Fig. 4—Tests on No. 14 (43 mm) bars (Series 2) under monotonic (Test 4) and cyclic load with full cycles (Test 3) and half cycles (Test 5).

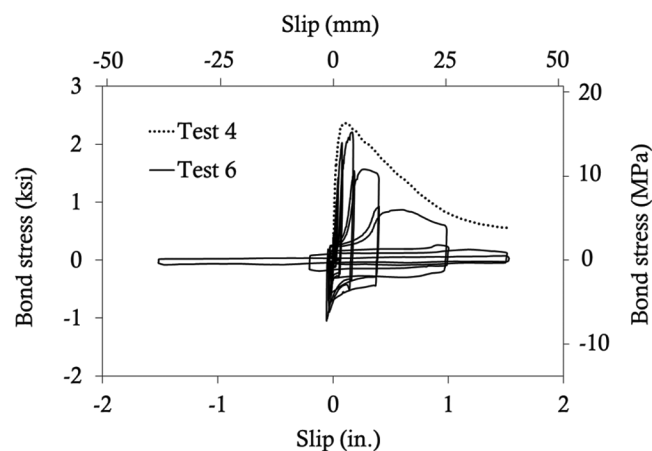


Fig. 5—Tests on No. 14 (43 mm) bars (Series 2) under monotonic (Test 4) and cyclic load with double cycles (Test 6).

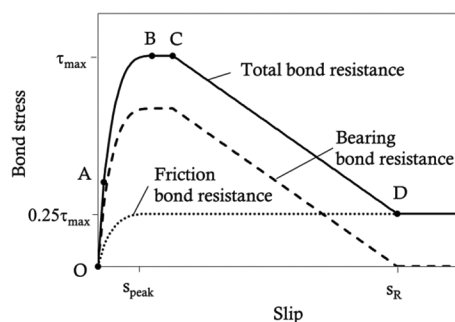
here because, as mentioned in a previous section, the bond strength calculated for the 8 ksi (55 MPa) concrete could be influenced by the yielding of the bars.

Based on a large number of lap-splice tests, Zuo and Darwin¹⁵ concluded that for splices not confined by transverse reinforcement, the average bond strength is proportional to $f'_c^{1/4}$ and the additional bond strength attributed to the presence of transverse reinforcement is proportional to $f'_c^{3/4}$. Hence, it appears that the relation between the compressive strength of concrete and the bond strength depends on the level of the confinement, which could explain the different conclusions obtained in different studies.

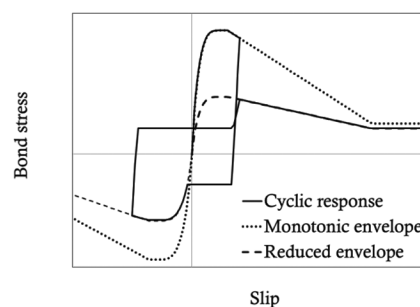
Effect of bar size

The test results show a slight increase of the bond strength with increasing bar size. The bond strength for No. 14 (43 mm) bars is approximately 7% higher than that for No. 11 (36 mm) bars, and that for No. 18 (57 mm) bars is approximately 8% higher than that for No. 14 (43 mm) bars. However, ACI 318-08⁷ and AASHTO LRFD⁸ provisions for the development length imply that the bond strength is reduced with increasing bar size.

Ichinose et al.⁹ noted that the influence of the bar size on the bond strength depends on the level of confinement. To inter-



(a) monotonic response



(b) cyclic response

Fig. 6—Analytical bond stress-slip model.

pret and compare results from different tests with different confinement levels, a factor used in ACI 318-08⁷ to calculate the required development length of deformed bars in tension is used as a confinement index. This index, which is denoted as CI in this paper, is expressed as $(c_b + 40A_{tr}/sn)/D_b$, in which c_b is the distance of the center of a bar to the nearest concrete surface, s is the spacing of the transverse reinforcement, A_{tr} is the transverse reinforcement area within distance s , and n is the number of bars being spliced or developed at the plane of splitting. According to ACI 318-08,⁷ when CI is less than 2.5, splitting failure is likely, and for values greater than 2.5, pullout failure is expected. Studies have shown that when the confinement level was low enough that bond failure was governed by concrete splitting, the bond strength would increase significantly with the decrease of the bar size.^{9,10} The value of CI considered in these studies ranges from 2 to 5. For pullout tests with CI between 5 and 16,^{9,11} splitting failure was prohibited and the effect of the bar size was negligible. The tests reported in this study had a CI between 11 and 17. A small increase in the bond strength with the bar size observed herein is consistent with the observation made by Ichinose et al.⁹

An explanation for the aforementioned observations is that larger bars have larger ribs, which induce a more severe dilatation effect and, thereby, a larger concrete splitting stress as a bar slips. With little or no confinement, this would result in an earlier splitting failure. With high confinement, not only splitting failure would be prohibited but also the dilatation effect induced by the wedging action of the ribs would induce a higher passive confinement pressure. An increase of the confining pressure would result in a higher bond stress based on results from other studies.¹⁶ Nevertheless, in the studies of Eligehausen et al.³ and Viwathanatepa et al.,¹³ even though the specimens were well-confined and had bond failure governed by the pullout of the bars, there was a slight increase of the bond strength for smaller bars. It should be noted that the specimens used by Eligehausen et al.³ and Viwathanatepa et al.¹³ had a CI between 3 and 13 and 9 and 14, respectively. They are, on average, lower than that considered in this study and the study of Steuck et al.,¹¹ which had a CI between 9 and 16.

Effect of pull direction

The influence of the pull direction on the bond strength was examined in the tests of No. 11 (36 mm) bars, which showed a lower bond strength and bond stiffness when a bar

was pulled downward instead of upward (refer to Table 1 and Fig. 3). This difference for specimens cast with the bar in a vertical position has also been observed in previous studies.^{1,17} It is well-known that a bar cast horizontally near the top surface of a concrete member will have a weaker bond strength than a bar that is near the bottom because voids are created underneath the top bar due to concrete settlement and bleed water.² The same happens to the concrete beneath the ribs of a vertically cast bar; consequently, the ribs push against a weaker concrete when the bar moves downward.¹

Effect of slip history

Similar to the observation made by Eligehausen et al.³ from cyclic bond-slip tests on No. 8 (25 mm) bars, the experimental results presented herein show that the peak bond strength was reduced when a prior load cycle went beyond 70 to 80% of the peak of the monotonic bond stress-slip curve, indicating the deterioration of the concrete surrounding the bar.

BOND STRESS-VERSUS-SLIP LAW FOR WELL-CONFINED CONCRETE

A phenomenological bond stress-slip law for bars embedded in well-confined concrete has been developed based on the experimental data presented in the previous section and on similar models proposed by others.³⁻⁶ In this model, the relation between the bond stress and slip for monotonic loading is described by a set of polynomial functions. For cyclic loading, a similar bond stress-slip relation is used but is reduced at each slip reversal using two damage parameters whose values are based on the slip history to account for cyclic bond deterioration. In addition, cyclic unloading and loading rules similar to those proposed by Eligehausen et al.³ are adopted to describe bond resistance right after slip reversal. This model is distinct from other models in that it has only three parameters to calibrate and can be applied to any bar size and concrete strength. The model is described in detail in the following.

Monotonic bond stress-slip relation

The monotonic bond stress (τ)-slip (s) relation assumed in this model is shown in Fig. 6(a). It is defined piecewise by five polynomial functions, as shown in Eq. (2), in terms of three governing parameters: the peak bond strength τ_{max} , the slip at which τ_{max} is attained (s_{peak}), and the clear spacing between the ribs s_R . These functions are given in the following.

Table 3—Model parameters

Test	τ_{max} , ksi (MPa)		s_{peak} , in. (mm)		s_R , in. (mm)
	From tests	Estimated	From tests	Estimated	
Series 1	2.20 (15.2)	2.40 (16.5)	0.12 (3.0)	0.10 (2.5)	0.75 (19.1)
Series 2	2.35 (16.2)	2.40 (16.5)	0.11 (2.8)	0.12 (3.0)	0.98 (24.9)
Series 3	2.55 (17.6)	2.40 (16.5)	0.12 (3.0)	0.16 (4.0)	0.96 (24.4)
Series 4	3.45 (23.8)	3.40 (23.4)	—*	0.12 (3.0)	0.98 (24.9)
No. 8 (25 mm) bar by Eligehausen et al. ³	2.00 (13.9)	2.15 (14.8)	0.07 (1.8)	0.07 (1.8)	0.40 (10.2)
No. 5 (16 mm) bar by Lundgren ¹⁷	2.90 (20.0)	2.50 (17.2)	0.04 (1.0)	0.04 (1.1)	0.30 (7.6) [†]

*Monotonic bond stress-slip curve not available.

[†]Value estimated.

$$\tau(s) = \begin{cases} 4 \frac{\tau_{max}}{s_{peak}} s & 0 \leq s < 0.1s_{peak} \\ \tau_{max} \left[1 - 0.6 \left(\frac{s - s_{peak}}{0.9s_{peak}} \right)^4 \right] & 0.1s_{peak} \leq s < s_{peak} \\ \tau_{max} & s_{peak} \leq s < 1.1s_{peak} \\ \tau_{max} \left[1 - 0.75 \frac{s - 1.1s_{peak}}{s_R - 1.1s_{peak}} \right] & 1.1s_{peak} \leq s < s_R \\ 0.25\tau_{max} & s \geq s_R \end{cases} \quad (2)$$

Until reaching 40% of the peak strength τ_{max} (Point A in Fig. 6(a)), the bond stress increases linearly with the slip. The nonlinear hardening behavior is represented by a fourth-order polynomial (A-B), followed by a plateau at τ_{max} (B-C). The bond strength decay is described by a linear descending branch (C-D). When the slip equals the clear rib spacing of the bar s_R (Point D), a residual bond stress equal to 25% of τ_{max} is assumed, and this value remains constant for larger slip values.

The use of the proposed law requires the determination of the three governing parameters. The value of s_R is a known geometric property of the bar, and it is usually between 40 to 60% of the bar diameter. As discussed in a previous section, τ_{max} depends on many factors and no theoretical formulas are available to accurately estimate its value. The same situation applies to s_{peak} . Therefore, these values have to be determined experimentally for each case if possible. For the No. 11, No. 14, and No. 18 (32, 43, and 57 mm) bars and concrete strengths considered in this study, experimental data are provided in Table 3.

When no experimental data are available, the following approximations based on data obtained in this study and by others can be used to determine τ_{max} and s_{peak} . The bond strength can be assumed to be 2.4 ksi (16.5 MPa) for 5 ksi (34.5 MPa) concrete, regardless of the bar size. This is based on the average τ_{max} value obtained from Test Series 1 to 3 shown in Table 3. The slight increase of the bond strength with the increase of the bar size observed herein can be ignored in view of the lack of a comprehensive study with a broad range of bar sizes to examine this influence. For concrete strengths other than 5 ksi (34.5 MPa), τ_{max} can be scaled accordingly with the assumption that it is proportional to $f_c'^{3/4}$. As shown in Table 3, the slip s_{peak}

at which the maximum bond resistance develops for the No. 11, No. 14, and No. 18 (32, 43, and 57 mm) bars is approximately 1.7 times that for No. 8 (25 mm) bars³ and three times that for No. 5 (16 mm) bars.¹⁷ This seems to indicate a scale effect with respect to the bar size, but these values could also be influenced by other factors, such as the confinement, concrete properties, and loading conditions. In addition, some studies^{3,4} have indicated that s_{peak} also depends on the relative rib area. Owing to the lack of more conclusive data, it is recommended that s_{peak} be taken to be 7% of the bar diameter, which is the average of the experimentally obtained s_{peak} values, as presented in Table 3, normalized by the respective bar diameters.

The experimental results presented herein and those obtained by Lundgren¹⁷ have shown that the bond strength and the bond stiffness are reduced when a vertically cast bar is pulled downward. Based on this data, the monotonic bond stress-slip relation for a vertically cast bar pulled downward is described by Eq. (3). Note that for a bar pulled downward, the slip and bond stress have a negative sign here.

$$\tau(s) = \begin{cases} -2.3 \frac{\tau_{max}}{s_{peak}} |s| & -0.15s_{peak} \leq s < 0 \\ -\tau_{max} \left[0.85 - 0.505 \left(\frac{|s| - 1.5s_{peak}}{1.35s_{peak}} \right)^4 \right] & -1.5s_{peak} \leq s < -0.15s_{peak} \\ -0.85\tau_{max} & -1.6s_{peak} \leq s < -1.5s_{peak} \\ -\tau_{max} \left[0.85 - 0.60 \frac{|s| - 1.6s_{peak}}{s_R - 1.6s_{peak}} \right] & -s_R \leq s < -1.6s_{peak} \\ -0.25\tau_{max} & s < -s_R \end{cases} \quad (3)$$

Furthermore, previous experimental studies^{13,14} have shown that the bond resistance can be considerably reduced after a reinforcing bar yields in tension. This observation could not be verified in this study, although the bars yielded in two of the specimens. The reason is that in these two cases, the concrete had a compressive strength of 8 ksi (55 MPa), and there were no other specimens tested monotonically that had the same concrete strength but no bar yielding. In addition, in these specimens, yielding occurred at one end of the bar, while the other remained unstrained. As a result, yielding might have occurred only in the upper portion of the bonded region and the total bond force was probably only slightly affected by this. Several empirical laws have been proposed to account for the yielding effect by modifying the bond stress-slip law with a reduction factor that depends on

the longitudinal strain and the yield strain of the reinforcing bar.^{6,18,19} These laws can be used to modify the bond stress-slip law presented herein to account for the tensile yielding of a bar.

Cyclic law

The extension of the bond stress-slip law to cyclic loading is based on the experimental evidence presented in this study and the bond-slip mechanism hypothesized by Eligehausen et al.³ It is assumed that at a large slip, part of the concrete in contact with the ribs on the bearing side is crushed and a gap has been created on the other side of the ribs. This gap needs to close before the bearing resistance in the opposite direction can be activated. Hence, the initial bond resistance developed upon slip reversal after a large slip can be attributed solely to friction. Once contact is resumed on the bearing side of the rib, the bond resistance increases. However, this resistance is lower than that under a monotonic load for the same level of slip due to the deterioration of the concrete around the ribs.

In most phenomenological models, bond deterioration under cyclic slip reversals is simulated by scaling the monotonic bond stress-slip relation, and the scale factors are updated upon each slip reversal. Some of these models adopt a single damage parameter that is a function of the energy dissipated by bond slip³ or of the slip history⁶ to estimate a scale factor. Some models^{4,5} distinguish the bearing and friction resistances, which are scaled independently as a function of the maximum slip and the number of load cycles. The latter approach has been adopted herein based on the experimental evidence that the reduction of the peak strength is, in general, more rapid than that of the residual strength. The peak strength in a monotonic bond stress-slip curve is mainly contributed by the bearing resistance, while the residual strength is entirely due to friction. Friction deterioration is caused by the smoothening of the interface between the steel and concrete and, therefore, can be assumed to be dependent on the total cumulative slip. The deterioration of bearing resistance is caused by the crushing and/or shearing of the concrete between the ribs. Therefore, it can be assumed to be dependent only on the maximum slips attained in the two loading directions. For sliding between previously attained levels of slip, there will be no bearing contact between the concrete and the ribs and, therefore, no further crushing and shearing of concrete could occur. These mechanisms are consistent with the cyclic behavior observed herein. Fully reversed cycles are more damaging than half cycles because the maximum slip excursion and the total slip accumulated are larger, causing more deterioration in both the bearing and the friction resistances. Double cycles between the same slip levels induce slightly more damage than single cycles because the second cycle causes a further reduction of the friction resistance.

Based on the aforementioned reasoning, the monotonic bond stress-slip relation in this model is separated into a bearing component and a friction component, as shown in Fig. 6(a). From the origin to the end of the plateau at the peak of the curve (Point C), the bearing resistance τ_b is assumed to be 75% of the total bond resistance, and the remaining 25% is assumed to be contributed by the friction resistance τ_f . After the peak, τ_b is assumed to decay linearly to zero when the slip is equal to s_R —that is, when the concrete between the ribs has been completely crushed or sheared off. The friction resistance τ_f is assumed to remain constant as slip continues

to increase after the peak. To model the cyclic bond deterioration, the following damage law is used

$$\begin{aligned}\tau_{red} &= \tau_{b,red} + \tau_{f,red} \\ \tau_{b,red} &= (1 - d_b)\tau_b \\ \tau_{f,red} &= (1 - d_f)\tau_f\end{aligned}\quad (4)$$

in which τ_{red} is the reduced bond resistance; $\tau_{b,red}$ is the reduced bearing resistance; $\tau_{f,red}$ is the reduced friction resistance; d_b is the damage parameter for the bearing resistance; and d_f is the damage parameter for the friction resistance. The bond stress-slip relation is updated using Eq. (4) when the load is reversed. The damage laws have been calibrated using the experimental data from Test Series 1, 2, and 3. Data from Series 4 were not used because of the lack of a reliable experimental monotonic bond stress-slip relation to allow a direct comparison with the cyclic behavior.

The damage parameter affecting the bearing resistance, d_b , is defined as a function of the maximum slip

$$d_b(\bar{s}_{max}) = 1 - 1.2e^{-2.7\left(\frac{\bar{s}_{max}}{s_R}\right)^{0.8}} \geq 0 \quad (5a)$$

where

$$\bar{s}_{max} = 0.75\max(s_{max}^+, s_{max}^-) + 0.25(s_{max}^+ + s_{max}^-) \quad (5b)$$

in which s_{max}^+ and s_{max}^- are the maximum slips reached in the positive and negative directions in absolute value. Because full cycles produce more damage than half cycles, the maximum slip considered herein is a weighted average of the absolute maximum slip reached in any of the two directions and the sum of the maximum slips in the two directions. As mentioned previously, cyclic deterioration starts to become apparent after the maximum bond stress in a previous cycle has reached 70 to 80% of the peak bond strength developed under a monotonic load. This is represented in the aforementioned damage index.

The friction resistance decreases progressively as a result of the smoothening of the bond interface, which depends on the total cumulative slip. However, more severe deterioration has been observed in the residual bond strength as the maximum slip increases in a subsequent cycle. Therefore, the damage parameter for the friction resistance, d_f , is assumed to be a function of both the absolute maximum slip attained in each loading direction and the cumulative slip s_{acc} .

$$d_f(s_{acc}, s_{max}^+, s_{max}^-) = \frac{\min(s_{max}^+ + s_{max}^-, s_R)}{s_R} \left(1 - e^{-0.45\left(\frac{s_{acc}}{s_R}\right)^{0.75}} \right) \quad (6)$$

To avoid an overestimation of damage that could otherwise be caused by a large number of small cycles, s_{acc} is considered to be zero before the slip displacement has exceeded s_{peak} for the first time. This is a reasonable assumption if one agrees that friction should play a minor role at the beginning when bearing is significant.

Right after each slip reversal, unloading and reloading in the other stress direction follows the initial stiffness of the monotonic curve until the friction resistance in the oppo-

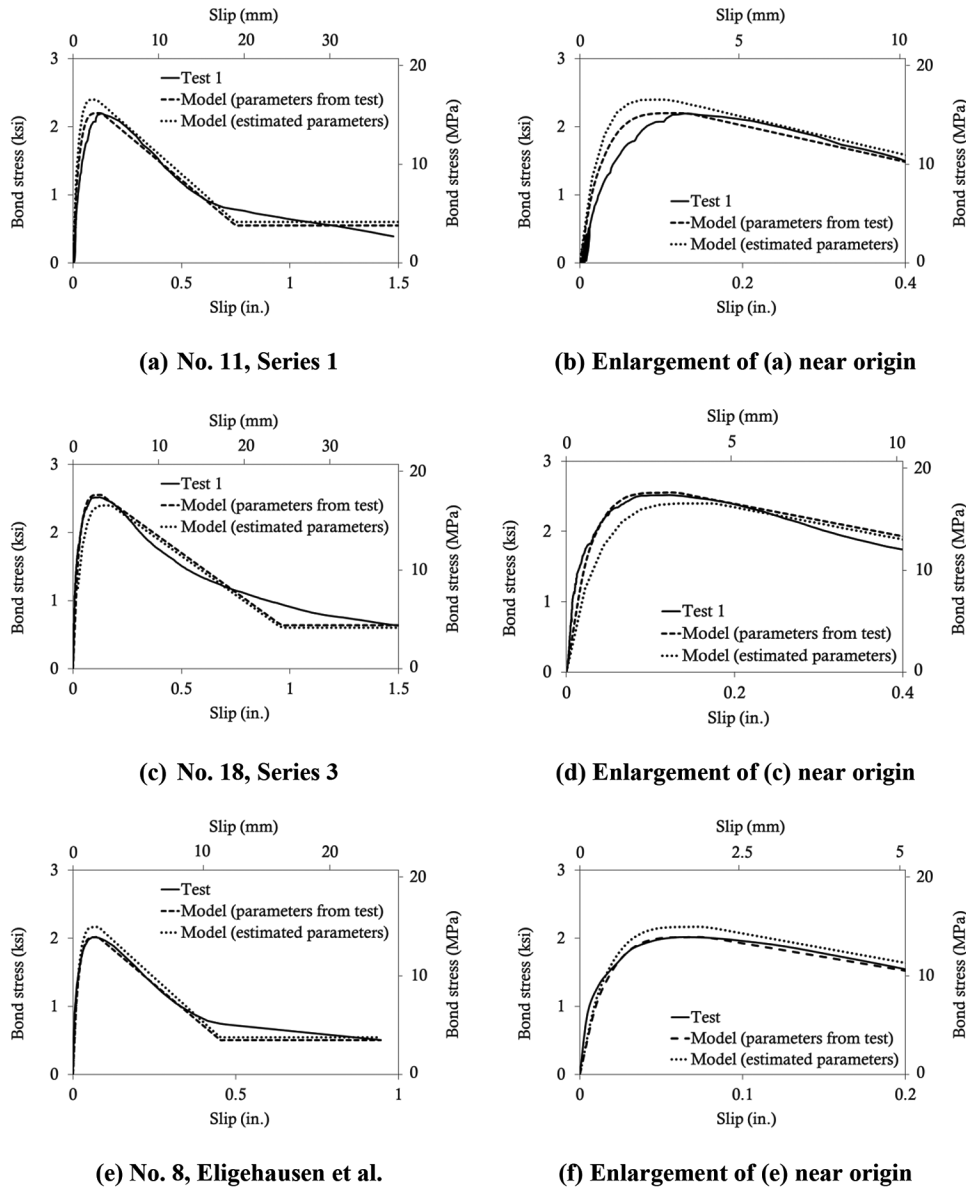


Fig. 7—Analytical and experimental results for monotonic loading.

site direction is reached. If the maximum slip ever achieved exceeds the slip at the peak resistance s_{peak} , the resistance τ_{rev} right after slip reversal is equal to the reduced friction $\tau_{f,red}$ given in Eq. (4). Otherwise, it is a fraction of the reduced friction, as shown in Eq. (7), which is a modification of what was suggested by Eligehausen et al.³

$$\tau_{rev} = k_{rev} \tau_{f,red} \quad (7a)$$

where

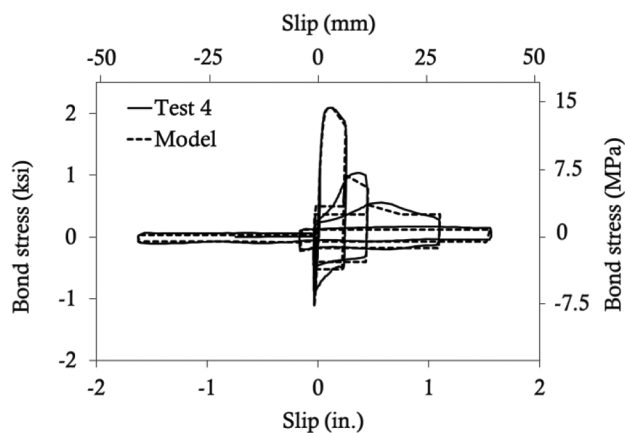
$$k_{rev} = \frac{\max(s_{max}^+, s_{max}^-)}{s_{peak}} \leq 1 \quad (7b)$$

COMPARISON OF ANALYTICAL AND EXPERIMENTAL RESULTS

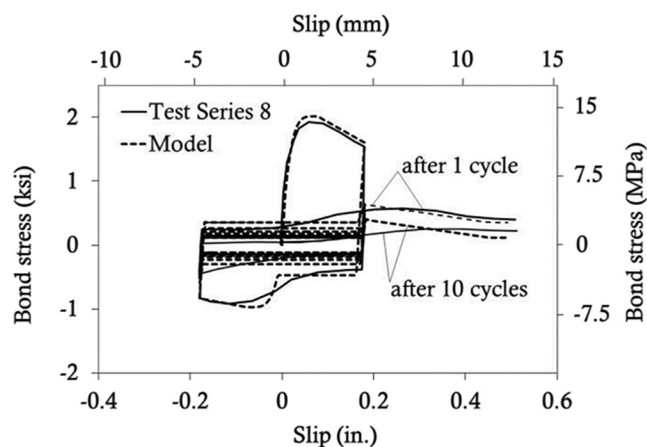
The ability of the analytical model to reproduce the bond stress-slip relations obtained from the experiments presented

in this paper and by others has been evaluated. The experimental and analytical results for monotonic load tests in Series 1 and 3 and for No. 8 (25 mm) bars tested by Eligehausen et al.³ are presented in Fig. 7. Two sets of analytical curves were generated. The first set is based on the values of τ_{max} and s_{peak} directly obtained from the monotonic tests, while the second set is based on the values estimated with the recommendations provided in a previous section. The values for both sets are presented in Table 3. The results in Fig. 7 show that once the values of τ_{max} and s_{peak} have been determined with experimental data, the ascending and descending branches are fairly well represented by the proposed polynomial functions. The curves based on the estimated values also provide a satisfactory match given the simplicity of the rules used to derive these values.

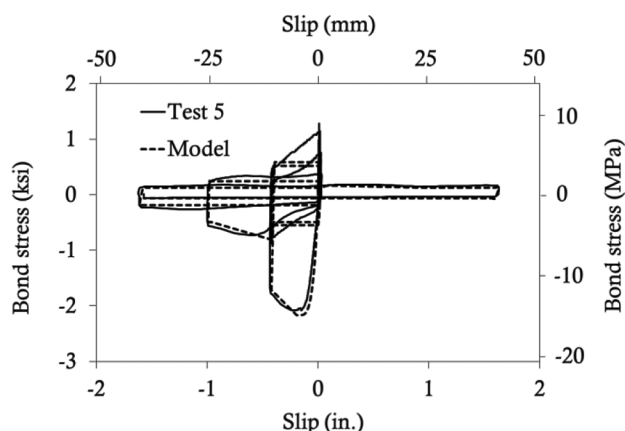
The cyclic bond stress-slip relations were reproduced analytically using the parameters calibrated with the monotonic tests. The analytical and experimental results for selected tests in Series 1, 3, and 4 are compared in Fig. 8. The model is successful in reproducing the cyclic bond stress-slip relations, including the bond strength decay. Experiments by



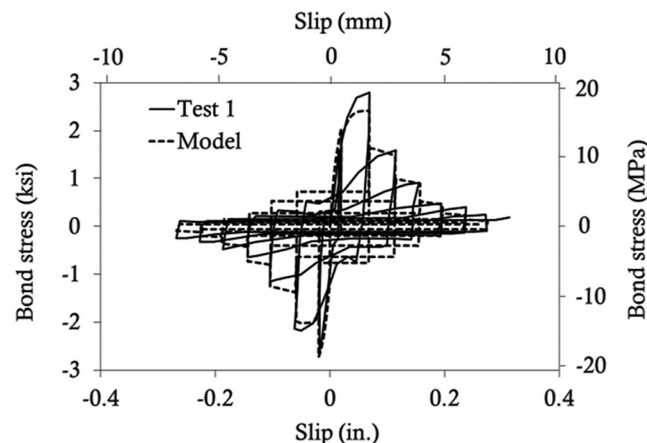
(a) Test 4, Series 1



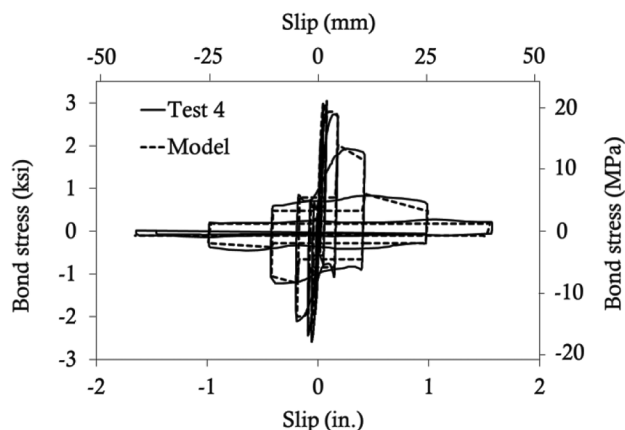
(a) No. 8 bar tested by Eligehausen et al.



(b) Test 5, Series 3



(b) No. 5 bar tested by Lundgren



(c) Test 4, Series 4

Fig. 8—Analytical and experimental results for cyclic loading.

Eligehausen et al.³ and Lundgren,¹⁷ which had smaller bars, more cycles per amplitude level, and cycles with finer amplitude increments, are also well reproduced by the analytical model, as shown in Fig. 9.

SUMMARY AND CONCLUSIONS

The bond strength and bond-slip behavior of large-diameter bars embedded in well-confined concrete are examined

Fig. 9—Analytical and experimental results for cyclic tests conducted on No. 8 (25 mm) and No. 5 (16 mm) bars.

herein. Monotonic pullout tests and cyclic pull-pull tests were conducted on No. 11, No. 14, and No. 18 (36, 43, and 57 mm) bars. All the specimens failed by pullout of the bar from the concrete. The large-diameter bars exhibited a bond stress-slip relation similar to that of No. 8 (25 mm) and smaller bars, including the bond deterioration behavior under monotonic and cyclic loads. These tests have also shown that the bond strength tends to increase slightly with increasing bar size and that the compressive strength of concrete has a notable effect on the bond strength. The bond strength observed herein is proportional to $f'_c{}^{3/4}$. Results from this and other studies have indicated that the influence of the concrete strength and bar size on the bond strength depends on the level of confinement in the concrete specimen. However, the data supporting this conclusion are limited, and a systematic and comprehensive investigation of the effects of the bar size and concrete strength on the bond strength under a wide range of confinement levels is needed to further confirm this observation and arrive at more general guidelines. Finally, for a bar positioned vertically during casting, the bond strength is smaller when the bar is pulled down than when it is pulled up. This observation is consistent with other studies.

The experimental results have been used to develop, calibrate, and validate an analytical model to predict the bond stress-slip relation of reinforcing bars embedded in well-confined concrete. The proposed monotonic bond stress-slip

curve is described by a set of polynomial functions. With the calibration of only three parameters, the model can simulate the bond stress-slip relation for bars of different sizes and concrete of different strengths. The degradation of the bond strength under cyclic slip reversals is governed by two damage parameters, which control the degradation of the bearing and friction resistances, respectively. The model successfully reproduces the bond-slip behavior of the large-diameter bars tested in this study as well as that of smaller bars tested by others, including the decay in bond strength under different load histories.

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NOTES:
