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Bond of Reinforcement in High-Strength Concrete

by Joseph Jones and Julio A. Ramirez

This paper describes an examination of the development of tension lap splices in high-strength concrete and the applicability of the restriction in the ACI 318 code where the values of $\sqrt{f_c'}$ *used to calculate development length shall not exceed 100 psi. The focus is on the development of splices with Grade 60 epoxy-coated and uncoated bars in normalweight concrete. A summary of the behavior of bond is presented to provide context for the study. It is followed by a literature review to analyze the relevant previous experimental work in this area. Based on the analysis of those data, a modification to the current code limitation is proposed. The main conclusion of this paper is that the limit of 100 psi (0.689 MPa) on the square root of the specified compressive strength in Section 12.1.2 of ACI 318-11 (Section 25.4.1.4 of 318-14) may be extended to 120 psi (0.827 MPa) based on the analysis of available data without significant modifications to the current provisions for concrete strengths up to 16,000 psi (110 MPa).*

RESEARCH SIGNIFICANCE

Over the past couple of decades, the use of high-strength concrete for structural applications has continued to increase. While there are benefits, there are also limitations imposed on the use of higher-strength concretes in design. The ACI 318 Code has an upper bound of 100 psi (0.689 MPa) on the square root of the specified concrete compressive strength ($\sqrt{f_c'}$) when used in the determination of development length for deformed reinforcement. Specifically, this requirement limits the benefit of higher-strength concretes in calculation of development and lap splice lengths of bars in tension. This paper examines the justification behind the strength limitation, and evaluates previous experimental research to evaluate the strength of splices of Grade 60 uncoated and coated deformed bars in high-strength normalweight concrete to determine if the limit on the square root of the specified concrete strength can be increased.

ACI 318 CODE PROVISIONS

The requirements for splices are found in Chapter 12 of ACI 318-11 and Chapter 25 of ACI 318-14. Specifically, Section 12.1.2 of 318-11 (ACI Committee 318 2011) and 25.4.1.4 (ACI 318-14) place a limit of 100 psi (0.689 MPa) on the square root of the specified concrete compressive strength. The equation for development of deformed bars in tension found in Section 12.2.3 as Eq. (12-1) (ACI 318-11) and in Section 25.4.2.3 (ACI 318-14) as Eq. (25.4.2.3a), and is repeated as follows

$$\ell_d = \left(\frac{3}{40} \frac{f_y}{\lambda \sqrt{f_c'}} \frac{\psi_t \psi_e \psi_s}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b$$

In this equation, ℓ_d is the required development length; f_y is the specified yield strength of the bars being developed, λ is a reduction factor when lightweight concrete is used; d_b is the nominal diameter of the bars being developed; c_b is the smaller of the distance from the center of the bar to edge of concrete or half the center-to-center spacing of bars; and K_{tr} is a factor to account for the confining effects of transverse reinforcement. The ψ factors are to account for changes in bond strength due to top cast bars (ψ_t), epoxy-coated bars (ψ_e), or bar size (ψ_s). Section 12.2.4 (ACI 318-11) and Table 25.4.2.4 (ACI 318-14) define the ψ factors as follows: ψ_t is 1.3 for members with more than 12 in. (305 mm) of fresh concrete cast below the splice and 1.0 otherwise; ψ_e is 1.5 for epoxy-coated bars with cover less than three bar diameters or clear spacing between bars less than six bar diameters, 1.2 for other epoxy-coated bars, and 1.0 for uncoated bars; and ψ_s is 0.8 for No. 6 bars and smaller, and 1.0 for larger bars; and, the product $\psi_t\psi_e$ does not need to be taken greater than 1.7. The confinement term $(c_b + K_{tr})/d_b$ is restricted to a maximum of 2.5 in Section 12.2.3 (ACI 318-11) and Section 25.4.2.3 (ACI 318-14). According to Section 12.2.1 (ACI 318-11) and 25.4.2.1 (ACI 318-14), development lengths must be a minimum of 12 in. (305 mm) long.

In the case of lap-spliced bars in tension, Section 12.14.2 (ACI 318-11) and 25.5.1.1 (ACI 318-14) limit the use up to No. 11. Tension lap splices are further classified in Sections 12.15 and 12.15.2 (ACI 318-11) and Section 25.5.2 (ACI 318-14) in accordance with two criteria: 1) the ratio of area of steel provided to area of steel required over the length of splice; and 2) maximum percent of area of tension steel spliced within the required lap length, into Class A and Class B splices. Table 1 (Table 25.2.2.1 in ACI 318-14) summarizes these requirements.

BOND BEHAVIOR

To identify the consequences of raising the upper bound for the $\sqrt{f_c'}$ in the use of ACI 318 development length equation in tension lap splices, it is important to recognize the physical behavior of bond of reinforcement in concrete. Bond forces have been classified in three categories in ACI 408R-03 (Joint ACI-ASCE Committee 408): 1) the chemical adhesion between the bar and surrounding concrete; 2) frictional forces caused by roughness between bar and concrete; and 3) bearing forces on the concrete from the bar deformations. Figure 1 shows these forces on embedded reinforcement.

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After the initial slip of the bar, the chemical adhesion is lost and no longer plays a role in the bond force transfer. While most of the force after the initial slip of the bar is transferred by the bearing between the ribs and concrete, frictional forces still play an important role in force transfer. This is evidenced with epoxy-coated bars having a lower coefficient of friction and consistently attaining lower bond capacities in tests. As the bar continues to slip, the bearing forces on the rib faces increase and frictional forces on the barrel of the bar decrease. The increase in bearing force on the rib faces increases the frictional forces on this surface. Therefore, the contact surface between the rib face and concrete is the primary location for force transfer beyond initial slip.

The bearing and frictional forces on the rib faces of the bar result in compressive and shear forces in the concrete. These forces are resolved into tensile forces, which create cracks perpendicular to the bar and conical cracks that run parallel to the bar. If the spacing between adjacent bars or the bar and the edge of the concrete surface is small, the cracks will primarily project perpendicular to the bar and lead to splitting cracks and eventual failure. If the concrete cover is large enough and there is enough space between bars, the perpendicular cracks will be suppressed by the surrounding concrete and the conical cracks will result in a pullout type failure. It has been observed that in most structural applications, splitting failure is more common and that pullout failure exists with conditions of substantial transverse reinforcement confining the surrounding concrete and/or short bond lengths (Joint ACI-ASCE Committee 408 2003). Figure 2 displays the primary cracking and failure modes for bond failure of reinforcement in concrete.

With the primary cause of failure in structural applications related to splitting cracks, it follows that the concrete tensile strength would be one of the governing factors related to the bond strength. It is assumed in the calculation of tension development and splice lengths that the tensile strength of concrete increases roughly with the square root of compressive strength ($\sqrt{f_c'}$). However, Darwin et al. (1996) indicated that the force development in development length and splice tests increases at a lesser rate with increasing compressive strength. Nevertheless, ACI Committee 318 deemed the square root variation to be sufficiently accurate for values up to 100 psi (0.689 MPa).

From a behavioral standpoint, when testing for bond failure, it has been observed that as concrete strength increases, there is less crushing of concrete in front of the ribs at failure. Because as the compressive strength increases, the tensile strength is increasing at a slower rate, the result is that tensile failure before crushing occurs if the concrete

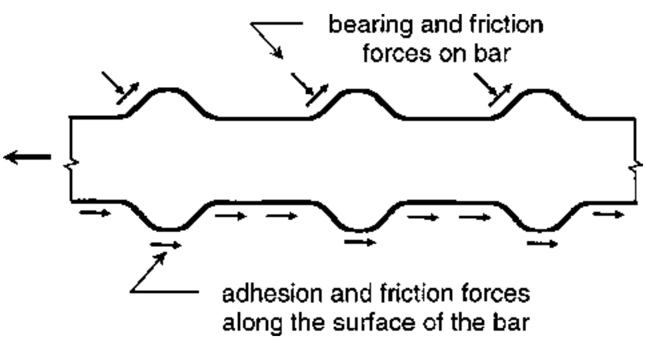

Fig. 1—Transfer forces of embedded reinforcement in tension (Joint ACI-ASCE Committee 408 2003).

Table 1—Lap splice lengths of deformed bars and deformed wires in tension (ACI 318-14)

A_{sprov}/A_{sreqd}* over length of splice	Maximum percent of A_s spliced within required lap length	Splice type	ℓ_{st}	
≥ 2.0	50	Class A	Greater of:	$1.0\ell_d$ and 12 in.
	100	Class B	Greater of:	$1.3\ell_d$ and 12 in.
< 2.0	All cases	Class B		

*Ratio of area of reinforcement provided to area of reinforcement required by analysis at splice location.

Note: 1 in. = 25.4 mm.

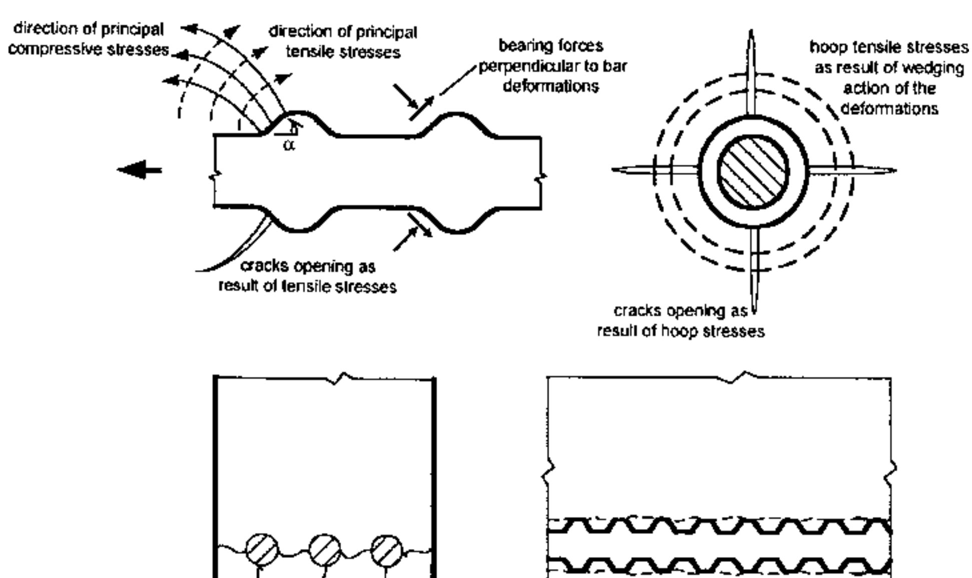

Fig. 2—Cracking mechanisms for bond failure of reinforcement embedded in concrete (Joint ACI-ASCE Committee 408 2003).

strength is high enough (Joint ACI-ASCE Committee 408 2003). Less crushing in front of the ribs translates to less slipping of the bar that concentrates the loads to fewer ribs triggering the splitting failure of the member.

Due to the consideration of concrete cover and minimum lap lengths in practice, the splitting mechanism becomes a primary concern and, thus, the role of transverse reinforcement in higher-strength concrete cannot be ignored. Transverse reinforcement acts to confine the surrounding concrete that suppresses the splitting cracks from propagating to failure, allowing the bars to reach higher bond strength in higher-strength concrete.

When removing or changing the upper limit on the square root of the specified concrete strength from the code equation, some of the more important factors to consider are concrete cover and confining reinforcement. Increasing the strength limit of the code means that using Eq. (12-1) of ACI 318-11 (Eq. (25.4.2.3a) of ACI 318-14) will yield shorter development lengths for higher-strength concretes, so shorter splice lengths should also be scrutinized.

LITERATURE REVIEW OF EXPERIMENTAL WORK

There has been extensive research done in the area of bond of deformed bars in concrete, and much of it has been summarized in ACI 408R-03. In the 408R report, a database of test results from several different experimental programs is given. The database is a compilation of results from bond tests of uncoated black bars consisting of: 478 bottom-cast, 111 top-cast, and 46 side-cast specimens. The 46 side-cast specimens were not included in this paper. Ramirez and Russell (2008) performed an extensive review of existing material as part of a program sponsored by the National Academy of Sciences through the National Cooperative Highway Research Program (NCHRP) to evaluate the current code restrictions for bond of reinforcement in AASHTO and ACI. The review included information about spliced uncoated and epoxy-coated bars, including tests from the ACI 408R-03 database.

Ramirez and Russell also performed 18 additional splice tests (12 epoxy-coated and six uncoated) to explore the behavior of bond in high-strength concrete. More specimens containing epoxy-coated splices were conducted due to the paucity of epoxy-coated bar data in high-strength concrete. In addition, Ramirez and Russell, in their analysis as part of the NCHRP Report 603 (2008), included 75 beam splice tests available in the literature from other studies by Treece and Jirsa (1989) (12 tests); Cleary and Ramirez (1991) (four tests); Choi et al. (1991) (seven tests); Hester et al. (1993) (29 tests); Cleary and Ramirez (1993) (five tests); Hamad et al. (1993) (six tests); Darwin et al. (1996) (10 tests); and Hasan et al. (1996) (two tests).

In this paper, 595 beam splice tests of uncoated bars (589 from the ACI 408 Database and six performed by Ramirez and Russell) and 87 beam splice tests of epoxy-coated bars (12 performed by Ramirez and Russell and 75 from other studies) are included in the analysis of data presented in subsequent sections.

Table 2 summarizes the total number of tests by the concrete compressive strength used, and Table 3 shows the number of tests by bar diameter for tests with concrete strength over 10,000 psi (69 MPa).

Table 2—Number of tests used for analysis by concrete compressive strength

f_c'	No. of tests, uncoated bars	No. of tests, epoxy-coated bars
$f_c' \leq 5000$ psi	305	21
5000 psi $< f_c' \leq$ 10,000 psi	164	52
10,000 psi $< f_c' \leq$ 13,000 psi	50	4
13,000 psi $< f_c' \leq$ 15,000 psi	31	2
$f_c' >$ 15,000 psi	45	8
Total	595	87

Note: 1 psi = 0.00689 MPa.

Table 3—Number of tests used for analysis by bar diameter for tests with concrete strength over 10,000 psi (68.9 MPa)

Bar diameter	No. of tests, uncoated bars	No. of tests, epoxy-coated bars
d_b = 0.75 in.	4	7
d_b = 0.984 in.	12	0
d_b = 1.0 in.	58	0
d_b = 1.128 in.	4	0
d_b = 1.41 in.	48	7
Total	126	14

Note: 1 in. = 25.4 mm.

Joint ACI-ASCE Committee 408 concluded that for concrete strengths up to 16,000 psi (110 MPa) without transverse reinforcement, the force developed in a bar in development and lap splice tests increased with the one-quarter power ($f_c'^{1/4}$). For members with transverse reinforcement, the conclusion was that the force developed in a bar increased with the power between three fourths ($f_c'^{3/4}$) and one ($f_c'^{1.0}$). These powers diverge from the currently used square root of compressive strength or strength raised to the one-half power ($f_c'^{1/2}$). This indicates that for higher-strength concretes, the ACI Code may be overestimating the contribution of the concrete strength to bond strength for members without transverse reinforcement while underestimating the concrete strength contribution for members with transverse reinforcement. However, ACI Committee 318 deemed the square root as sufficiently accurate for values of $\sqrt{f_c'}$ up to 100 psi (0.689 MPa).

Ramirez and Russell (2008) concluded that for epoxy-coated bars, the provisions in Chapter 12 of ACI 318 could be extended for concrete strengths up to 15,000 psi (103 MPa) while using only one epoxy coating factor of 1.5, regardless of the bar clear spacing or concrete cover. For uncoated bars, it was concluded that the ACI 318 equations for splice lengths of top-cast uncoated bars could be extended up to 16,000 psi (110 MPa) for normalweight concrete.

ANALYSIS OF DATA

The data compiled by Joint ACI-ASCE Committee 408 and those by Ramirez and Russell (2008) of deformed coated and uncoated bars are re-examined in the this section to eval-

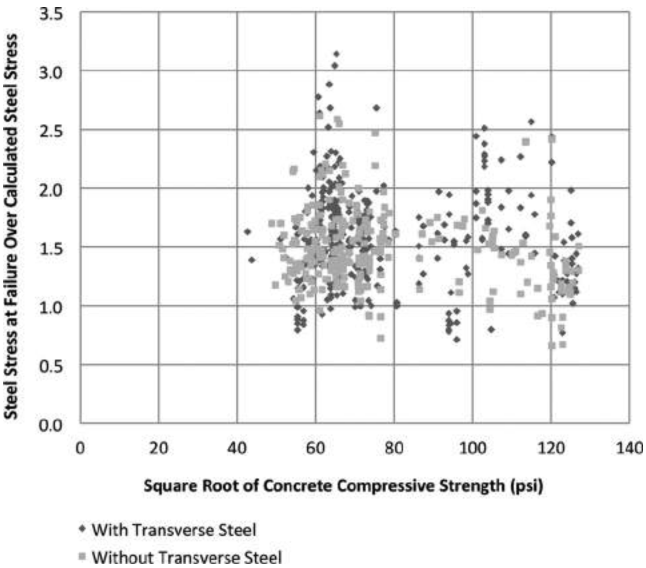

Fig. 3—Calculated steel stress over steel stress at failure versus square root of concrete compressive strength for uncoated black bars. (Note: 1 psi = 0.00689 MPa.)

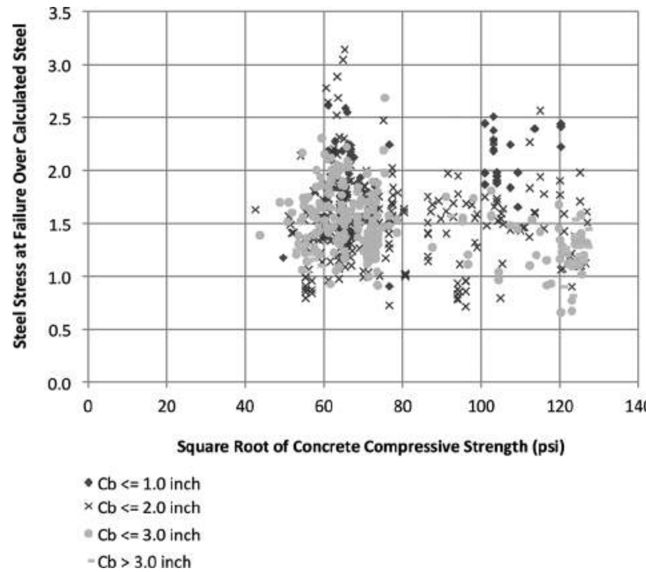

Fig. 4—Calculated steel stress over steel stress at failure versus square root of concrete compressive strength for uncoated black bars. (Note: 1 psi = 0.00689 MPa.)

uate the potential for changing the current limit of 100 psi (0.689 MPa) as the specified concrete compressive strength is increased. Details of the all the specimens included in this evaluation can be found in ACI 408R-03 and in the Ramirez and Russell NCHRP report.

Uncoated bars

For this analysis, only the top and bottom bar data were used and side-cast specimens were omitted. The first step in analysis was to rearrange Eq. (12-1) of ACI 318-11 (Eq. (25.4.2.3a) of ACI 318-14) to solve for steel stress in terms of splice length, concrete strength (ignoring the current code limit), bar size, bar cover, and bar property factors. This rearrangement created the following equation

$$f_s = \left(\frac{40\lambda\sqrt{f_c'}}{3} \frac{\left(\dfrac{c_b + K_{tr}}{d_b}\right)}{\psi_t \psi_e \psi_s} \right) \frac{\ell_s}{d_b}$$

In this equation, f_y (yield stress of the steel) is replaced with f_s (calculated bond stress) and ℓ_d (required development length) is replaced with ℓ_s (splice length used in the test specimen). All other variables remain the same. From this equation, the calculated steel stress could be used to obtain a bond efficiency ratio. The ratio is calculated dividing the experimental stress in the spliced bar at failure by the calculated stress from the equation aforementioned. Ramirez and Russell (2008) followed this approach as well.

Figure 3 shows the plot for specimens with transverse steel and without transverse steel. Figure 4 displays the same information but broken into the ACI 318-defined c_b value (smaller of center of bar to concrete surface or one-half center-to-center spacing). Note that a value of 1.0 on the y-axis indicates a test that the calculated steel stress at failure was identical to the test value, while values lower or higher than 1.0 represent cases where stress at failure was lower or higher than the ACI calculated value. For uncoated bars shown in Fig. 3, the average value of the bond-efficient ratio for tests with concrete strength at or below 10,000 psi (69 MPa) was 1.56 with a coefficient of variation of 4.19. For tests with concrete strengths above 10,000 psi (69 MPa), the average was 1.48 with a coefficient of variation of 3.43. This is summarized in Table 4.

From Fig. 3, it can be seen that beyond the code limit of 100 psi (0.689 MPa) on the square root of concrete compressive strength, only 10 of the tests had a steel stress at failure below the ACI Code-calculated value. This is equates to approximately 8% of the tests with concrete strength above 10,000 psi (69 MPa) not meeting the code equation, and only two of the 10 tests had transverse steel.

Further, one of the two tests with transverse reinforcement with concrete strength over 10,000 psi (69 MPa) that had a bond efficiency ratio less than 1 came from the study by Azizinamini et al. (1999). It is interesting to note that this particular specimen used the longest splice length, by 12.5 in. (305 mm), of any other test of a similar nature. This may be the cause for the lower bond efficiency ratio. With the longer splice length being used for calculation, the stress estimated by the ACI equation comes out to be nearly 90,000 psi (620 MPa). This particular bar's yield strength was 74,000 psi (510 MPa) so it is unlikely for a splice to reach this calculated value of stress that is drastically over its yield point. The specimen failed with a maximum stress in the bar reaching nearly 70,000 psi (483 MPa), which was near the yield stress of the bar. The other test with concrete strength greater than 10,000 psi (69 MPa) concrete with transverse steel and a bond efficiency ratio below 1 was completed by Kadoriku (1994). This test had No. 6 bars for the splices, which means the bar size factor (ψ_t) is reduced to 0.8. This reduction factor increases the projected stress in the bar for a given splice length. It should also be pointed out that the cluster of specimens with transverse steel and bond efficiency ratios less than 1 that lie below

Table 4—Average and coefficient of variation (COV) of bond efficiency ratios for uncoated black bars*

Concrete strength	Average	COV
$\sqrt{f_c'} \leq 100$ psi	1.56	4.19
$\sqrt{f_c'} > 100$ psi	1.48	3.43

*As seen in Fig. 3 by concrete compressive strength.

Note: 1 psi = 0.00689 MPa.

the 100 psi (0.689 MPa) mark in Fig. 3 all have No. 6 or smaller bars. This is consistent with the recommendation by Joint ACI-ASCE Committee 408 (2003) to remove the 0.8 reduction factor for smaller bars. If 1.0 is used for all specimens in the database, all but three of the specimens with transverse steel above the 90 psi (0.62 MPa) mark on the horizontal axis in Fig. 3 would have bond efficiency ratios greater than 1.

Another consideration is that for the square root of concrete compressive strengths between 100 and 120 psi (0.689 and 0.83 MPa), 6.7% of tests were below 1 on the bond efficiency ratio. This is similar to the 6.4% of tests that fell below 1.0 on bond efficiency ratios for specimens with square root of concrete compressive strengths value below 100 psi (0.689 MPa).

Figure 4 shows the same data identified in function of c_b. In this figure, it can be seen that the ACI 318 provisions are more conservative for members with values of c_b less than 2.0 throughout the range of concrete strengths. It appears also from the plot that for higher-strength concretes, the code may slightly under-predict the stress in the steel at failure for these cases.

In a typical design scenario, ACI 318-11 Eq. (12-1) (ACI 318-14 Eq. (25.4.2.3a)) is used to calculate the splice length needed for development. It is possible to define a development length efficiency ratio calculating the development length from Eq. (12-1) and dividing it by the splice length used in the same tests. Figure 5 shows a plot of the development length efficiency ratio versus the ratio of steel stress at failure to yield stress only for tests with concrete strength above 10,000 psi (69 MPa) (neglecting the current code limit). This graph permits a comparison of the splice length and whether the steel in the specimen reached yield. Note that a value of 1.0 on the x-axis represents the case where the splice length used is the same as the code-calculated development length. A value less than 1.0 represents a case where the splice length was longer than the code-calculated development length. A value of 1.0 on the y-axis defines a specimen that failed with the steel reaching its yield strength while a value lower or higher is a specimen that failed with the steel below or above its yield strength, respectively.

From Fig. 5 it can be seen that the majority of tests with concrete strengths over 10,000 psi (69 MPa) had splice length less than the code-calculated development length as researchers looked for splitting type failures to evaluate the bond strength of the splice prior to yielding of the steel in the splice region. There were 14 tests with concrete strengths above 10,000 psi (69 MPa) that had splice lengths above or near the calculated development length, and of those, seven failed when the steel stress was still below the yield point.

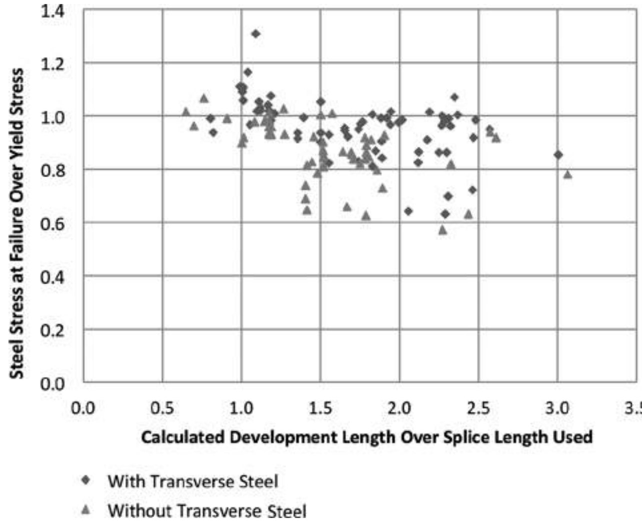

Fig. 5—Calculated development length over used splice length versus steel stress at failure over yield stress for uncoated black bars with concrete strength greater than 10,000 psi (68.9 MPa).

Of the seven that had splices near to or longer than the calculated development length and failed at a steel stress below the yield point, four did not have transverse steel.

Considering all data for concrete strength above 10,000 psi (69 MPa), it is possible to conclude that the addition of transverse reinforcement resulted in increases in the stress at failure for all splice lengths. With this, it might be tempting to extrapolate the findings of shorter splice length tests to longer splice lengths, but this may be inappropriate, particularly for high-strength concrete. As discussed previously, higher-strength concrete results in higher concentrated forces in both the concrete and the bar, meaning that bar stresses are not distributed evenly over the lap length; thus, results from shorter splices should not be linearly extrapolated to account for longer splices without additional tests to support this extension.

Ramirez and Russell (2008) examined the contribution of transverse reinforcement more closely. The six tests performed on uncoated black bars consisted of three pairs of nearly identical tests with the addition of transverse steel to one of the specimens within each pair. This allowed a more direct comparison of test results with transverse steel being the main variable. Each of the three pairs had different splice lengths. The variation in concrete strengths for the six tests ranged from 14,600 to 16,200 psi (100 to 112 MPa). This variation is small enough to have only a marginal effect on the outcome of the results.

Figure 6 shows the bond efficiency stress ratio against the splice length for these six tests. There is a deceptive trend in this figure that suggests decreasing bond efficiency with the addition of the transverse steel. However, it is important to point out that the stress at failure in all six tests was at or above the yield strength of the steel. Because the steel had yielded for these tests, the exact bar stresses could not be known and the visual trend in Fig. 6 of decreasing bond efficiency ratio with the addition of transverse steel is not a tangible result. The steel stress at failure did not increase

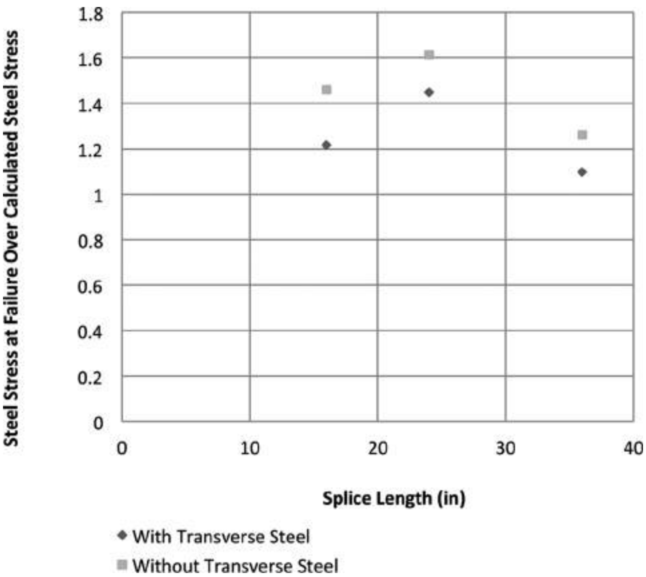

Fig. 6—Calculated steel stress over steel stress at failure versus splice length for six uncoated black bar tests by Ramirez and Russell. (Note: 1 in. = 25.4 mm.)

Table 5—Average and coefficient of variation (COV) of bond efficiency ratios for epoxy-coated bars*

Concrete strength	Average	COV
$\sqrt{f_c'} \leq 100$ psi	1.50	4.87
$\sqrt{f_c'} > 100$ psi	1.37	4.56

*As seen in Fig. 7 by concrete compressive strength.

Note: 1 psi = 0.00689 MPa.

proportionally to the increase in transverse steel because the longitudinal lapped steel had already reached yield, thus making the bond efficiency ratio decrease. The stress in the steel would not be able to increase beyond the yield point unless the displacement was large enough to cause strain hardening. In typical splices designed for strength, it is desirable for the steel to reach yield to provide a more ductile failure instead of a brittle one.

Epoxy-coated bars

For epoxy-coated bars, an identical process was used to analyze the data compiled by Ramirez and Russell. Because there were only two tests with concrete strengths above 10,000 psi (69 MPa) in the database before the tests completed by Ramirez and Russell, all of the data on epoxy-coated bars throughout the range of concrete strengths were considered from the start. Figure 7 shows a plot of the bond efficiency ratio on the vertical axis against the square root of the concrete compressive strength on the horizontal axis. Refer to Table 5 for the average and coefficient of variation (COV) of the bond efficiency ratio for the epoxy-coated bars by concrete compressive strength. The data in Fig. 7 indicate that the bond efficiency ratio on the vertical axis was greater than 1.0 in all but one of the tests above the 100 psi (0.689 MPa) code limit. Thus, the actual steel stress at failure was larger than the calculated ACI stress in all but one of the tests above the limit of 100 psi (0.689 MPa). The one specimen where the calculated code value was less than the stress at failure was a test performed by Ramirez and Russell that did not have transverse reinforcement. At first glance, it also appears the use of transverse steel has less of an impact on epoxy-coated bars than for uncoated bars, although every specimen with transverse reinforcement was above 1.0 for the bond efficiency ratio.

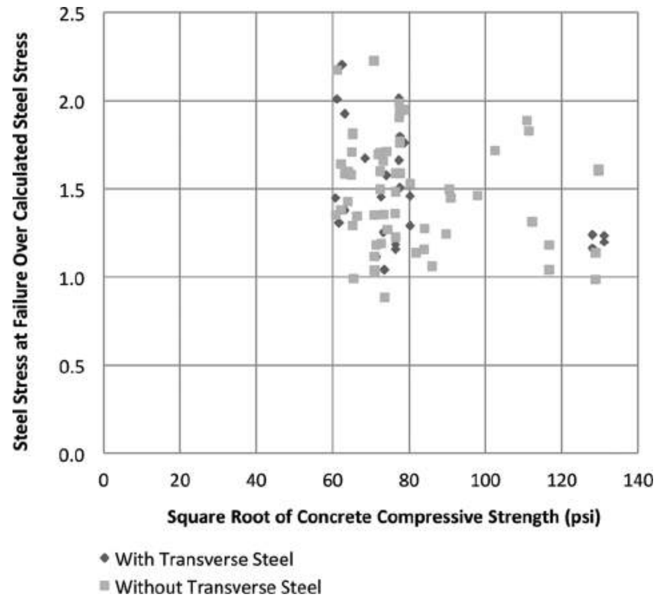

Fig. 7—Calculated steel stress over steel stress at failure versus square root of concrete compressive strength for epoxy-coated bars. (Note: 1 psi = 0.00689 MPa.)

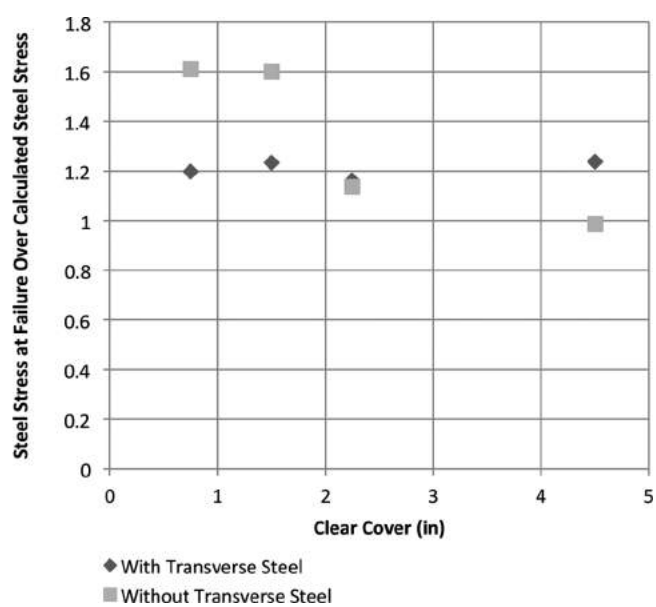

Fig. 8—Steel stress at failure over calculated steel stress versus clear cover for epoxy-coated bars in high-strength concrete. (Note: 1 in. = 25.4 mm.)

The tests performed by Ramirez and Russell included 12 tests of epoxy-coated bars in high-strength concrete. The final eight tests had a range of concrete strengths from 16,400 to 17,200 psi (113 to 119 MPa); this variation is small enough to possibly assume that there would be little effect between them due to concrete strength. This set of eight can be categorized into four sets of pairs with different clear cover for each

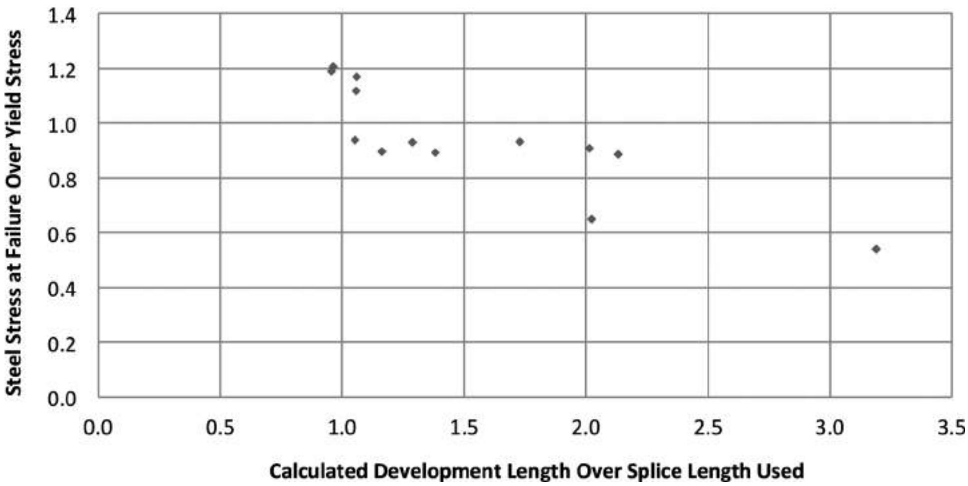

Fig. 9—Calculated development length over used splice length versus steel stress at failure over yield stress for epoxy-coated bars in concrete of 10,000 psi (68.9 MPa).

set and the addition of transverse steel being added to one of the specimens in each of the four sets. Figure 8 displays these eight pairs and their corresponding bond efficiency ratios plotted against the concrete cover on the horizontal axis.

With smaller concrete cover, Fig. 8 looks similar to the graph for uncoated bars (Fig. 6) with the addition of transverse steel lowering the bond efficiency ratio. However, as the clear cover of the specimens increases, the specimens without transverse steel see a drop in the bond efficiency ratio. This corroborates a similar conclusion to that of Ramirez and Russell that the epoxy coating factor should not be reduced even if the cover is large and should remain 1.5 for all epoxy-coated bars. The transverse reinforcement appears to also provide some consistency with the bond efficiency ratio, as all the values with transverse reinforcement from Fig. 8 fall between 1.14 and 1.23.

As mentioned previously, the epoxy coating factors (ψ_e) vary from 1.5 and 1.2, depending on cover and bar spacing. The drop in epoxy coating factor is mitigated with the presence of transverse steel. The smaller epoxy factor occurs with larger cover, while larger cover increases the confinement term $(c_b + K_{tr})/d_b$ in the denominator of Eq. (12-1) of ACI 318-11 (Eq. (25.4.2.3a) in ACI 318-14), essentially canceling out the decrease of ψ_e in the numerator.

Figure 9 displays the ratio of steel stress at failure to yield stress of the bar against the ratio for calculated development length to splice length for concrete strengths above 10,000 psi (69 MPa). With so few data, the tests with and without transverse steel were combined into the same series. As can be seen in Fig. 9, there are very few data with higher-strength concretes for epoxy-coated bars. All but two tests with concrete strengths greater than 10,000 psi (69 MPa) were performed with splice lengths less than the development length calculated by the ACI. While caution should be exercised in the extrapolation to longer development lengths with higher-strength concretes, it does appear that development length calculated using the ACI 318 provision is likely to result in conservative estimates of the steel stress at failure.

CONCLUSIONS AND FUTURE WORK

The experimental data for splices with uncoated black bars indicated that for concrete strengths up to 14,400 psi (99 MPa), the ACI-calculated stress exceeded the stress at failure in the splice region, while for strengths up to 16,000 psi (110 MPa), the ACI-calculated stress exceeded the experimental value when the splice is confined by transverse reinforcement. The exception of two specimens with transverse steel whose stress at failure did not exceed the calculated value were previously discussed and could be attributed to yielding of the longitudinal steel prior to splitting failure due to the length of the splice or to the smaller bar diameter used in the splice. The current code calculations for epoxy-coated bars with concrete strengths up to 17,000 psi (117 MPa) were conservative for all tests with the exception of one specimen without transverse reinforcement.

With regard to development lengths of Grade 60 epoxy-coated and uncoated reinforcement in normalweight concrete, the evaluation of data conducted in this paper suggests that the limit of 100 psi (0.689 MPa) in the ACI 318-11 and ACI 318-14 on the square root of concrete compressive strength can be extended to 120 psi (0.827 MPa) with no change in the Eq. (12-1) of ACI 318-11 (Eq. (25.4.2.3a) in ACI 318-14), as there is a similar percentage of specimens with calculated stresses less than experimentally achieved stresses with the square root of concrete compressive strength between 100 and 120 psi (0.689 and 0.827 MPa) (6.7%) as for those with concrete compressive strengths below the code limit value of 100 psi (0.689 MPa) (6.4%).

Although at this time there is further evidence that the limit could be potentially extended to 125 psi (0.861 MPa) and beyond, additional data should be developed to increase the robustness of the evidence. These tests should be focused on exploring the effects of transverse reinforcement throughout the splice region in higher-strength concrete. A further investigation into the failure mechanisms of splices in higher-strength concretes would need to be undertaken to ensure that the ductility of beam failures is maintained when the shorter and possibly more confined splices are being used. At this time, there are not enough data in higher concrete strengths to remove the limit entirely.

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