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A Reevaluation of Test Data on Development Length and Splices

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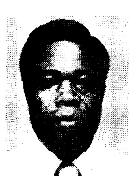
An equation is developed for calculating development and splice lengths for deformed bars. The equation is based on a nonlinear regression analysis of test results of beams with lap splices and reflects the effect of length, cover, spacing, bar diameter, concrete strength, and transverse reinforcement on the strength of anchored bars. Design recommendations are proposed and compared with current provisions for development splice length. The comparison shows that for the minimum cover and spacing (no transverse reinforcement) the proposed provisions require increases in anchorage lengths of 10 to 25 percent over current provisions. If cover is increased or transverse reinforcement is added, the splice length for large bars may be reduced by as much as 60 percent over that required by present provisions.

Keywords: **anchorage (structural)**; beams (supports); bond (concrete to reinforcement); cover; **deformed reinforcement**; failure; lap connections; regression analysis; reinforced concrete; **reinforcing steels**; **splicing**; stress transfer; structural engineering.



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THE DESIGN OF LAP SPLICES and development lengths in reinforced concrete structures is of continuing interest to structural engineers because of the implications of splice length on detailing and on structural performance.

Splice lengths in current codes[1,2] are based on the development length l_d. Depending on the severity of stresses, the splice length is increased. For example, if more than 50 percent of the bars are spliced in the region of maximum stress ($f_s > 0.5f_y$), the splice length $l_s = 1.7l_d$. The basic premise is that the cover on the bar may be at a minimum value and that the splice should develop at least 25 percent more stress than computed from a consideration of moments at the splice region.

It should be noted that development lengths l_d in ACI 318-71 are based on ultimate bond stresses specified in ACI 318-63. Ultimate bond stress for bottom bars was a function of concrete strength f_c' and bar diameter d_b where $u = 9.5\sqrt{f_c'}/d_b \leqq 800$ psi (56 kgf/cm^2). Assuming a uniform distribution of bond stress along a bar with area a_b, the length needed to develop 125 percent of yield is determined in the following manner. Equating the tensile force on the bar with the total bond force on the surface area of the bar and solving for l_d the equation in ACI 318-71[1] is derived.

$$l_d = \frac{a_b\,(1.25f_y)}{d_b\,(9.5\sqrt{f_c'}/d_b)} \approx 0.04\,a_b f_y/\sqrt{f_c'} \qquad (1)$$

No ϕ factor was specified for development length computations because the area of steel provided at a section was based on a $\phi = 0.9$ (flexural reinforcement), and, in addition, the length was based on assuming that the steel develops $1.25f_y$.

It is important to note that the data available regarding the strength of lapped splices was limited at the time the current provisions were developed. Therefore, a reevaluation of design specifications for splices and development lengths considering recent test data is needed.

A FAILURE HYPOTHESIS FOR ANCHORED BARS

Stress transfer between reinforcing bars in concrete

Stress from a deformed bar is transferred to the concrete mainly by mechanical locking of the lugs with the surrounding concrete. The resultant force exerted by the lug on the concrete is inclined at an angle β to the axis of the bar (Fig. 1) and the radial component causes splitting of the surrounding concrete at failure. If the stress component parallel to the axis of the bar is u, the radial stress component of the bond force is $u \tan \beta$. The radial stress can be regarded as a water pressure acting against a thick-walled cylinder having an inner diameter equal to the bar diameter and a thickness C, the smaller of (1) the clear bottom cover C_b, or (2) half the clear spacing C_s between the next adjacent bar (see Fig. 2a). The capacity of the cylinder depends on the tensile strength of the concrete. With $C_b > C_s$, a horizontal split develops at the level of the bars, and is termed a "side split failure." When $C_s > C_b$, a "face-and-side split failure" forms with longitudinal cracking through the bottom cover followed by splitting along the plane of the bars. When $C_s >> C_b$, a "V-notch failure" forms with

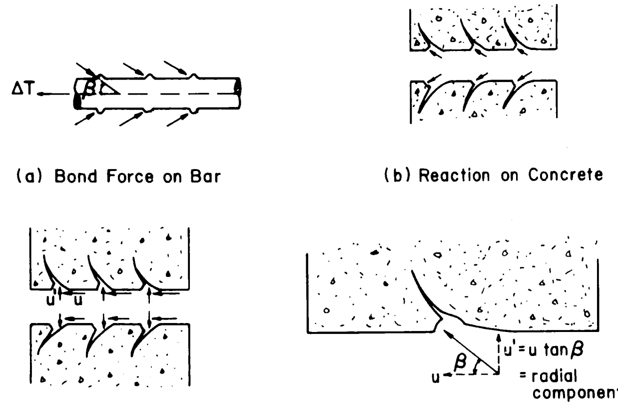

Fig. 1—Forces between deformed bar and concrete

longitudinal cracking followed by inclined cracking. The splitting patterns in Fig. 2 have been described previously.[3] In a lap splice where the bars are side by side, the two cylinders to be considered for each splice interact to form, in section, an oval ring, as shown in Fig. 2b. The failure patterns are similar to those of single bars.

It is possible with the water pressure analogy to analyze the stress in a concrete cylinder surrounding a single bar and this has been done by Tepfers.[4] No attempt has yet been made to analyze the stresses in the concrete cylinder having an oval ring cross section surrounding two bars as

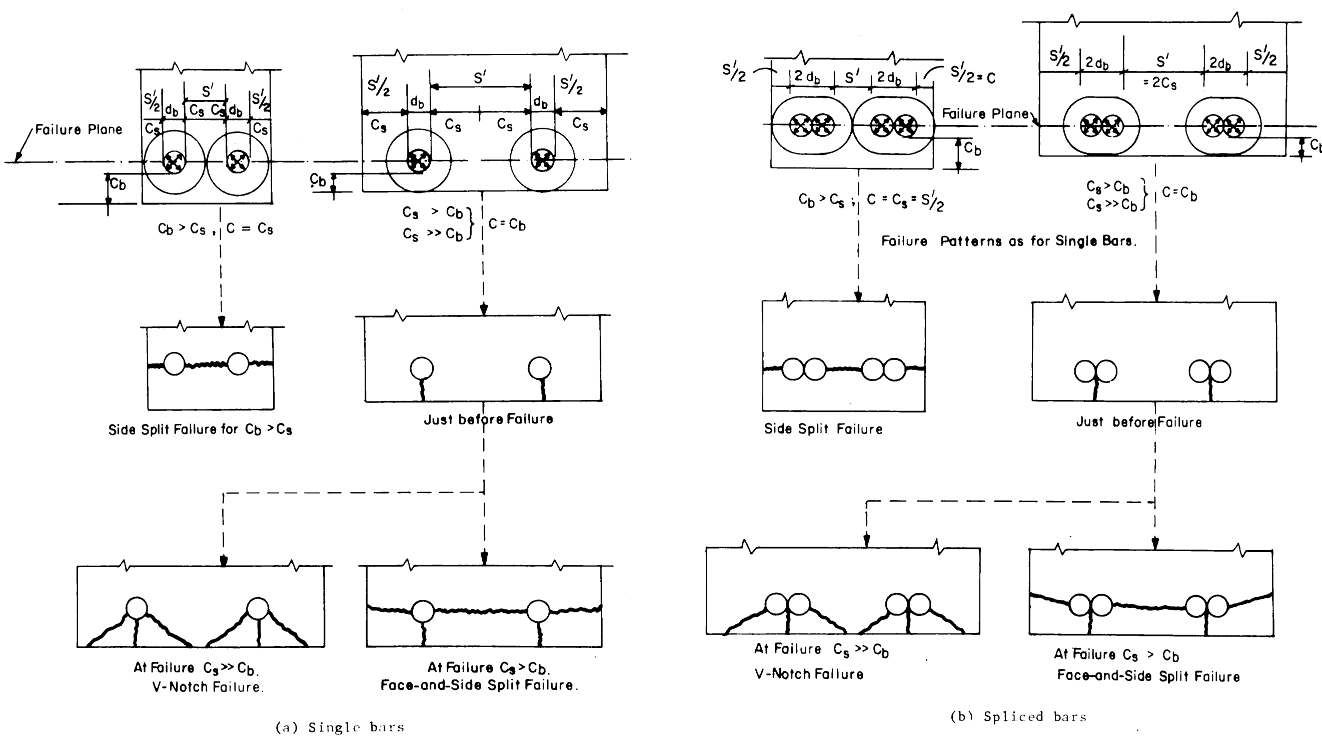

Fig. 2—Failure patterns of anchored bars

in Fig. 2b because of the uneven distribution of bond stress and the uncertainty in the value of β.

Measurement of bar strains along lap splices by Ferguson and Briceno[5] and also by Tepfers[4] shows that the strain variation along the splice becomes approximately linear near the ultimate load. Therefore, the tangential stress u is constant and can be determined from the maximum stress in the bar, i.e., $u = d_b f_s / 4l_s$. Consequently, if the value of β is known, it is possible to determine the radial force causing splitting in the failure plane. Goto[6] determined experimentally that the angle of inclination of the force can vary from 45 to 80 deg and depends on the orientation of the bar ribs. By equating the tensile resistance of concrete to the splitting forces, a relationship between material and geometrical properties of the splice section can be determined. With this concept Ferguson and Briceno[5] developed equations for side split and face-and-side split failures assuming $\beta = 45$ deg. This was later modified[7] by assuming that the splitting force is related to bar force but may not be equal to it (i.e., β may be more or less than 45 deg).

BARS WITHOUT TRANSVERSE REINFORCEMENT

Analysis of data—Splice tests

Since the value of β can vary substantially depending on the assumptions made, an empirical approach appeared to be more promising than a theoretical one. It was assumed that the failure of the splice occurs following the appearance of cracks either at the sides or on the tension face (Fig. 2). This reduces to one parameter the influence of cover and spacing and is an essential departure from the empirical approach by Ferguson and Krishnaswamy,[7] where both bottom cover and side spacing were considered as separate parameters.

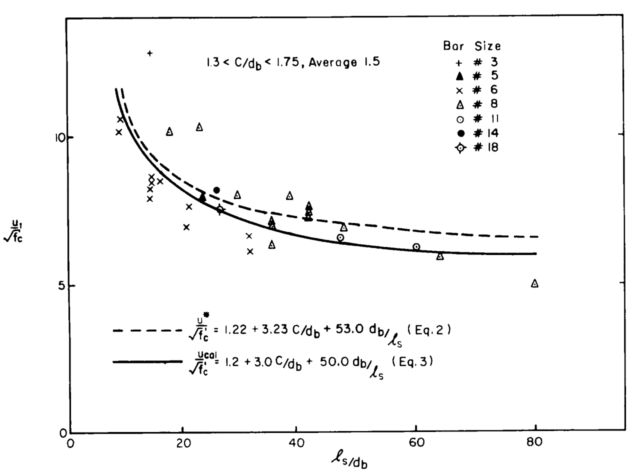

Fig. 3—Variation of $u_t/\sqrt{f_c'}$ with l_s/d_b at an average C/d_b of 1.5

Test results indicate that the average bond stress ($u = d_b f_s / 4l_s$) for a lap splice in a constant moment region and without transverse reinforcement depends on the tensile strength of the concrete, the cover C (as defined in Fig. 2), the diameter d_b of the bar, and the length of the splice l_s. The concrete tensile strength is considered to be proportional to $\sqrt{f_c'}$. Bond tests by Mathey and Watstein[8] indicated that u varies approximately linearly with d_b/l_s. Various functions were investigated with the aim of retaining a simple equation for conversion to a design provision. The constants in the equations studied were determined from a nonlinear regression analysis of the results of 62 beams which were tested by Chinn, Ferguson, and Thompson,[9] Ferguson and Breen,[10] Chamberlin,[11] and Ferguson and Krishnaswamy.[7] The beams had one or two splices with the bars in contact and all the bars were spliced at the same section. All the beams were tested in flexure with constant moment all along the splice length. A review of test results indicates that failures due to splitting after the bar yields are dependent on the magnitude of inelastic bar elongation relative to the concrete. Splitting may be produced in the cover in one test just as the steel reaches yield, while for different geometry a splitting failure occurs at yield stress but after the bar has undergone large inelastic deformations. In both cases the recorded strength is the same, but the phenomena producing failure is different. It should also be noted that if a bar yields and the test is stopped without splitting the concrete, the same anchorage strength (i.e., yield of the bar) will be recorded as for a companion specimen with twice the development or anchorage length. The anchorage strength is the same but the tests give no information regarding the response as a function of embedded length or due to concrete splitting phenomena. Since the objective was to develop an approach for predicting anchorage strength, not ductility, only specimens in which the steel did not yield were included. The regression analysis gave the following equation:

$$u^*/\sqrt{f_c'} = 1.22 + 3.23\, C/d_b + 53 d_b/l_s \qquad (2)$$

where u^* denotes the selected best fit equation for beams with constant moment over the splice length.

The test results were grouped according to C/d_b ratios and the measured bond stresses [$u_t = f_s$ (measured) $\times\, d_b/4l_s$] divided by $\sqrt{f_c'}$ are plotted against l_s/d_b in Fig. 3 for tests with C/d_b ranging from 1.3 to 1.75. Eq. (2) is shown for the average C/d_b ratio (1.5) of the tests plotted. Similar plots were made for other ranges of C/d_b ratios and are

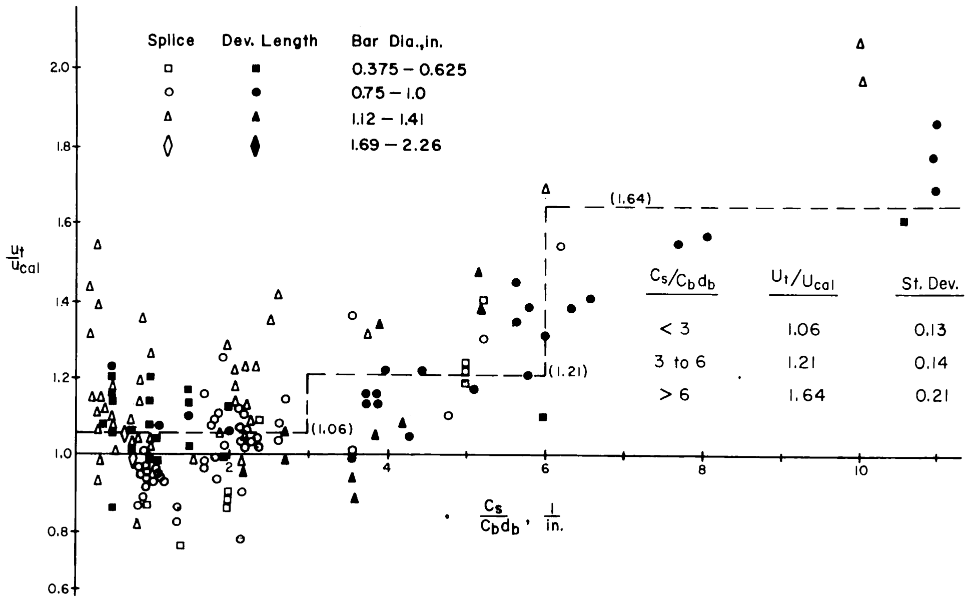

Fig. 4—Effect of wide spacing

contained in Reference 12 along with a complete tabulation of all test results used in this report. Eq. (3) was obtained by rounding off the coefficients in Eq. (2) to produce a slightly lower value of $u/\sqrt{f_c'}$ as shown in Fig. 3.

$$u_{cal}/\sqrt{f_c'} = 1.2 + 3C/d_b + 50d_b/l_s \qquad (3)$$

Using the results from the 62 tests mentioned previously, ratios of u_t/u_{cal} were calculated. The average $u_t/u_{cal} = 1.07$ for all 62 tests with a standard deviation of 0.15. If eight tests with the ratio of side to bottom cover $C_s/(C_bd_b)$ greater than 3 are eliminated, the average u_t/u_{cal} for the remaining 54 tests is 1.03 with a standard deviation of 0.12. In Eq. (2) and (3) the anchorage strength increases as the cover to bar diameter ratio increases. However, it is obvious that at large C/d_b ratios the mode of failure may be a direct pullout without splitting the cover. Most of the data on which the empirical equation is based are for C/d_b ratios of 2.5 or less. For design purposes, where C/d_b exceeds 2.5, Eq. (2) and (3) should be evaluated using $C/d_b = 2.5$.

Influence of moment gradient

Ferguson and Briceno[5] and Ferguson and Krishnaswamy[7] tested beams in which the splice was in a region of varying moment and suggested a modification for splice strength as follows:

$$u_{cal}\text{ (moment gradient)} = u_{cal}\text{ (constant moment)}$$

$$\frac{2}{(1+k)} \qquad (4)$$

where k is the ratio of the smaller stress to the larger stress at the two ends of the splice. However, if the failure of a splice coincides with the failure of a "cylinder" of concrete surrounding the bar or bars, a moment gradient should have little or no effect on the stress at failure. An anchored bar, either an individual bar or one bar in a splice, is subjected to the same stresses at the boundaries —maximum at the lead end and zero at the tail end. To determine the validity of this approach, the ratio u_t/u_{cal} was computed for 28 splice tests reported in References 5 and 7, in which a moment gradient existed along the splice. Considering 20 tests in which $C_s/(C_bd_b) < 3$, the average value of u_t/u_{cal} was 1.12 with a standard deviation of 0.13. There was no tendency for the ratio of u_t/u_{cal} to become large with smaller values of k. Although Eq. (3) slightly underestimates the strength of splices subjected to a moment gradient, a modification for such splices does not seem necessary. However, in the tests with the splice in the region of variable moment the splices were subjected to a fairly low constant shear force. There are indications that a splice may not perform as well in a region of high varying shear.[13]

Other splice tests

A number of additional splice tests reported in the literature were omitted in the initial development of Eq. (2) and (3). A series of eleven wide specimens containing five or six spliced bars to simulate a retaining wall was tested by Thompson, et al.[14] The purpose of the tests was to determine whether the outside or edge splice initiates failure of the specimen. At failure the stress in the edge splices was less than in the interior splices in most tests. Considering all splices in the section, the

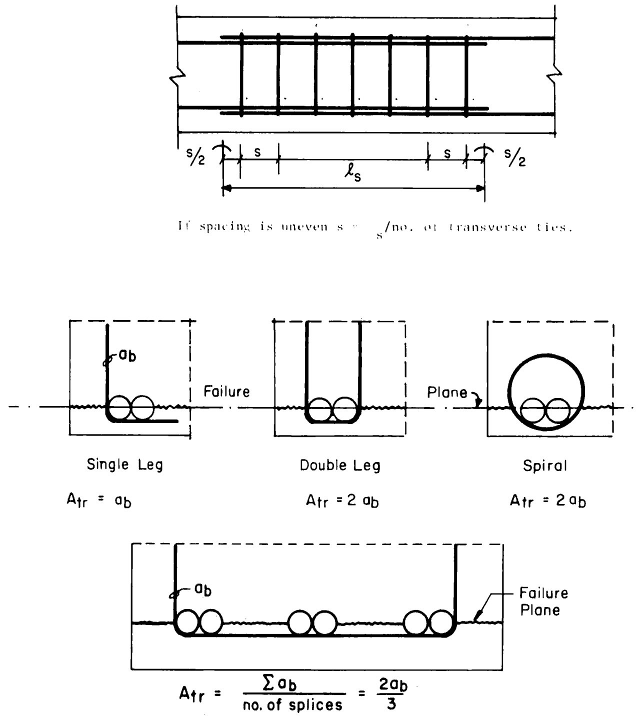

Fig. 5—Transverse reinforcement: Definitions

average u_t/u_{cal} was 1.13 and for the edge splices only u_t/u_{cal} averaged 0.97.

A major study of splices was reported by Tepfers.[4] The average u_t/u_{cal} was 1.18 for 92 tests and the standard deviation was 0.32. While the correlation between computed and measured stresses was not as close for Tepfers' tests as for the other tests reported here, the deformed bars appeared to be different from those used in the United States and concrete strengths were reported for cubes and required conversion to cylinder strength for use in Eq. (3).

Application to development lengths

Similar behavior in cracking and splitting has been observed in tests for development lengths and lap splices (Fig. 2). Therefore, the empirical equation for splice strength should be applicable to development lengths. To check this, Eq. (3) was used to predict strength in development length tests conducted by Ferguson and Thompson[15,16] and Chamberlin.[17] Fig. 4 is a plot of u_t/u_{cal} versus the ratio $C_s/(C_b d_b)$ for both development length and splice tests. There is no definitive trend for splice or development length tests to be segregated. For the same bar diameter, cover, clear spacing, and concrete strength, the same length is required for a lap splice as for development length.

Effect of wide spacing

The reduction of the cover parameter to a single ratio (cover to bar diameter) simplifies the form of the empirical equation and appears to work well as long as the ratio $C_s/(C_b d_b)$ is not large ($<$ 3 or 4). However, with large side or clear spacing, the concrete outside the "minimum" cylinder surrounding the bar tends to restrain splitting across the plane through the anchored bars. Evidence of this is the "V-notch" type of failure observed in tests with large bar spacings. In examining the ratios of u_t/u_{cal} in Fig. 4, it is obvious that with increasing values of $C_s/(C_b d_b)$, u_t/u_{cal} increase proportionally. The average value of u_t/u_{cal} is shown in dashed lines in Fig. 4 for three ranges of $C_s/(C_b d_b)$. For design purposes it may be sufficient to modify splice and development lengths in those cases where $C_s/(C_b d_b)$ is greater than 3. It should be noted that crack control provisions may determine maximum spacings of bars for flexure in many cases.

BARS WITH TRANSVERSE REINFORCEMENT

Influence of transverse reinforcement

In order to evaluate the effect of transverse reinforcement, the results of splice tests reported in References 5, 7, 10, and 18 and development length tests reports in Reference 15 and 8 were analyzed. As discussed above, only tests in which failure occurred before the bars yielded were included. However, it was clear from the test results that transverse steel improves ductility of the anchorage. The variations of strength with several parameters reflecting the confinement provided by the transverse steel were examined. The area of transverse reinforcement A_{tr} was defined, as shown in Fig. 5. The spacing s is the average spacing of ties along the development length or splice length. The parameter selected was $A_{tr}f_{yt}/sd_b$. Since $A_{tr}f_{yt}$ represents the force which can be developed at a tie location, it is to be expected that the effectiveness of a tie is inversely proportional to the spacing of the ties and diameter of the bar enclosed. This parameter was selected because it allows considerable simplification for design purposes.

For tests with transverse reinforcement, the value of u_{tr} and $u_{tr}/\sqrt{f_c'}$ was calculated and plotted against $A_{tr}f_{yt}/sd_b$, where u_{tr} represents the difference in bond stress measured (u_t) and that calculated using Eq. (3). The correlation was about the same using u_{tr} or $u_{tr}/\sqrt{f_c'}$. The value of $u_{tr}/\sqrt{f_c'}$ was selected because it permits some design simplifications as will be discussed later. As expected, the greater the transverse restraint relative to bar diameter, the greater the strength or increment of stress over that provided by the concrete cover alone. However, beyond a certain point transverse reinforcement will no longer be effective and an upper limit is needed. Examination of tests with extremely heavy transverse reinforcement indicated that an upper limit of $u_{tr} = 3\sqrt{f_c'}$ was reasonable. Fitting a straight line through the test results led to the following equation:

$$\frac{u_{tr}}{\sqrt{f_c'}} = \frac{1}{500}\left(\frac{A_{tr}f_{yt}}{sd_b}\right) \leq 3 \qquad (5)$$

Some of the data appears to vary considerably from the curve shown in Fig. 6, but u_{tr} is an increment added to the strength contributed by the concrete surrounding the bar and the differences are not as significant when the anchorage strength is computed. Combining Eq. (5) with Eq. (3), the strength of a bar with transverse reinforcement is

$$u'_{cal} = u_{cal} + u_{tr}$$

$$= \left[1.2 + \frac{3C}{d_b} + \frac{50d_b}{l_s} + \frac{A_{tr}f_{yt}}{500sd_b}\right]\sqrt{f_c'} \qquad (6)$$

where u'_{cal} is the strength of bars with transverse reinforcement. For the 27 splice tests considered, the average u_t/u'_{cal} was 1.10, with a standard deviation of 0.05. For 27 development length tests, the average value of u_t/u'_{cal} is 1.03, with a standard deviation of 0.15. Comparison of calculated values using Eq. (6) with measured values indicates generally excellent agreement. It should be noted that in the tests considered transverse reinforcement was present at each bar. Where a number of bars are contained within a single hoop, as shown in Fig. 5, the transverse reinforcement may not be as effective in restraining the splitting at interior bars. Warren[13] tested specimens containing from two to seven bars within a single tie which indicated that the transverse steel was less effective as the number of bars increased. More work is needed to clarify this aspect of behavior.

Other tests with transverse reinforcement

A large number of tests have been conducted by researchers in Europe on the strength of bars confined by transverse reinforcement. Tepfers[4] conducted 29 specimens with the prime variable being the amount of transverse reinforcement. A major study was conducted by Robinson, Zsutty, et al.[19] in which a total of 425 specimens were tested to evaluate the influence of transverse reinforcement on the anchorage capacity of reinforcing steel. A wide range of transverse steel variables was considered, including diameter, spacing, and strength. Concrete strength varied from 1200 to almost 6000 psi (85 to 420 kgf/cm^2). A total of 146 specimens from eight different series in the study were selected to give a representative sample of the study.

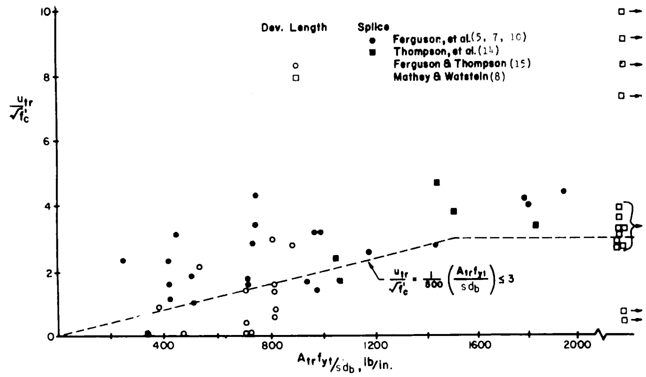

Fig. 6—Effect of transverse reinforcement

Once again only specimens which did not reach yield were included. In many cases the tests were terminated at yield or splitting failures did not develop. A series of tests reported in Reference 18 provides additional data concerning the influence of transverse reinforcement. Each specimen had bars in the top and bottom faces and the results provide data useful for examining the influence of top casting on anchorage strength.

For the tests discussed, Eq. (6) was used to calculate the strength of the specimens and the ratio of u_t/u'_{cal} was determined. Table 1 is a brief summary of the correlation achieved. As can be seen, Eq. (6) provides excellent agreement between calculated and measured anchorage strengths, except for Series B in which it appears that shear was developed between the bars as a result of the loading arrangement, and Series S in which the sample is small (7 tests) and may not be significant.

Effect of top casting and type of aggregate

A major parameter influencing the strength of anchored bars is the position of the bar relative to the height of the concrete lift during casting. Current design specifications define a top cast bar as one in which 12 in. (30 cm) or more of concrete is cast below the bar and require an increase in development or splice length. Ferguson and Thompson[15,16] and Thompson, et al.[14] tested a total of 12 specimens with top cast bars, for which the

TABLE 1—SUMMARY OF TEST CORRELATION

Test program	Number of tests	Average	Standard deviation
Tepfers[4],	29	1.24	0.20
Robinson, Zsutty et al.[19]			
Series D, Y	19	1.10	0.12
Series B	21	0.93	0.14
Series A	38	1.25	0.15
Series R	13	0.98	0.14
Series S	7	0.90	0.16
Series V	19	1.02	0.11
Series W	29	1.14	0.26
CUR[18]	22	1.08	0.11

average ratio of u_t/u_{cal} [Eq. (6)] was 0.88 with a standard deviation of 0.07. For the tests reported in Reference 18, in which each specimen had both top and bottom bars, the average measured u_{top}/u_{bottom} was 0.82 with a standard deviation of 0.12. Until additional research is conducted to evaluate the influence of top casting, the anchorage strength of top cast bars should be taken as about 70-75 percent of the value for bottom cast bars.

It should be noted that only tests on normal weight concrete were considered in this study. Additional research is needed to evaluate the performance of anchored bars in lightweight concrete.

AN APPROACH FOR DESIGN

Modification of empirical equation for design

For design purposes it is desirable to determine the splice or development length rather than average bond stress. Since $u = f_s d_b / 4 l_d$, Eq. (6) can be solved for l_d.

$$l_d = \frac{\left(d_b \dfrac{f_s}{4\sqrt{f_c'}} - 50 \right)}{\left(1.2 + 3 \dfrac{C}{d_b} + \dfrac{A_{tr} f_{yt}}{500 s d_b} \right)} \quad (7)$$

Eq. (7) can be further simplified in the following manner. The term $(f_s/4\sqrt{f_c'} - 50)$ can be rewritten as $(f_s - 200\sqrt{f_c'})/4\sqrt{f_c'}$. Since $f_s - 200\sqrt{f_c'}$ will be fairly insensitive to the concrete strength, it can be conservatively assumed that $(f_s - 200\sqrt{f_c'})$ equals $f_s - 11{,}000$ psi ($f_c' \approx 3000$ psi, 210 kgf/cm^2). For Grade 60 reinforcement the development length is

$$l_d 60 = \frac{d_b (49{,}000)}{4.8\sqrt{f_c'} \left(1 + 2.5 \dfrac{C}{d_b} + \dfrac{A_{tr} f_{yt}}{600 s d_b} \right)}$$

$$= \frac{10{,}200 d_b}{\sqrt{f_c'} \left(1 + 2.5 \dfrac{C}{d_b} + K_{tr} \right)} \quad (8)$$

For Grade 40 and Grade 75 the constant in the numerator is adjusted accordingly.

The current ACI and AASHTO provisions[1,2] are based on substituting $1.25 f_y$ for f_s in the design equations. It is assumed that by using a stress 25 percent greater than yield, ductility requirements will be satisfied. It should be noted that in the current provisions the development length is directly proportional to f_s. Therefore, an increase requiring $1.25 f_y$ led to a 25 percent increase in development length over that required to develop yield. Examination of Eq. (7) shows that a 25 percent increase in f_s will lead to a somewhat smaller increase in l_d. Therefore, it is recommended that a capacity reduction factor ϕ be used in development length calculations rather than an increase in f_s. The capacity reduction factor is intended to account for deviations in material properties, dimensional errors, and, to some extent, the uncertainty involved in the calculation. Based on the data analyzed, a capacity reduction factor $\phi = 0.8$ seems reasonable.

Proposed design recommendations

For deformed bars in tension the development length l_d (in inches) shall be computed as the product of the basic development length of (a) and the applicable modification factor or factors in (b), but l_d shall be not less than 12 in.

(a) The basic development length for Grade 60 reinforcement is

$$\frac{10{,}200 d_b}{\sqrt{f_c'} \left(1 + 2.5 \dfrac{C}{d_b} + K_{tr} \right) \phi}$$

The capacity reduction factor ϕ shall be taken as 0.8; C shall be taken as the lesser of the clear cover over the bar or bars or half the clear spacing between adjacent bars; C/d_b shall not be taken as more than 2.5 and the transverse reinforcement term

$$K_{tr} = \frac{A_{tr} f_{yt}}{600 s d_b} \leqq 2.5$$

where A_{tr} is the area of transverse reinforcement in square inches normal to C. Where several bars are confined by a single hoop A_{tr} shall be taken as the area of transverse reinforcement divided by the number of bars or splices contained therein.

(b) The basic development length shall be multiplied by the applicable factor or factors for

Grade 40 reinforcement	0.6
Grade 75 reinforcement	1.3
Top reinforcement (12 in. of concrete below bar)	1.3
Wide spacing such that $C_s/(C_b d_b)$ is greater than 3	0.9
Wide spacing such that $C_s/(C_b d_b)$ is greater than 6	0.7
Reinforcement in a flexural member in excess of that required	(A_s required) / (A_s provided)

The length of a tension lap splice l_s shall be computed as for development length l_d with the appropriate cover C determined from a consideration of the clear cover and the clear spacing between the splices.

For lap splices of #14 and #18 bars, minimum transverse reinforcement shall be provided such that $A_{tr} f_{yt} / s d_b \geqq 600$ psi.

Comparison of proposed recommendations with ACI 318-71

The proposed design equation represents a considerable advance over current methods because it takes into account the effect of clear cover, spacing, and transverse reinforcement. By using the same equation for both splice and development lengths, the number of different design conditions is reduced substantially.

Fig. 7 shows a comparison of required lengths for Grade 60 (4220 kgf/cm²) steel with $f_c' = 3000$ psi (210 kgf/cm²). Development lengths proposed for bars with 1½ in. (38 mm) cover which are typical in many structural applications will be 10 to 40 percent greater than those called for in current specifications. Tests conducted by Warren[13] clearly show the adverse effect of small cover or spacing on the strength of anchored bars and confirm the importance of cover as a design parameter. Note that for current provisions l_d remains the same regardless of cover or transverse reinforcement. With increase in cover to 3 in. (76 mm) or addition of maximum effective transverse reinforcement, the required length for #8 and smaller bars is about the same as currently specified. However, for bars larger than #8, the required length is reduced over current specifications if the cover is increased or transverse steel is added (25 percent for #11 bars). Advantage may also be taken of wide spacing which may further reduce the development length required. For slabs or walls with ¾ in. (19 mm) cover, the development or splice length would be increased over current specifications.

The differences in required anchorage lengths shown in Fig. 7 may be traced to the data on which current provisions are based. The equation for determining development lengths was based primarily on tests of large single bars in wide beams by Ferguson and Thompson[15] and the bond beams tested by Mathey and Watstein[8] which had extremely heavy transverse reinforcement over the development length. Consequently, higher average bond stresses were obtained which led to shorter development lengths.

The design proposals are also compared with current provisions in Fig. 7 for Class C splices—all bars spliced in a region of maximum moment and spaced closer than 6 in. (15 cm) on centers—the most severe splicing condition. It is seen from Fig. 7 that ACI provisions require a greater splice length than proposed for all bar sizes ($f_y = 60$ ksi, $f_c' = 3000$ psi). For a clear cover of 1½ in. (38 mm) on sides or bottom, the proposed provisions represent a reduction in lap lengths of about 10 percent for #6, and about 20 percent for #11 bars.

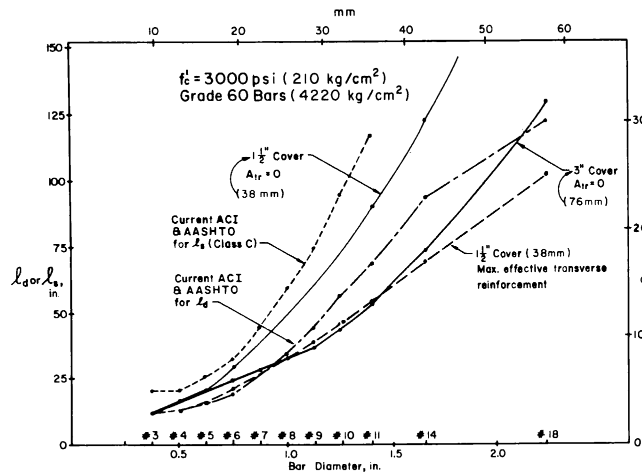

Fig. 7—Comparison of l_d or l_s by current and proposed design methods

With larger clear cover and with transverse reinforcement, the reductions are even more pronounced. If the maximum effective transverse steel is provided, the lap lengths will be reduced by about 20 percent for #6, and about 50 percent for #11 bars. On the basis of the data considered, there does not appear to be sufficient reason to prohibit lap splices in #14 and #18 bars. However, the splice lengths will be very large unless transverse steel is provided or the cover is increased and the proposed provisions suggest lap splices for large bars only if some transverse steel is provided.

SUMMARY AND CONCLUSIONS

The basic design equation developed in this study has been verified through successful application to tests from various sources. The development and splice lengths were found to be identical and could be expressed in terms of steel stress, concrete strength, bar diameter, minimum side or bottom cover, and transverse reinforcement—factors which have been shown by tests to affect the strength of anchored bars.

Comparison of current provisions for development length with the proposed design recommendations shows that for minimum cover current provisions are unconservative. However, with increase in cover or addition of transverse reinforcement considerable reduction in development length can be realized by using the proposed provisions. For lap splices in a region of high stress, the proposed provisions lead to considerably shorter splice lengths over those now used and permit splices of #14 and #18 bars, provided some transverse reinforcement is specified.

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NOTATION

- a_b = area of bar, sq in.
- A_{tr} = area of transverse reinforcement normal to the plane of splitting through the anchored bars, sq in.
- C = the smaller of C_b or C_s, in.
- C_b = clear bottom cover to main reinforcement, in.
- C_s = half clear spacing between bars or splices or half available concrete width per bar or splice resisting splitting in the failure plane, in.
- d_b = diameter of main reinforcement, in.
- f_c' = concrete cylinder strength, psi
- f_s = maximum stress in bar, psi
- f_{yt} = yield strength of transverse reinforcement, psi
- k = ratio of steel stresses
- K_{tr} = an index of the transverse reinforcement provided along the anchored bar, $A_{tr}f_{yt}/600\, sd_b$
- l_d = development length, in.
- l_s = length of lap splice, in.
- s = spacing of transverse reinforcement, center-to-center, in.
- S' = clear lateral spacing between bars, in.
- u = average bond stress, psi
- u_{cal} = calculated average bond stress—no transverse reinforcement, psi
- u'_{cal} = calculated average bond stress—with transverse reinforcement, psi
- u_t = average bond stress obtained in tests, psi
- u_{tr} = portion of strength contributed by transverse reinforcement, $u_t - u_{cal}$, psi

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