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Bond Stress—The State of the Art

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The nature of bond failure is discussed and the influence of splitting is emphasized. The large bond stresses adjacent to any crack and the complications caused by this "out-and-in" bond are emphasized. Factors influencing splitting and the weak planes in splitting are related. The importance of beam width or bar spacing on bond resistance is developed. The concepts of end anchorage, flexural bond, and anchorage or development bond are contrasted.

Present knowledge of bond behavior and the absolute value of bond resistance are reviewed, first in terms of splices and special members, and then in the negative moment region of a beam. Some of the data justifying the 1963 Building Code provisions are presented. The value of end anchorage is indicated and the importance of development bond or development length is emphasized.

The effect of top bar position and the influence of lightweight concrete are briefly discussed. Finally, weak spots in existing knowledge and areas needing further investigation are mentioned.

Key words: ACI committee report; anchorage; bond; bond stress; building codes; lightweight aggregate concrete; reinforced concrete; reinforcement; steel.

■ BOND STRESS IS THE NAME ASSIGNED to the unit shear force acting parallel to the bar on the interface between bar and concrete. In transferring from concrete to bar, this shear stress (bond stress) modifies the steel stress in the bar, either increasing or decreasing it. It has been customary to define bond stress as stress per unit area of bar surface, using the nominal surface of the deformed bar (which ignores the extra surface created by the lugs and ribs). Bond stress could also be measured by the rate of change of *steel* stress in the bar. Whether one chooses to think in these terms or not, there can be no bond stress unless the bar stress changes, and there can be no change in bar stress without bond stress.

In the strict sense there is one exception to this necessary linkage between bond and steel stress. It is possible to load a tension hanger through an attachment welded to the steel or to hold such a hanger by a nut and washer supported in turn on a concrete surface. Similarly, a simple beam can act as a two-hinged arch if the bars are left unbonded over their length but are held at their ends by welded steel plates. For the purpose of this discussion, such devices will be called end anchorages. End anchorages may take many forms and the concept will be discussed later.

Bond stress is less apt to be critical in design today than it was 30 years ago. Deformed bars have provided an extra element of strength and safety. However, bond is probably less thoroughly understood today than it was in the days of plain bars. The deformed bars have created new problems even as they solved old ones. The advent of high strength steels and larger bar sizes has required that engineers re-examine their basic knowledge and reappraise practices which were safe with the 62.4-kip yield capacity of a #11 bar of intermediate grade steel. The possible five-fold extrapolation to a #18 bar of ASTM A 431 steel (300-kip yield capacity) requires a re-examination of the elementary ideas of bond.

This report reviews the present state of knowledge as regards bond stress on deformed bars (ASTM A 305 and A 408) and attempts to remove some of the confusion often existing because of changing nomenclature and concepts. Wire fabric, deformed wire, and reinforcement for prestressed concrete are not included in this report.

THE NATURE OF BOND FAILURE

When plain bars without surface deformations were used, bond was often thought of as chemical adhesion between concrete paste and bar surface. However, even a low bar stress causes slip sufficient to break the adhesion immediately adjacent to a crack in the concrete. Over the slipping length only a friction drag is left and the highest adhesive bond stress cannot act except close to this slipping portion. Bond resistance is thus due to a maximum or ultimate bond stress over a short length where adhesion is about to fail with a much lower friction drag over the length where adhesion has failed. Quite possibly zero bond exists over some portion of the length not stressed by the pull. With the usual concrete and hot rolled plain steel bars, shrinkage makes the friction drag a substantial part of the total. With a very smooth bar, such as a cold rolled bar, the friction is lower. A hot rolled plain bar may pull loose either by longitudinal splitting of the concrete or by pulling out, but a cold rolled bar will pull loose and leave a slick bore or hole.

Deformed bars were designed to change this behavior pattern. Adhesion and friction still assist, but the chief reliance has been changed to bearing of lugs on concrete and to shear strength of concrete sections between lugs. With deformed bars a pullout specimen* nearly always fails by splitting, the concrete splitting into two or three segments rather than failing by crushing against the lugs or by shearing on the cylindrical surface which the lugs tend to strip out. Splitting results from the wedging action of the lugs against the concrete. Occasionally, most often with small bars or top cast bars, or with large cover over the bars, the lugs will shear the concrete and pull out without splitting the concrete. More recently a preliminary report of larger bars in lightweight concrete pullout specimens[1] has indicated as many as a third of the bars, especially top cast bars, shearing out rather than splitting the concrete. Nearly the same was true for the ordinary concrete used for comparisons.

Strictly interpreted, splitting is not the same thing as bond failure, by the traditional concept. It would be desirable to set up a separate criterion against splitting, as a tension phenomenon, like splitting of a frozen water pipe. But knowledge has not progressed far enough to provide the necessary data. For the present, splitting must remain lumped together with other aspects of bond. Progressive splitting is usually the first evidence of bond distress and, except as just noted, it always accompanies final collapse in bond, apparently as a primary cause. Hence splitting is considered here as a normal part of bond resistance. It should also be noted that a split on *one* face of a concrete member does not, by itself, represent complete failure; significant bond resistance remains. When stirrups or spirals are present, this remainder may be larger and will persist even after two or more cracks form.

Since bond stress distribution cannot be established on the basis of the present knowledge of strains, experimental studies for each particular bond situation are necessary to predict numerical values accompanying failure. Only a few cases have been evaluated to date, and some of these have been more by sampling than by systematic exploration. Knowledge of the actual splitting forces developed by deformed bars and the resistance of members to splitting are needed for a more fundamental study.

THE PULLOUT TEST

In the ordinary pullout test,[2] the bar, either plain or deformed, is initially embedded in a cylinder or prism. While the concrete is held by the reaction pressure on one end, the bar is pulled out from the same end. Since the bar is in tension and the concrete in compression, differential strains force a relative slip at very low steel stresses,

*Pullout specimens are described in a later section.

usually by the time the bar carries a stress of 2000 to 3000 psi. This slip represents local loss of adhesion. Because a very high bond stress can be resisted over a short length of any type of bar, such slip at small loads is localized near the loaded end, some adhesive bond exists slightly beyond the slip zone, and most of the bar is entirely unstressed by the tension pull. As the pull is increased the length which develops slip is increased and the length which is unstressed is correspondingly reduced. The nominal bond stress u for a specimen of length L is simply calculated from the total pull P divided by the total bar surface $L\Sigma o$ within the concrete ($u = P/L\Sigma o$). This nominal bond stress is always an average of adhesion over a short length, a reduced value over the portion where the slip has occurred, and, certainly at lower loads, a zero stress over a portion of the length. The shorter the specimen the more nearly the average bond stress approaches the ultimate value in adhesion; the longer the specimen, the lower is the average bond stress which can be obtained.

In the particular case of deformed bars, the slip near the loaded end gradually brings the lugs into bearing and thus raises the average bond stress, but, because of splitting when ordinary cover is used, possibly not up to the level which a short length can develop by adhesion. The nearly uniform slope in Fig. 1 (from Reference 2) of the uppermost steel stress curve in a pullout specimen indicates a nearly uniform ultimate bond stress. The deformed bar specimen, especially a large bar specimen,[3] eventually fails by splitting unless the cover over the bar is more than 3 or 4 in. The average bond stress obtainable decreases sharply with increase in length.

The pullout test has been discussed in some detail because:

1. It gives a reasonable measure of the necessary anchorage length of a bar when it is embedded in a pier or an inactive mass of concrete.

2. It represents clearly the basic idea of *anchorage,* namely, that a certain length of bar is needed beyond a point of maximum steel stress to keep the bar from pulling out, even if that anchorage length is not in a region of shear or moment.

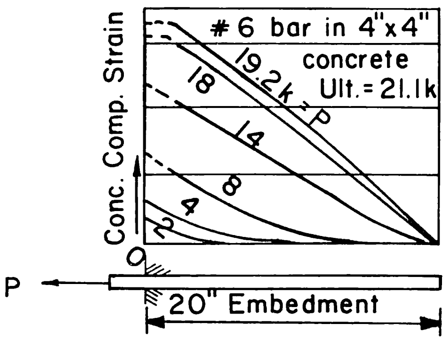


Fig. 1—Compression strain in concrete measures change in load transferred from bar. Slope is proportional to bond stress. Bar is #6, deformed but not A305. (Reference 2)

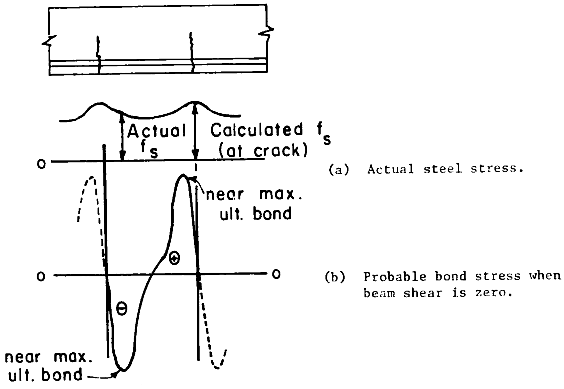


Fig. 2—Constant moment region in beam. (After Reference 4)

3. It vaguely represents what happens adjacent to any concrete crack, where a bar always carries more tension than exists in nearby sections.

4. The bar slip at the loaded face is often considered to be half the beam crack width which would result at the same steel stress. (This crack estimate is actually too large. In a beam the tension strain in the concrete subtracts from the tension strain in the steel to establish crack width at the bar as a difference of the two. In a pullout specimen the compression strain in the concrete adds to the strain differential and increases the necessary slip at any given steel stress level.)

5. The effect of reduced cover over the bar can be simulated by adaptations of the pullout test which place the bar off center and nearer one face.

The weakness of the simple pullout test as a standard is that the concrete in compression eliminates transverse tension cracking. In the next section flexural tension cracking is shown to have an adverse effect on the average bond stress which can be developed. Friction on a loaded end bearing also retards splitting locally, but usually does not totally eliminate it.

BOND STRESS IN A CRACKED BEAM

Even where shear is zero and moment is constant, large local bond stresses exist adjacent to each flexural crack. At the crack most of the tension is carried by the bars and the steel stress is maximum. Between cracks the concrete carries tension and the steel stress drops off in a compensating manner, as in Fig. 2a.[4,5] Thus bond must take stress out of the steel adjacent to a crack and put it back in the steel just before the next crack is reached, as in Fig. 2b. The rate of change of steel stress is a function of the bar size (area) relative to the area of the concrete. As in the pullout test near the loaded end, bond stresses are very high adjacent to the cracks; high enough with large bars at high stresses to create significant splitting. Note the extensive splitting in

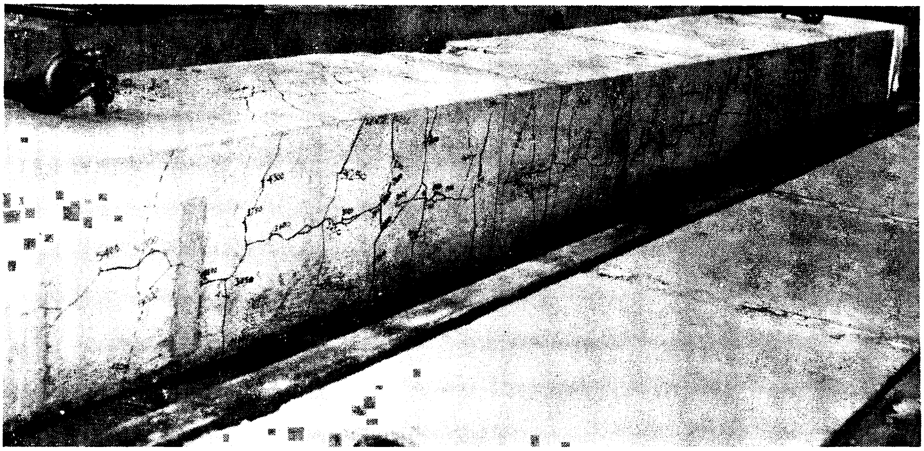
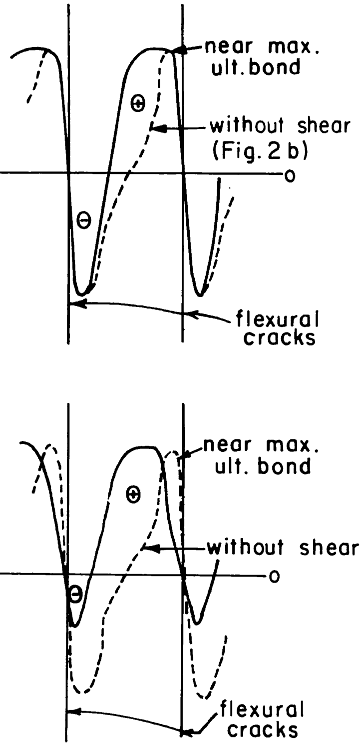


Fig. 3—Longitudinal cracking over constant moment length (middle third) with single #18S bar of A431 steel

a constant moment region when a high stress is carried by a #18S bar (Fig. 3)[6] and the more limited but still obvious splitting in Fig. 4 where two #11 high strength bars were used. Between adjacent cracks the direction of the bond stress reverses, as indicated in Fig. 2b. In such a constant moment region the "out-and-in" bond stresses shown result in no net change of steel stress from one crack to the next.

Where moment is changing from one crack to the next, the steel stress must also change. Then there must be a net bond stress over and above the "out-and-in" bond stresses discussed above. Just how this occurs is not yet established, but, since ultimate bond stresses already exist near each crack (under constant moment), these stresses cannot increase. Either a greater length of bar must carry the high stress as in Fig. 5a, or near the adjacent crack the reverse kind of bond must be reduced as in Fig. 5b. The details of this stress distribution are difficult to measure and are at present unknown. If some splitting starts, possibly the stress near the right-hand crack decreases even more than

Fig. 4—With two #11 bars at approximately same stress as in Fig. 3, longitudinal cracking is much reduced, although still obvious



(a)

(b)

Fig. 5—Bond stress in beam carrying a positive shear. Two possibilities are shown

shown by the alternate in Fig. 5b. It seems reasonable that considerable "out-and-in" bond remains, which is obviously not very efficient. Tension cracking thus must lower the net bond efficiency, in comparison with a pullout specimen.

The redistribution of stresses, which occurs when a diagonal tension crack opens, requires both higher steel stress and more bond stress on the side of the crack nearer the point of inflection or the simple beam support. In addition the dowel action of the bar puts vertical tension into the concrete which adds to the splitting caused by high local bond stress.

The bond capacity of compression bars will normally be greater than the usable bond capacity of tension bars because compression bars do not cross open cracks. In addition, compression bars may have some resistance developed by end bearing, if the bars stop within the concrete. Thus bond is not so apt to be critical on compression bars.

FACTORS IMPORTANT IN SPLITTING RESISTANCE

Obviously, clear cover over a reinforcing bar will be significant in connection with splitting resistance. Thin cover can be easily split; very thick cover can greatly delay splitting if bars are not too closely

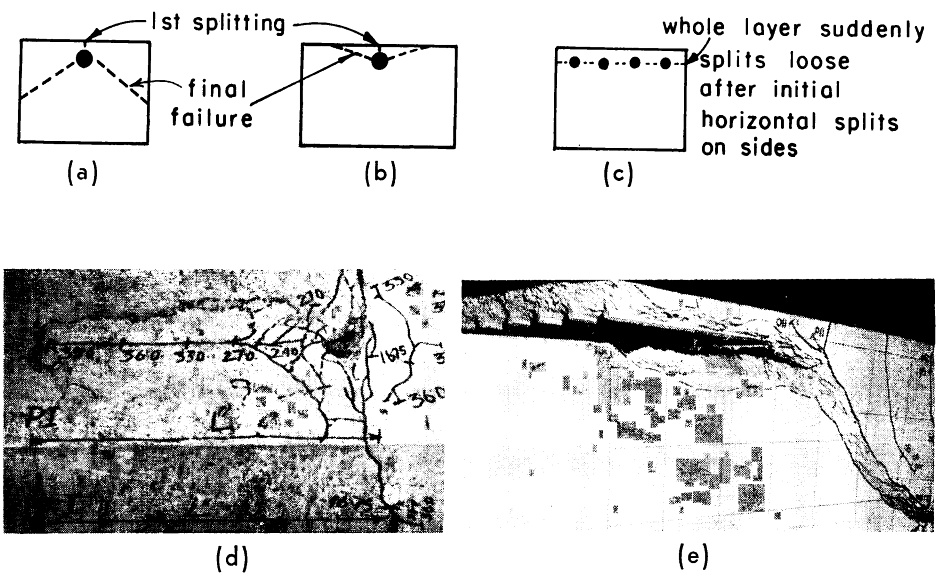


Fig. 6—Splitting cracks and ultimate failure. (a) Typical. (b) and (d) Very wide beam. (c) and (e) With closely spaced bars

spaced laterally. While it is not economical to increase bond strength by varying cover, the designer should recognize that bond strength in a slab with 0.75 in. of cover is lower than in a beam with 1.5 to 2 in. of cover, unless bar spacing is so close as to lead to the horizontal splitting failure of Fig. 6c and 6e.

A single tension bar will split a wide* beam as in Fig. 6a with final failure along the dotted lines as shown in Fig. 15b. In a very wide beam, the failure will be as in Fig. 6b and 6d. In a narrow beam, or with a number of closely spaced bars in a wide beam, the failure will be as in Fig. 6c and 6e. As splitting is a tension failure it is not surprising that ultimate bond stress seems to vary approximately as the square root† of f_c' as does the modulus of rupture. Where splitting controls the available bond resistance, it would also be logical that the splitting force per unit of length would be more significant than the bar diameter. The ACI Building Code now specifies bond stresses which vary as the reciprocal of bar diameter. This leads to the same maximum change in bar tension ΔT in a unit length regardless of the bar diameter. This variable was established by test values, but it is probably significant that one only needs to assume that splitting force is proportional to ΔT to link bond stress to splitting force.

*The term "wide" has not yet been adequately bounded by tests, but a #3 bar in a 6 in. width, a #7 bar in a 16 or 18 in. width, or a #11 bar in a 24 in. width seem to follow this pattern. See also discussion of width relating to Fig. 15 and 16.

†Dr. James Chinn has informally reported better correlation with $(f_c')^{0.7}$.

Splitting seems to start at flexure cracks, being most evident where steel stress is largest. Thus splitting is a progressive phenomenon, working its way gradually along the length of embedment. Splitting may not be continuous from flexure crack to flexure crack. A splitting crack often stops short of the next flexure crack in a bond test beam; in fact the opening of a new flexural crack usually occurs beyond the end of a splitting crack, and added splitting then develops from the new flexure crack (Fig. 15a). Normally the splitting will eventually close the gap. Splitting can develop over 60 to 75 percent of the bar length without loss of *average* bond strength. Apparently splitting is one means by which some of the unevenness in bond stress distribution may be smoothed out. However, the final failure is sudden (in the absence of stirrups) as the split suddenly runs through to the end of the bar.

Possibly because of a changing splitting pattern, width of beam influences the bond resistance. A single #11 bar in an 18 in. beam width will develop higher bond stress than in a 16 in. width, and less than in a 24 in. width. The resistance of closely spaced bars creating a plane of weakness (as in Fig. 6c and 6e) is substantially lower. This close spacing effect is one of the more serious factors still needing further investigation. Only in the case of lapped splices has it been reflected in the ACI Building Code.

When bars are cut off in a tension zone, a flexural crack forms prematurely* at the discontinuity. The sudden resulting change in bar stress produces large (maximum or ultimate) bond stresses locally and causes splitting, often on both sides of the flexure crack. This is one reason a splice must be designed for a lowered bond stress under the ACI Code.

Stirrups slow the propagation of a splitting crack and keep this crack width small. If surplus stirrups are present, ultimate bond stress is increased.† If, however, stirrups are simply adequate (by the usual standards) for the expected shear, ultimate bond strength is little influenced, although toughness is considerably improved. A spiral over a bar materially improves bond resistance, although this is an unusual usage and one inadequately explored experimentally.

NOMENCLATURE

End anchorage has already been defined. End anchorage may be complete, as in the case of the hanger mentioned earlier, or it may be used for only a portion of the strength of the bar, as when a bar is extended beyond a point of inflection to make up some deficiency of bond inside the point of inflection.

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*Shear strength is also lowered by a diagonal crack which tends to develop from this flexure crack.

†In one test, an increase in the stirrup ratio $r = A_v/bs$ from 0.0015 to 0.0036 raised the ultimate bond stress by 38 percent.

Flexural bond is the bond defined by the equation $u = V/(\Sigma ojd)$, the bond stress which has long been used. It is a measure of the local bond stress necessary to produce the local ΔT bar pull demanded by flexure. Although it sounds like the real stress, flexural cracking reduces it to the status of a local average stress. The basic assumptions which led to this formula are violated whenever bars are cut off or bent in a tension zone.

Anchorage bond and *development bond* are identical concepts. An anchorage length is the length necessary to take a given stress out of a bar; development length is the length necessary to put a given stress into a bar. Whether stress is going in or out is a simple vector concept dependent on whether the interval is taken from left to right or right to left. Hence, either adjective may be used. Some feel that anchorage might better be used where the length is not much influenced by local external moment or shear, with development used where flexural bond is present. In a constant shear region in the simplest case development and flexural bond are identical. However, it might be noted that an extension of the bar beyond a point where the moment is zero adds end anchorage to the development length and lowers the average development bond stress; it has no influence on flexural bond, which is determined locally by the shear V or change in moment ΔM.

The feeling is growing that the most significant bond calculation is that of development or anchorage bond, which is only an average bond stress. It is impossible now to state the exact distribution of bond stress. It is certain that even simple tests reveal average stresses which are combinations of ultimate bond with small values (possibly even negative bond stresses). Practice is forced to depend on test values, preferably tests under conditions as nearly practical as possible. Beam tests are now preferred in this country, although special pullout specimens to simulate beam action are occasionally used here and quite often abroad.

EXPERIMENTAL VALIDATION

Since the real bond stresses are not at present susceptible to theoretical analysis, it is necessary to understand where experimental evidence has verified the possible average allowable bond or development length

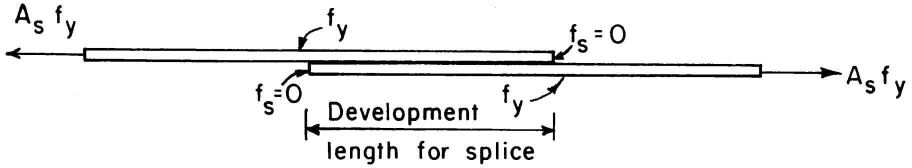

Fig. 7—Discontinuity of splice causes extra wide flexure cracks at each bar cutoff

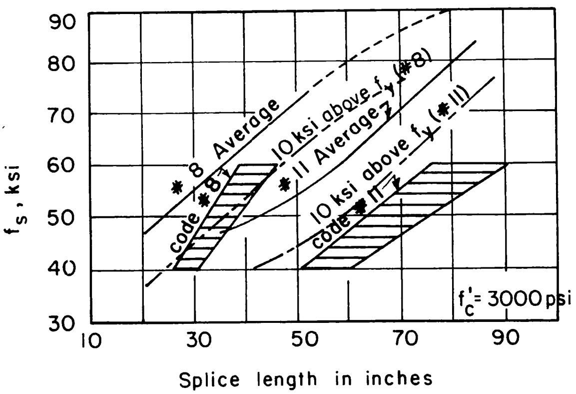

Fig. 8—Splice lengths specified (shaded) and observed strengths

and where gaps in our knowledge exist. A few specialized cases are discussed first because these tend to be more clearly defined as to surrounding conditions.

TENSION SPLICE TESTS

A lapped tension splice in a constant moment region must furnish a development length for each bar, as in Fig. 7. Tests of such splices of #8 and #11 bars of high strength steel (A 432) have been reported[7] which give the results shown in Fig. 8. The shaded areas show the ACI Code length requirements for wide lateral spacing of splices at the left boundary and for more closely spaced splices at the right boundary. The dot-dash line shows the splice length required from test results to attain a steel stress 10 ksi in excess of the nominal yield strength. This degree of excess stress capacity is desirable to postpone a brittle type of failure. For both #8 and #11 bar splices the actual lateral spacing of the tested splices was about 10*D*, which is less than the 12*D* spacing specified for the left boundary length. For the #8 bars, the Code specification is conservative at 40 ksi, about right at 50 ksi, and a trifle less safe at 60 ksi. It is obvious that the present Code plan of using 75 percent of the usual bond stress in designing splices should not be extrapolated for #8 bars to a 75 ksi steel stress. On the other hand, the Code #11 bar splices seem to be somewhat on the conservative side and probably would be safe with 75 ksi steel. The present AASHO code would call for a lap of only 17*D* for an f_y of 40 ksi or 50 ksi, which appears too much on the low side.*

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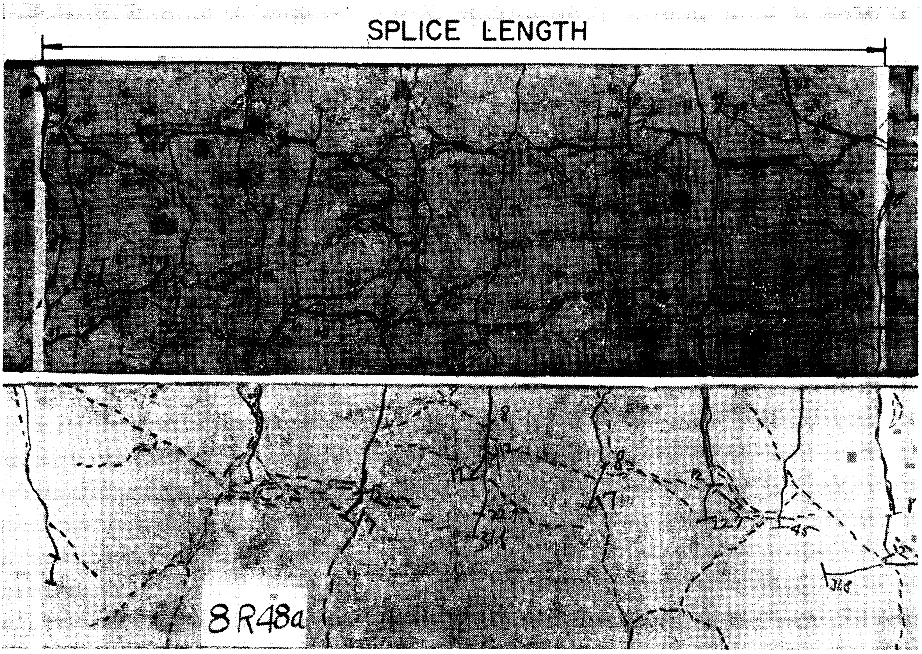
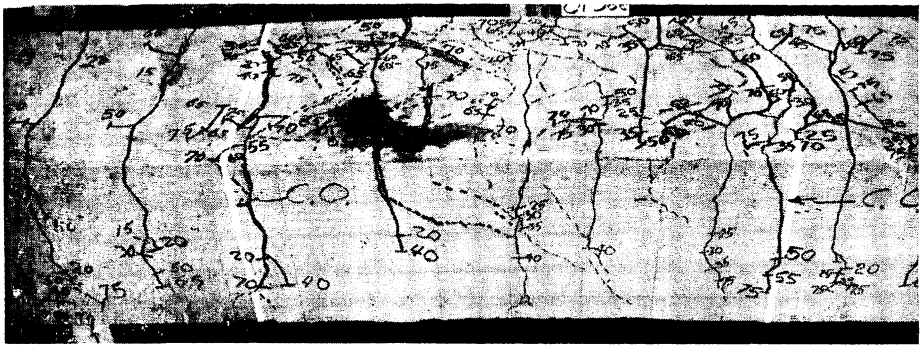
*Verified most recently at the University of Oklahoma.

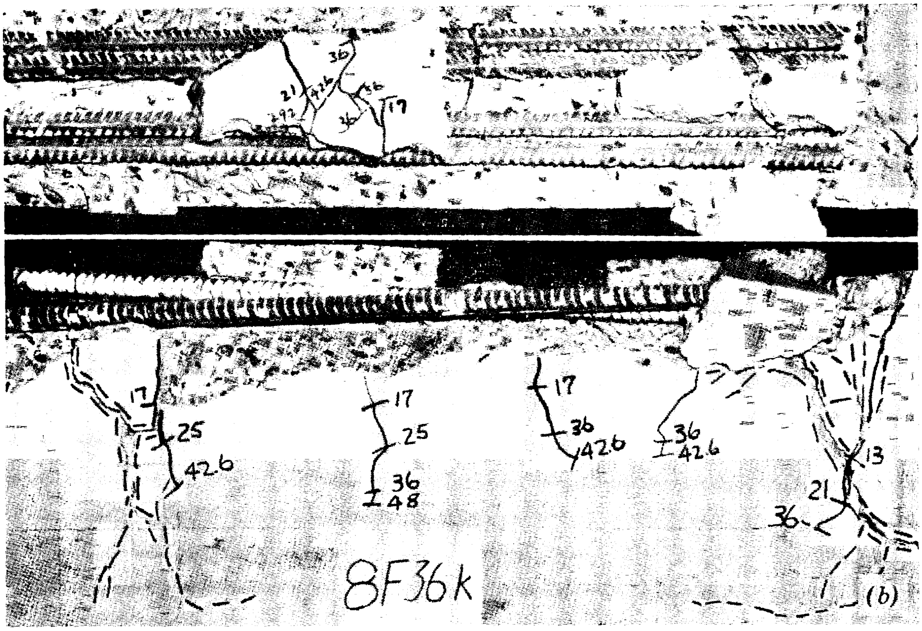


(a)



(b)

Fig. 9—Double splices at failure. Splices end at white lines. Dotted lines mark cracks occurring at last load increment. (a) #8 bars, 48 diameter lap, no stirrups. Tension face and side view. (b) #8 bars, 36 diameter lap, with stirrups



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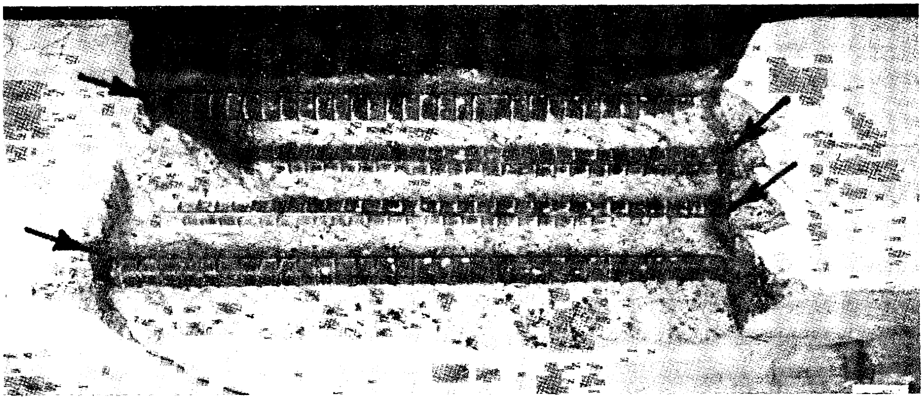


(b)

Fig. 10—Splice failures. (a) Violent failure of #8 bars lapped 36 diameters. (b) Shorter splice with cracked concrete removed to show splitting surface. Arrows mark end slips of bars

Although fewer tests have been run with smaller bars, the Code procedures were originally based on data* including limited tests on smaller sized bars and splices in narrow beams. Further documentation for small bars would be desirable. Tests on bar splices within a varying moment region† have been conducted which seem to show these splices require slightly less length, possibly because one end of the splice is then at a lower bar stress.

Tests on splices in constant moment regions always show a premature flexural crack at each end of the splice. Splitting starts from each end crack, moving towards the middle as shown by the numbers which mark the extent of cracks at the load shown in Fig. 9a. Flexure cracks form between the end cracks and splitting also develops from these.[7] Failure (Fig. 10) then occurs with a sudden final splitting over the remaining 20 to 40 percent of the splice length, even sometimes 50 percent for the very long splices required for 75 ksi. Stirrups over splices slow up splitting but do not prevent it (Fig. 9b). They increase splice strength considerably, in the order of 20 to 50 percent, and also cause a less violent failure, holding the concrete together which would otherwise break off, as in Fig. 10a.

It appears that the usable bond in splices is reduced by three factors:

1. The discontinuity crack at the splice ends leads to splitting from both ends.
2. Added flexural cracks across long splices create new splitting opportunities.
3. When the lateral spacing of splices is close, the splitting on a horizontal plane is resisted by less concrete, as in Fig. 6c and 6e.

Tension splices in *spiral* columns could probably be reduced in length if there were available test data. The spiral should considerably reduce the ill effect of longitudinal splitting over the spliced bar.

COMPRESSION LAPPED SPLICES

In contrasting compression splices with tension splices, one must note that in a compression splice the bearing against the end of the bar strengthens this splice while in a tension splice a premature flexural crack opens at the end of the bar, which is a weakening effect. Research at the PCA laboratory[8] on #8 high strength bars spliced in columns concentrically loaded led to the results shown in Fig. 11. The data are for concrete of about 3500 psi cylinder strength. Spiral columns showed that the Code specified splice lengths (Section 805(c)1) of 20 bar diameters for 50 ksi steel, 24 diameters for 60 ksi, and 30 diameters for 75 ksi are ample. For tied columns this specification tends to be on the skimpy side, although reasonably satisfactory for 50 ksi steel. The stress developed with an end lap of zero indicates

*Unpublished data from the University of Colorado.

†A study by C. P. Siess and others at the University of Illinois on tension splices in columns. A report on this work is to be published in the near future.

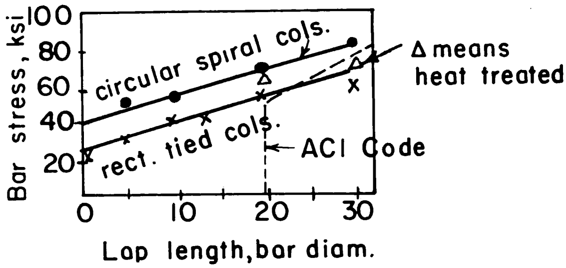


Fig. 11—Compression splice strengths in columns. (Reference 8)

the importance of end bearing on the bar. The chief difference between splices in tied and spiral columns seems to lie in the higher end bearing in the latter. In fact the slope of the lines in Fig. 11 seems to show an average bond stress, aside from end bearing, of only some 350 psi, an indication that bond does not work efficiently in cooperation with the end bearing.

Confirming data on larger bars, such as #11, #14S, and #18S, would be desirable.

BOND IN BRACKETS, SHORT CANTILEVERS, AND DEEP BEAMS

Brackets, short cantilevers, and deep beams involve a totally different bond situation. Although the maximum steel stress at the section of maximum moment can be calculated quite simply and safely as for any cracked beam, the usual development length beyond this maximum moment point is not at all adequate. Deep beam theory (elastic uncracked section) and cracked member behavior both show that where the moment approaches zero (at the load in cantilevers

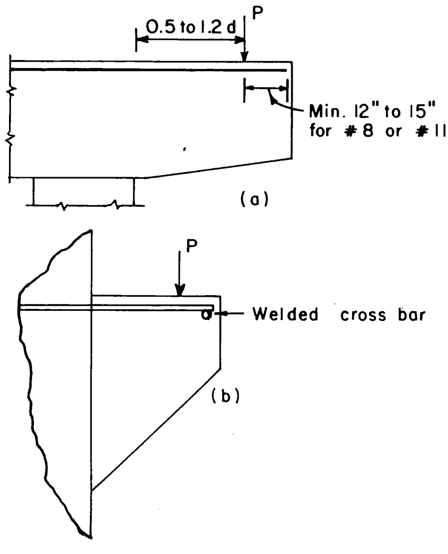


Fig. 12—Anchorage for short cantilevers and brackets. (References 9 and 10)

and at the reaction in deep beams) the steel stress actually remains quite large. The reinforcing must be end anchored beyond this point sufficiently to develop this stress.

For a load resting on top of a cantilever, some tests[9] have shown that a 12 to 15 in. extension of a #8 or #11 bar beyond the load will provide the necessary end anchorage (Fig. 12a). Where this length is unavailable, other tests[10] have shown that straight top bars should be run straight as far as possible and can best be anchored by a crossbar of the same diameter welded to the underside of the main steel as in Fig. 12b.

For a beam with span not over four times its depth, inclined cracks cause the member to act almost like a tied arch, with full yield strength of the steel needed at the face of support. End anchorage is necessary beyond this point, either by bar extension or by a welded end anchor.

TENSION STEEL CONTINUOUS TO POINT OF ZERO MOMENT

This section is restricted to bars continuous from the point of zero stress to their point of maximum stress; that is, no bars are cut off at intermediate points. When part of the tension steel in a beam is cut off, shear strength is lowered and complications in bond stress exist similar to those at one end of a splice. Section 918(c) of ACI 318-63 attempts to discourage the stopping of tension bars in a tension zone. The requirements were set on the basis of problems encountered with the University of Texas beams of Fig. 14. Although in a few more recent tests these requirements have proved to be adequate, there is a minimum of data to substantiate the exact provisions. Some further exploration is under way at the University of Texas and at the Portland Cement Association.

Since splitting is a common failure mode in the case of bond, the reaction pressure on the bar of a simple span beam makes this a poor general test specimen unless it is modified to remove the pressure from the bar. Untrauer and Henry have shown[11] that lateral pressure

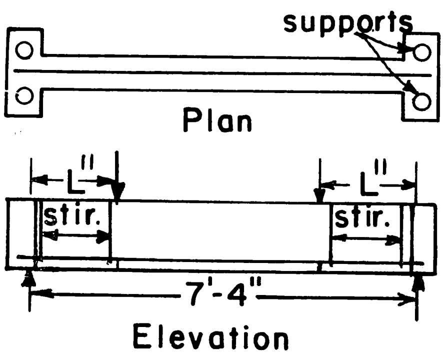


Fig. 13—Bureau of Standards beam with reactions away from bar tested. (References 12 and 13)

on a specimen containing a bar does increase bond resistance substantially.

Two types of beams without this reaction restraint are shown in Fig. 13 and 14. The National Bureau of Standards beam[12,13] provides two reactions at each end, both away from the tested bar. The University of Texas beam[6,14] uses the negative moment length of a semi-continuous type beam for the total bar. In general, results from the two specimen types agree as closely as could be expected, considering the fact that the first uses heavy stirrups and the other no stirrups or very light ones.

Typical failures with the University of Texas beam, recorded in Fig. 15, show the longitudinal splitting which occurs. The dotted lines trace the final increment of cracking. These tests showed that bond strength was a function of cover over the bars, an extra inch of cover increasing the bond stress resistance from 60 to 100 psi. As the concrete strength was increased, the bond resistance changed approximately as the square root of f_c'. (There is some evidence that 2000 psi concrete is a *little* better than would be predicted from 3000 psi concrete strengths and the ratio $\sqrt{2000/3000}$.)

Only wide beams developed the full bond stress shown in Fig. 16. The developed bond stress was reduced as the test beam width for a #7 bar was reduced from 18 in. width to 14 in., with an increasing tendency for diagonal tension cracking to form and complicate the stresses. With a 12 in. width the failure changed to diagonal tension. It might be postulated from this behavior and the calculated stresses that localized bond stresses around an isolated bar result in local shear concentrations and some loss of potential diagonal tension strength. Even a 24 in. width did not give the full bond stress possible for a high strength (75 ksi) #11 bar unless stirrups were used.

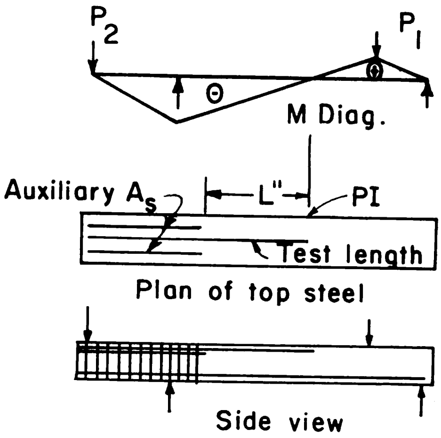


Fig. 14—University of Texas beam with test bar in negative moment region. (Reference 14)

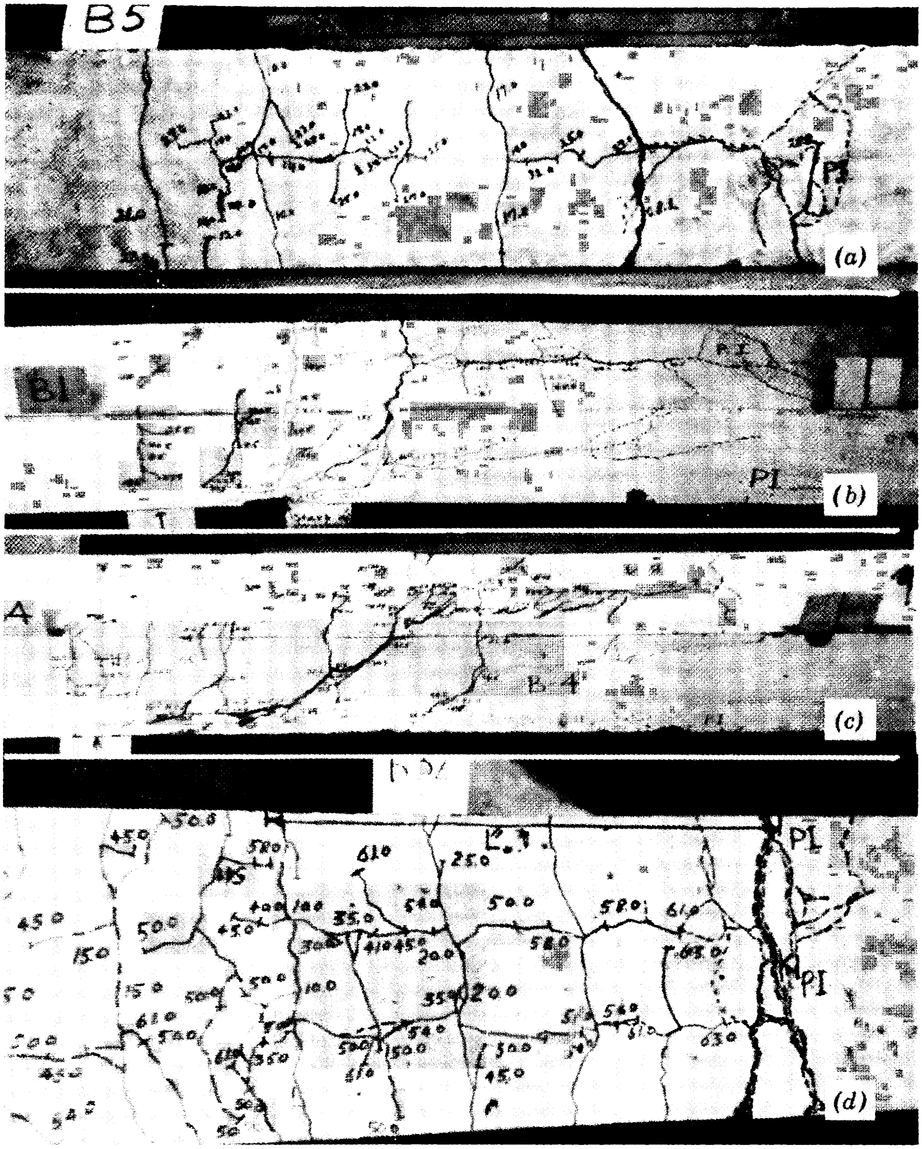
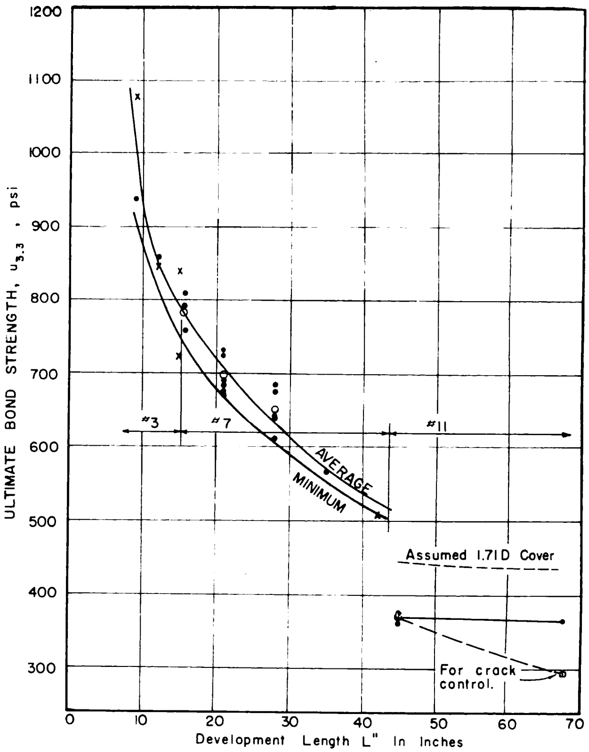


Fig. 15—Bond splitting photographs of beams tested as shown in Fig. 14. (a) Splitting is directly over bar and runs lengthwise. The cross cracks are flexural cracks. (b) Failure chiefly from bond splitting. (c) Diagonal tension (inclined side crack) combined with longitudinal splitting to produce failure. (d) Splitting over two bars in beam with heavy stirrups

Fig. 16 — Ultimate bond strength for b = 18 in., f_c' = 3300 psi, and 1.5 in. cover



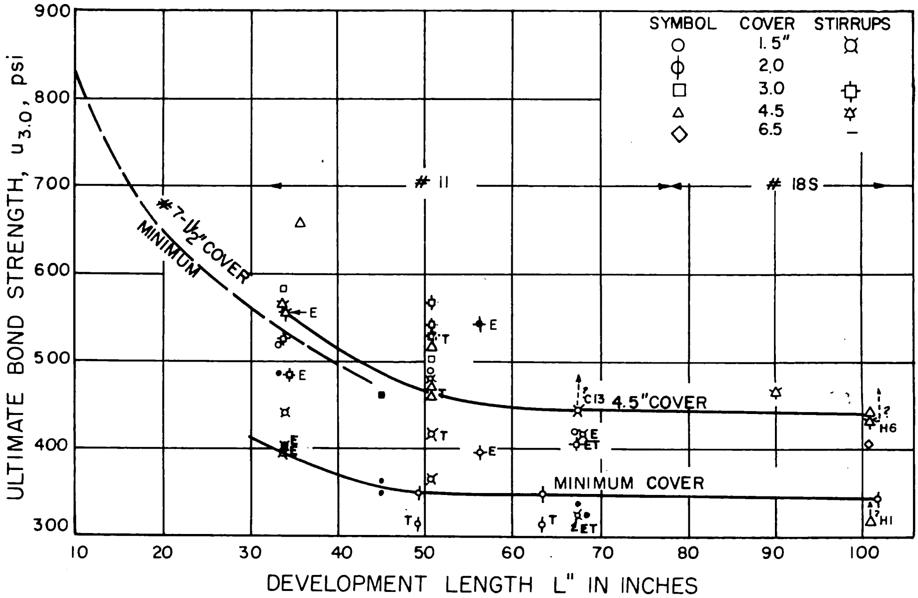


Fig. 17—Minimum ultimate bond strength for f_c' = 3000 psi. Lower curve based on cover of 1.5 in. over #11 bars and 2.25 in. over #18S bars

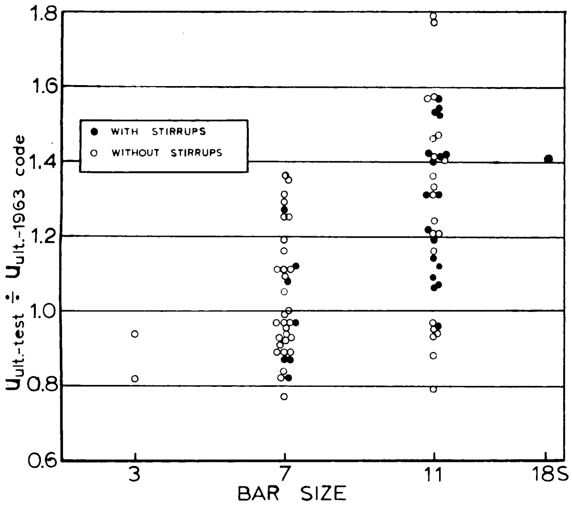


Fig. 18—Effect of bar size on ratio of ultimate bond test values to Code values

For #3 bars in a 6 in. width and #7 and #11 bars in an 18 in. width,* the bond resistance decreased with increasing development length as shown in Fig. 16. It appears from these data that the bar size itself would be a minor variable, in comparison with development length, if cover were made a constant number of bar diameters, but this is not common practice. Although the average bond resistance drops off rapidly with increasing development length L'', a check will indicate that the total bar stress developed increases with the length. This study was extended[6] to #11 bars in 24 in. wide beams with stirrups and in a few cases to #18S bars in beams in widths of 22 in. and 32 in. The ultimate bond stress tended to level off for development lengths in excess of 50 in. as indicated in Fig. 17. This curve is based on minimum strengths rather than the averages used in Fig. 16. In general, these data support the permissible bond values listed in the ACI Building Code for bars of A 432 steel ($f_y = 60$ ksi). Untrauer has made a study[15] of this in terms of bar size (Fig. 18) and in terms of bar cover (Fig. 19). The lower points are largely either for top bar specimens, which are known to have lower strengths, or for narrower beams. If only 40 ksi steel were used, slightly higher bond stresses could be used (with wide bar spacings) because development lengths could be shorter for this yield stress.

Crack width appears to be too large for the very long development lengths needed for large high strength bars. Fig. 20 shows a possible lower bond limit[6] if cracks are to be limited to a maximum of 0.02 in. at 55 percent of ultimate load, a limit probably much too liberal. A

*It is recognized that smaller beam widths per bar would be more critical, but adequate test data are unavailable.

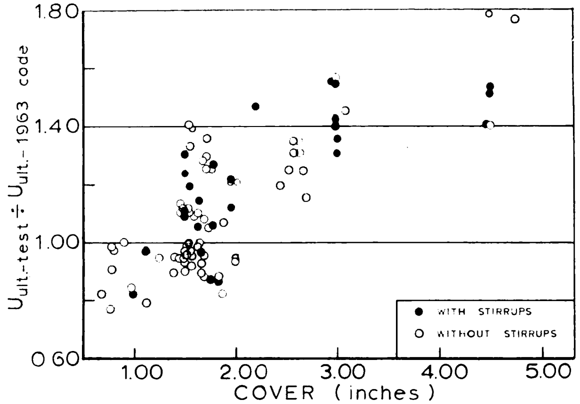


Fig. 19—Effect of cover on ratio of ultimate bond test values to Code values

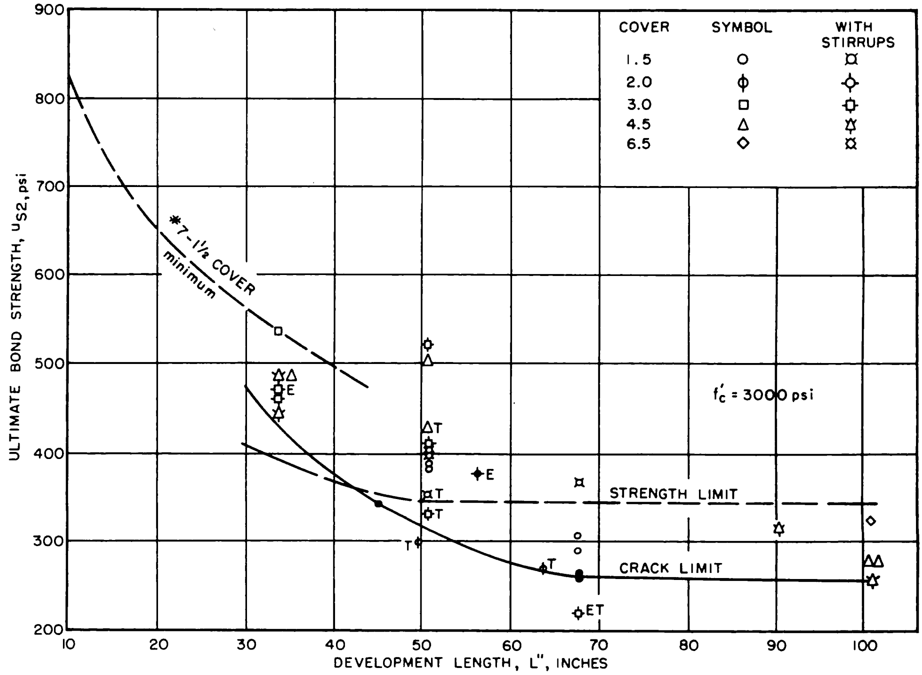


Fig. 20—Usable ultimate bond based on service load crack width limitation of 0.02 in. at tension face

still lower line would apply if the smaller usual crack width limits of 0.008 to 0.012 in. are used.

The development lengths required for three bar sizes are summarized in Fig. 21 for the data discussed above.

Lest this seem to indicate that the problem is completely solved, additional data for narrower width beams with 1.5 in. cover are plotted in Fig. 22. It appears that there probably should be a different curve for each bar size, for each clear cover over the bars, for each beam width, and for each ratio of stirrups. Hopefully, some of these curves will fall close together, but the experiments to date are not adequate to define the problem much closer than indicated here. A few tests have already indicated that a closely spaced layer of bars will split across the plane of the bars at stresses substantially below the ACI Code recognized values as in Fig. 6e. The Code values were derived largely from widely spaced bars or a single bar in a wide beam and may be presumed to be on the high side as a result. In extreme cases of close bar spacing the shear stresses may become large and bond may not govern, but bond failures may occur at low shear stresses in special cases.

Stirrups just adequate for expected shear seem to add little to bond strength, but excess stirrups, say double the required amount for

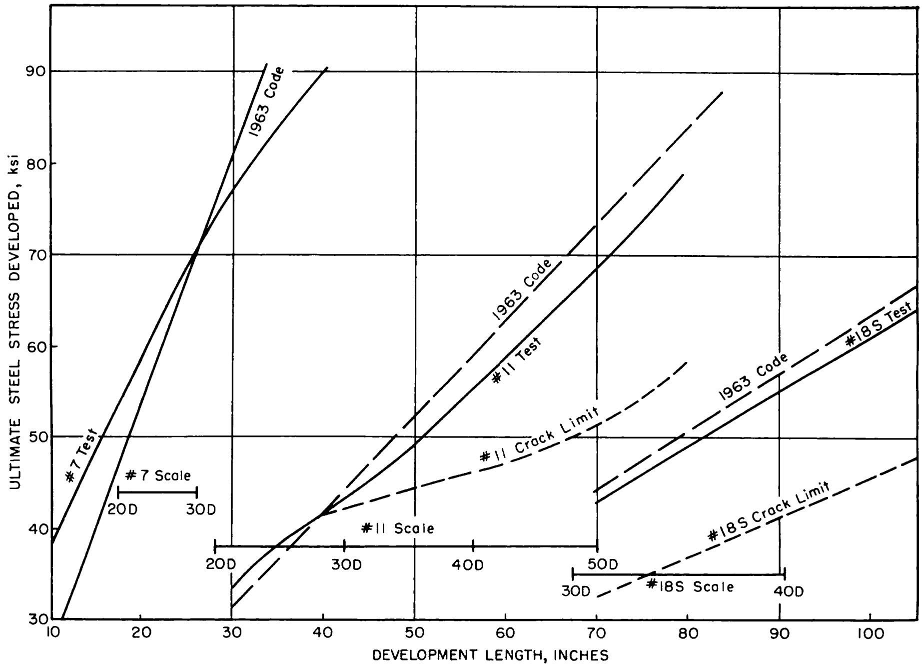


Fig. 21—Developed steel stresses compared to Code. Clear cover 1.5 in. except 2.25 in. for #18S. $f_c' = 3000$ psi

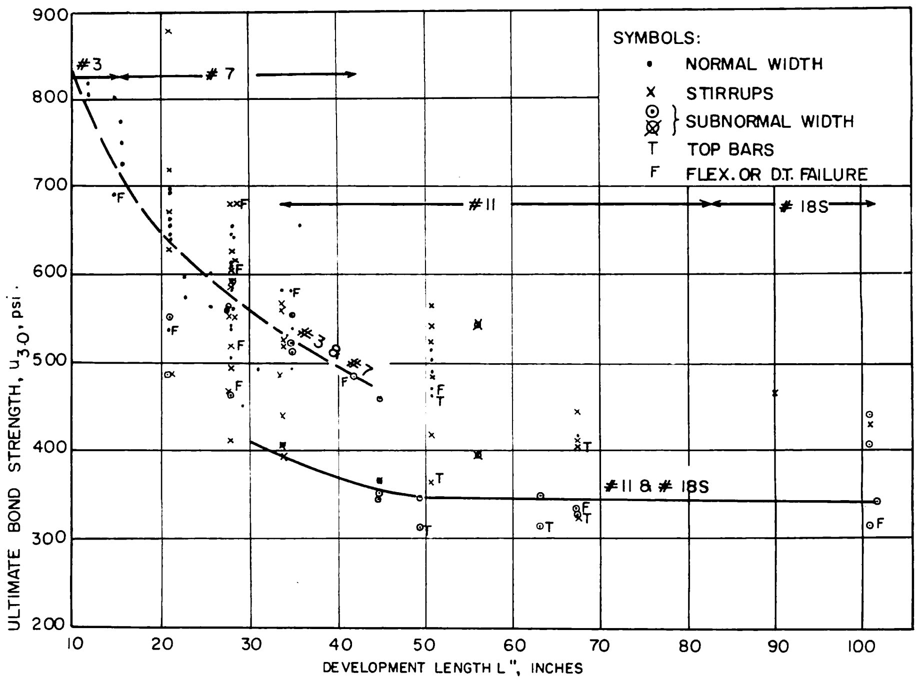


Fig. 22—Plot of all specimens with lower limit curves for 1.5 in. cover (2.25 in. for #18S)

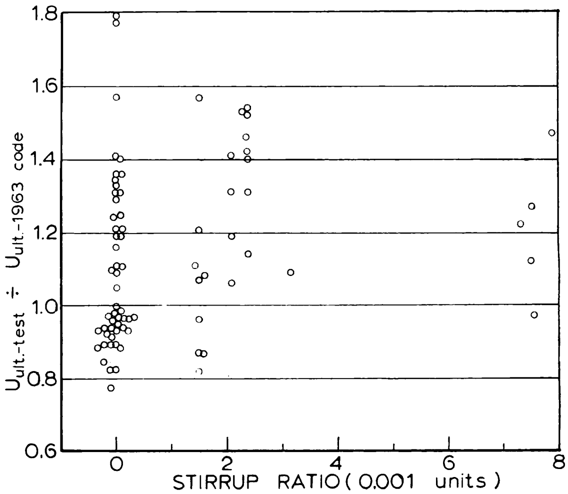


Fig. 23—Effect of stirrups on ratio of ultimate bond test values to Code values

shear, can add to bond strength.* This area has been little explored but Untrauer has pointed out (Fig. 23) that the low points in Fig. 22 are largely for beams without any stirrups; only one specimen having the stirrup ratio r as large as 0.002 fell below the ACI Code values and that one by only some 2 percent.

TENSILE STEEL EXTENDING BEYOND POINT OF INFLECTION

A few specimens have been tested where negative moment steel extended beyond the point of inflection. Although the data are not numerous enough to be conclusive, it appears that this added length may be considered as end anchorage adding directly to the development length. The Code permits such end anchorage to be evaluated equally with length within the point of inflection. It would seem that the bond stress which can be developed in this end anchorage in a compression area would, if anything, be better than where flexural tension exists. However, there is some evidence that the end anchorage may be slightly *less* effective (8 percent average on one series) than the usual development bond. If this is correct, it might arise from the complications resulting from the intrusion of a tension force into the top of the beam where a positive moment exists; or, more probably, it arises from excessive slip within the negative moment zone. This situation needs further clarification, both quantitatively and in terms of the mechanics involved.

TENSILE STEEL EXTENDING THROUGH A COLUMN

The vertical compression existing in a column would seem to grip a beam bar strongly and prevent bond splitting from developing. Untrauer and Henry[11] have run tests on simpler pullout specimens under lateral pressure on two faces which showed substantially increased ultimate bond stresses, increases being in the order of ⅔ to ⅞ for a lateral pressure of 1000 psi. Higher bond stress within the column should thus be permissible, but this is not recognized by the Code.

A special situation exists where large wind or earthquake moments occur in the beams on opposite faces of a column. In such cases a beam bar continuing through the column would theoretically carry high tensile stresses on one column face and compression on the opposite face. Within a heavily loaded column the necessary high bond stresses might be available, but the possibility of a loss of vertical loading in an earthquake might be considered. If the bond within the column should fail, the bar still cannot get loose because of its anchorage in the beam compression zone beyond the column. This should not result in any loss of the tension effectiveness of the bar if adequate

*One of the few comparative tests with extra stirrups is quoted in an earlier footnote. All the Bureau of Standards beams[12] contained very large stirrup ratios.

tension anchorage exists on the far side of the column, although two ill effects may result: (1) the excessive tension deformation near the column, because over more than a normal length; (2) the loss of effectiveness of the bar as compression steel on the far side and the increased total compression to be resisted there because of this tension force which has intruded into the cross section.

Where large earthquake moments exist in columns, moment above and below a floor may reverse just as in the beam just discussed. The same comments apply except that there is no normal compression force perpendicular to the bars and bond may not be as satisfactory as for the beam bars. This detail needs investigation since no test data are available.

DEVELOPMENT BOND VERSUS FLEXURAL BOND

At the end of a simple span development bond has no simple meaning because the steel stress is zero. Something such as flexural bond must be considered as a design restraint on the use of bars which are too large. This conclusion may be generalized to apply to any place where shear is large and moment is zero or small. Although not specifically stated in the Code, end anchorage should still be effective in such cases. Thus excessive flexural bond can be offset by added end anchorage even in this case.

When maximum moment and maximum shear for a uniform load occur at the same cross section, as in an isolated column footing, the calculation of flexural bond will typically lead to higher bond stresses than would a development bond calculation. However, this is closely related to the typical test situation from which allowable bond stresses are established. The bar cannot get loose unless it splits out. It is anchored near its ends in concrete which carries little moment and should be uncracked. Flexural bond stress near the column should be no more serious than the near ultimate bond stress adjacent to any crack, as already discussed. The ACI Code in Section 1801 (c) provides that flexural bond can be ignored in such cases, but because this is a sharp change in practice the Code has limited the anchorage bond stress to 80 percent of the usual allowable. It is not known whether this stress reduction is necessary or unnecessary; the case deserves research study.

TOP BARS

Many years ago it was established that the settlement of concrete in the form left the concrete better consolidated on top of the lugs of a vertical bar than beneath the lugs. The slip and ultimate strength were thus more favorable when the bar was pulled against the direction in which the concrete was deposited than when pulled in the direction in which the concrete settled. Likewise, a horizontal bar has better consolidated concrete above the bar than under it. Since any bleeding of the mix tends to accumulate water and air beneath the bar, the concrete mix and slump, rate of casting, and amount of vibration employed are each important. For typical cast-in-place concrete one can say the greater the depth of concrete below a top bar, the greater is the resulting loss in bond strength.

The magnitude of this loss of bond stress (considered as 30 percent in the Code for bars with over 12 in. of concrete below the bar) is not too well documented. When the original tests were run with bars rigidly supported, even greater losses were recorded. In recent tests with concrete depths of 12 to 18 in. below the bars, the loss has been less, in the order of 10 to 20 percent. Further experimental work is necessary before the designer can really know what bond strength is possible for top bars in ordinary cast-in-place beams of 6 to 10 ft depth, often made of 3 to 4-in. slump concretes.

In pullout tests of long high strength steel bars[3] one significant difference in behavior was observed. Bottom cast bars did not slip at the unloaded end until almost at their ultimate load. Top cast bars slipped at the unloaded end at very early (low) loads and then continued to accept much more load. Evidently there was enough open space (air and water under the bar) to permit it to slip, possibly by moving laterally, over the entire length prior to development of any serious splitting. This slip lowered the ultimate splitting resistance some 15 to 20 percent.

LIGHTWEIGHT CONCRETE

Data from the University of Missouri[1] indicate that in the order of a third of the bars in lightweight concrete pullout specimens pull out without splitting the concrete. This is particularly true for top cast bars. It is possible that bond failures of this type, where lugs crush or shear the concrete, may follow a strength variation different from the square root of f_c', which originated where tension splitting was dominant. The first analysis of these data shows lower bond strengths for the lightweight concrete than for regular concrete. The ratio varies considerably with the criterion selected, ranging from 87 percent on certain slip comparisons to 64 percent on an over-all ultimate strength basis. Such numerical values may indeed be premature, but even an order of magnitude may be helpful.

WEAK SPOTS IN EXISTING KNOWLEDGE

Although weak spots in existing knowledge have already been mentioned, it may be helpful to reassemble these. These weak spots include the effect of close spacing of bars (or beam width per bar), the efficiency of end anchorage beyond the point of inflection, the requirement for end anchorage in a short cantilever when loaded only through shears from intersecting beams, the variation in bond resistance with depth of concrete placed below the bar, the higher bond strength in compression, and the improvement available in both tension and compression bond from spirals or other binding. Other special details needing investigation are compression splices and mechanical splices, remedial measures to restore loss of shear strength when bars are cut off [Section 918(c) of ACI 318-63], and bond capacity in joints where reversal of moment applies for beam steel through a column or column steel through a beam (as in wind stresses). Lightweight concrete may require slightly modified provisions for bond. Where crack widths might control design, more knowledge is needed as to the cracks to be expected with various details (with either lightweight or ordinary concrete) and the effectiveness of various crack control methods.

The development of an adequate bond theory depends on the establishment of the real bond stress distribution, the real splitting forces developed, and what factors influence these two. Especially, a need exists for knowledge of what changes take place with repeated (or reversed) loading, even with fatigue loading. If empirical methods are to continue, the possibility of completely ignoring flexural bond stresses and using only development bond stresses should be further studied as a possible means of simplifying what is admittedly a complex set of Code provisions.

SUMMARY

Bond has been found to be an extremely variable type of stress such that average bond stress seems more significant than the local value at any particular point. This is particularly the case where flexural cracking exists. Each such crack creates points of bond stress concentration which influence the average usable stress.

When bars are cut off in a tension zone, large stress concentrations occur and shear strength is lessened, especially so at tension splices.

Because of the influence of tension cracks on bond strength, the usual pullout specimen is appropriate only for relative bond strengths, not absolute values. A proper test must be realistic as to the cover over the bar and the reinforcing surrounding it. At present, because of the lack of an acceptable detailed theory, many different stress situations have to be investigated individually.

Although much progress has been made with regard to bond stress over the years, there are still many weak spots in our knowledge.

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This report was submitted to letter ballot of the committee which consists of 22 members, all voted affirmatively. Title No. 63-53 is a part of copyrighted JOURNAL of the American Concrete Institute, Proceedings V. 63, No. 11, Nov. 1966. Separate prints are available at 75 cents each, cash with order.

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Discussion of this report should reach ACI headquarters in triplicate by Feb. 1, 1967, for publication in the Part 2 June 1967 JOURNAL. **(See p. iii for details.)**

Sinopsis—Résumé—Zusammenfassung

Adherencia—Estado del Conocimiento

Se discute la falla por adherencia enfatizándose la influencia del desgarramiento. Se enfatiza asimismo los enormes esfuerzos de adherencia adyacentes a una grieta y las complicaciones causadas por este esfuerzo de adherencia actuando hacia afuera y hacia adentro. Se describen los factores que influyen en el agrietamiento, así como los planos débiles en el desgarramiento. Se destaca la importancia del ancho de viga o el espaciamiento de barras en la resistencia a la adherencia. Se distinguen los conceptos de anclaje extremo, adherencia por flexión y adherencia por anclaje o desarrollo.

Se revisan los conocimientos actuales sobre el comportamiento en adherencia y del valor absoluto de la resistencia a la adherencia, primero en términos de empalmes y miembros especiales y después en la región de momento negativo de una viga. Se presentan algunas de las informaciones que justifican las disposiciones del Código de Construcción de 1963. Se indica el valor del anclaje extremo enfatizándose la importancia de la adherencia por desarrollo o de la longitud de desarrollo.

Se discute el efecto de la posición de las barras superiores y la influencia del concreto ligero. Finalmente, se mencionan los puntos débiles y los conocimientos existentes así como las áreas en las que se requieren más investigaciones.

Adhérence—Etat de la Question

Le rapport présente des considérations sur la nature de la rupture d'adhérence, en insistant sur l'influence du fendage du béton. L'attention est attirée sur le niveau élevé des contraintes d'adhérence au voisinage de toute fissure et sur les complications qui résultent de cette adhérence "extérieure et intérieure". Une relation est établie entre les facteurs qui influencent le fendage d'une part, et les plans préférentiels de fendage d'autre part.

Le rapport développe l'importance de la largeur de la poutre ou de l'espacement des barres sur la résistance d'adhérence. Les différences entre concepts d'ancrage d'extrêmitè, d'adhérence d'entrainement en flexion et de scellement, sont mises en évidence.

Le rapport passe en revue les connaissances actuelles relatives au comportement vis-à-vis de l'adhérence ainsi qu'à la valeur absolue de la résistance d'adhérence, tout d'abord en ce qui concerne les recouvrements de barres et les pieces spéciales, puis en ce qui concerne la région de moments négatifs d'une poutre. Une partie des données expérimentales justifiant les prescriptions du Code ACI 1963, est présentée. La valeur de l'ancrage d'extrêmité est indiquée et l'importance du scellement ou de la longueur de scellement est soulignée.

L'effet de la position de l'armature supérieure et l'influence du béton léger sont brièvement discutés. Enfin, les points faibles dans les connaissances actuelles ainsi que les domaines où des recherches supplémentaires seraient nécessaires sont mentionnés.

Die Verbundspannungen—Der Stand der Dinge

Grundsätze des Verbundbruches werden diskutiert und der Einfluss von Spaltspannungen hervorgehoben. Die hohen Verbundspannungen in Nähe eines jeden Risses und die Komplikationen, die durch diesen dauernden Wechsel entstehen, werden unterstrichen. Faktoren, welche die Spaltspannungen beeinflussen, werden zu den Ebenen niederer Spaltfestigkeit in Beziehung gesetzt. Die Bedeutung der Balkenbreite oder des Stababstandes für die Verbundfestigkeit wird abgeleitet. Die Grundprinzipien von Endverankerung, Biegeverbund und Verankerung oder Verbundentwicklung werden einander gegenübergestellt.

Der gegenwärtige Wissensstand über das Verbundverhalten und absolute Werte des Verbundwiderstandes werden zusammengestellt, zuerst für Bewehrungsstösse und spezielle Bauteile und dann für den Bereich negativer Momente in einem Balken. Einige Daten, welche die Vorschriften des Building Code 1963 rechtfertigen, werden gezeigt. Der Vorteil einer Endverankerung und die Wichtigkeit der Verbundentwicklung oder der Übertragungslänge wird hervorgehoben.

Die Bedeutung obenliegender Stäbe und der Einfluss von Leichtbetonen werden kurz diskutiert. Abschliessend werden die schwachen Stellen im gegenwärtigen Wissen und jene Gebiete erwähnt, die weiterer Forschung bedürfen.