ABSTRACT

Results reported in this paper constitute an investigation into bond between plain and deformed wires and micro-concrete in models. A total of 223 tests on eccentric pull-out specimens and 35 concentric pull-out specimens and 48 bond beam specimens were conducted to investigate the influence of concrete strength, clear cover, end anchorage, vertical stirrups and rust on bond characteristics in models.

Models of concentric pull-out specimens and eccentric pull-out specimens with special support conditions (developed at McGill) were used for the pull-out tests. Models of the University of Texas beam specimens and the symmetrical bond beam specimens (developed at McGill) were used for the beam tests. Rational bond criteria have been suggested for reinforced concrete models and small sized specimens using steel wire as reinforcing.

The results indicate that many factors affect bond characteristics in pull-out and bond beam specimens. No significant difference was noted between the average ultimate bond stress values from the eccentric pull-out and the symmetrical bond beam tests. The mechanisms of failure in pull-out and bond beam specimens were carefully examined. These failure mechanisms and crack patterns appear to agree reasonably with the prototypes tested by other investigators. Test results also suggest the possibility of a new design approach for L"/D ratios less than 15 in reinforced concrete design.

INVESTIGATION OF BOND IN
REINFORCED CONCRETE MODELS

by

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A thesis submitted to the Faculty of
Graduate Studies and Research in
partial fulfilment of the requirements
for the Degree of Master of Engineering

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April, 1969.

TO

MY PARENTS

AND

MY TWIN BROTHER

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Results reported in this paper constitute an investigation into bond between plain and deformed wires and micro-concrete in models. A total of 223 tests on eccentric pull-out specimens and 35 concentric pull-out specimens and 48 bond beam specimens were conducted to investigate the influence of concrete strength, clear cover, end anchorage, vertical stirrups and rust on bond characteristics in models.

Models of concentric pull-out specimens and eccentric pull-out specimens with special support conditions (developed at McGill) were used for the pull-out tests. Models of the University of Texas beam specimens and the symmetrical bond beam specimens (developed at McGill) were used for the beam tests. Rational bond criteria have been suggested for reinforced concrete models and small sized specimens using steel wire as reinforcing.

The results indicate that many factors affect bond characteristics in pull-out and bond beam specimens. No significant difference was noted between the average ultimate bond stress values from the eccentric pull-out and the symmetrical bond beam tests. The mechanisms of failure in pull-out and bond beam specimens were carefully examined. These failure mechanisms and crack patterns appear to agree reasonably with the prototypes tested by other investigators. Test results also suggest the possibility of a new design approach for L"/D ratios less than 15 in reinforced concrete design.

KEY WORDS

Bond, Anchorage Bond, Average Ultimate Bond Stress, Reinforced Concrete Models, Plain and Deformed Reinforcing Wires (A.S.T.M. A496-64). Micro-concrete, Eccentric and Concentric Pull-Out Tests, University of Texas Beams, The Symmetrical Bond Beams, Crack Patterns, Ultimate Strength Design Method, McGill Structural Laboratory.

NOTATIONS AND DEFINITIONS

NOTATIONS

The following symbols were generally used in this thesis.

L'' = Development length or embedment length, in.

D = Bar or wire diameter, in.

f'_c = Maximum compressive strength of concrete (unconfined), psi.

P = Ultimate load sustained by the bar (or wire) in the pull-out test or ultimate load at cantilever end in bond beam test, lb.

P^* = Total ultimate load recorded by the testing machine, lb.

P^{**} = Total weight of steel plates, I-beam and steel rollers, lb.

f_s = Tensile stress in bar (or wire), psi.

d' = Clear cover, in.

u = Average ultimate bond stress, psi.

u^* = Average ultimate bond stress in pull-out specimen, calculated from equation (3).

u^{**} = Average ultimate bond stress in beam specimens, calculated using working stress analysis equation. Value of j was taken as 0.875.

u^{***} = Average ultimate bond stress in beam specimens calculated using the ultimate strength criteria equation (13) or (13A).

A_s = Area of bar (or wire), in^2.

X = Bond coefficient, $= \dfrac{u}{f_c'^{\,0.5}}$

Y = Bond coefficient, $= \dfrac{u}{f_c'^{\,0.7}}$

b = Width of specimen, in.

t = Overall depth of specimen, in.

d = Effective depth of specimen, in.

Specimen No. PPS1P = Specimen number, pull-out test, plain bar, straight, S1 type, concrete compressive strength.

Specimen No. PDH3A(R) = Specimen number, pull-out test, deformed bar, hooked, H3 type, concrete compressive strength, rusted for one month.

Specimen No. BPS1A' = Specimen number, bond beam test, plain bar, symmetrical bond beam test, type 1, concrete compressive strength.

Specimen No. BDF1B'(R) = Specimen number, bond beam test, deformed bar, University of Texas beam test, deformed bar, concrete compressive strength, rusted for one month.

DEFINITIONS

Bond

Bond is used to describe the means by which slip between concrete and steel is prevented or minimized.

Bond Stress

Bond stress is the name assigned to the unit shearing force acting parallel to the bar on the interface between bar and concrete.

Pull-Out Test

In the pull-out test, the bar (either plain or deformed) is initially embedded in a cylinder or prism (either concentric or eccentric). This test is designed to predict the bond action in beams.

Bond Beam Test

This test is designed to investigate the bond behaviour in beams.

Flexural Bond

Flexural bond is the bond defined by the equation $u = \frac{V}{\Sigma o j d}$. It is a measure of the local bond stress necessary to produce the local ΔT bar pull demanded by flexure.

Model

A device which is so related to a prototype that observation on the model may be used to predict accurately the performance of the prototype in the desired respect.

Prototype

Original or actual structure.

Anchorage Length or Development Length

An anchorage length is the length necessary to take a given stress out of a bar while development length is the length necessary to put a given stress into a bar. ACI Committee 408[1] suggested that these two concepts are identical.

Bond Stress Distribution

Bond stress distribution is a distribution of average anchorage or development bond stress (or bond strength, or bond force) along the reinforcing bars.

Failure Criteria in Pull-Out Tests

(1) Bond Failure (B.F.) - this type of failure is due to bar (or wire) being pulled out.

(2) Steel Failure (S.F.) - the bar (or wire) reaches its ultimate strength and fractures.

(3) Steel Failure (Fracture) and Bond Failure (S.F. & B.F.).

(4) Bond Failure and Concrete Splitting (B.F. and C.S.).

(5) Steel Failure (Fracture), Bond Failure and Concrete Splitting (S.F., B.F. & C.S.).

(6) Steel Failure (Fracture), Bond Failure and Concrete Splitting (Diagonal Tension Failure included) (S.F., B.F., & C.S. (D.T.)).

Failure Criteria in Bond Beam Tests

(1) Bond Failure (B.F.) - this type of failure starts with a moment crack at the beginning of L" (right where the aluminium sheet is inserted).

(2) Bond Failure and Diagonal Tension Failure (B.F. & D.T.) - a combination of bond failure and diagonal tension failure, note that bond failure occurred first.

(3) Bond failure, Diagonal Tension Failure and Concrete Splitting (B.F., D.T. & C.S.) - a combination of bond failure, diagonal tension failure and concrete splitting.

(4) Serious Diagonal Tension Failure (S.D.T.) - this denotes severe diagonal tension distress. Note that diagonal tension failure occurred before bond failure. The test results of beam failing in this mode have not been analysed because of the difficulty and complications of analysis.

CHAPTER I

GENERAL INTRODUCTION

1.1 THE NATURE AND PROBLEM OF BOND

"Bond" is used to describe the means by which "slip" between concrete and steel is prevented or minimized. Wherever the tensile or compressive stresses in a bar change, bond stress must act along the surface of the bar to produce the change. Hence, "bond stress" is in effect longitudinal shearing stress acting on the surface between steel and concrete[(1)*].

The fundamental assumptions in the design and analysis of flexural members are:

(a) Plane sections remain plane after bending.

(b) Concrete does not resist any tension.

(c) Perfect bond exists between steel and concrete such that no slip occurs.

The ACI Committee 408[(1)], presented the following weak spots in the existing knowledge of bond:

(a) The effect of close spacing of bars (or beam width per bar).

(b) The efficiency of end anchorage beyond the point of inflection.

(c) The bar end anchorage requirement in a short cantilever when loaded through shears from intersecting beams.

(d) The variation in bond resistance with depth of concrete placed below the bar.

(e) The higher bond strength in bars in compression.

(f) The improvement available in both tension and compression bond from spirals or other types of binding.

(g) The investigations of compression and mechanical splices and remedial measures to restore loss of shear strength when bars are cut off.

(h) Bond capacity in beam-column joints where reversal of moment occurs.

(i) Modified bond provisions for light-weight concrete.

(j) Where crack widths might control design, more knowledge is needed on the nature and details of expected cracks and the effectiveness of various crack control methods.

---

* Indicates the references shown in Bibliography.

(k) Many different stress situations have to be investigated "individually" because of the lack of an acceptable comprehensive bond theory.

Weak spots detailed by other investigators have been discussed in Chapter II, III and IV.

Bond between steel and concrete is an important parameter affecting the behaviour and strength of reinforced concrete elements under individual and combined loadings(2). Comprehensive investigations on the strength of reinforced concrete members under axial loads, flexure and shear have been reported by ACI Committee 105(3), ACI-ASCE Committee 327(4), and Kani(5). However, experimentors investigating the strength and the behaviour of beams under torsion and its combinations had most of the parameters varying, thereby making the effect of any particular parameter a matter of conjecture. To establish a direct relationship between the strength of the section and the different parameters, it is clear that a number of test series must be run with only one parameter varied during the series. As available time, space and financing are all finite, the use of full sized specimens for such investigations appears impractical. Direct models have been extensively used as aid to reinforced concrete design(6) appear to be a possible solution for experimental investigations under combined loadings(7). Alami and Ferguson(2) experimented with six series of 19 beams of different sizes failing in diagonal tension, bond, and flexural compression. They observed that when shear or flexural compression failure was expected, models with scales as small as 0.221 appeared adequate to predict the behaviour of the prototype. The difference between the actual and predicted stiffness of a model became more significant as its scale was made smaller. They also concluded that models fail to predict the behaviour of reinforced concrete prototypes when bond was the primary reason of failure. Furthermore if bond was the secondary reason for failure, doubt was cast on the use of models. Similar conclusions were reached at McGill University as a result of tests under combined bending, shear, and torsion on quarter-scale reinforced concrete and one-eighth scale prestressed concrete models(7).

1.2 THE OBJECT OF PRESENT INVESTIGATION

Successful use of models as design and research tools depends on achieving reasonable bond similitude between the prototypes and the models.

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1.2 THE OBJECT OF PRESENT INVESTIGATION

Successful use of models as design and research tools depends on achieving reasonable bond similitude between the prototypes and the models.

This investigation was aimed at investigating the bond characteristics of plain and deformed mild steel wires in reinforced concrete models using eccentric pull-out and bond beam tests. These tests represent the loading conditions existing in beams in practice.

The relationship between the bond strength results from the pull-out and the bond beam tests was also an object of investigation.

Another objective of the present investigation was to develop a rational design criterion for bond in reinforced concrete models.

1.3 PREVIOUS BOND INVESTIGATIONS IN REINFORCED CONCRETE MODELS

(A) M.I.T. INVESTIGATIONS[8]: The M.I.T. team used bond beam specimens similar to the ones used by Ferguson at University of Texas[1] in studying the bond resistance of high strength deformed bars. They also used concentric pull-out specimens consisting of a fixed length of wire embedded in 2-inch diameter mortar cylinder. The M.I.T. findings were as follows:

(i) Effect of Rust:

(a) The unrusted wires showed significantly low values of bond as compared with similar but rusted samples.

(b) Good rusting for 7 days increased the bond resistance and prolonged rusting did not result in increased bond beyond that of 7 days. (Both pull-out and beam specimens were included).

(ii) Effect of Concrete Strength f'_c (beam tests):

The effect of concrete compressive strength on bond resistance could not be determined conclusively and continues to be an unsolved problem.

(iii) Effect of Embedment Length (beam tests):

SWG No. 12, 14 and 16 reinforcing showed a significant decrease of bond stresses with increased L"/D ratio but no such decrease was observed for SWG No. 10 wires.

(iv) Effect of Bar Diameter (beam tests):

The relation between bond strength and wire diameter was thought to be of cubic or higher order.

(v) The Correlation Between Pull-Out and Beam Tests:

(a) Beams and pull-out specimens were of the same D and L"/D ratio.

(b) The beam specimens showed twice as much bond resistance as the pull-test.

(c) The limited scope of the pull-out tests did not allow elaborate study of the different variables affecting bond strength.

(B) CORNELL UNIVERSITY TESTS[9]: White at Cornell University used 1-inch diameter pull-out specimens similar to those used by the M.I.T. team. Their principal findings were as follows:

(i) Plain wires showed a marked decrease in average ultimate bond stress with increasing embedment ratio (L"/D).

(ii) Deformed wires had bond strength comparable to large prototype bars; for L"/D ratios larger than 15 the specimens failed by steel yielding of the wire. (The mix for 1-inch diameter cylinder was 0.5:1:2.6. Using sand finer than U.S. sieve No. 8, curing was accomplished in a wet room for 28 days); or for L"/D >8 the specimens failed by steel yielding (wet curing for fifteen days).

(iii) Some suitably deformed wires showed pull-out strengths very close to those measured for prototype bars.

(C) McGILL UNIVERSITY TESTS[10]: The concentric pull-out tests were identical to those of M.I.T. and Cornell. The results can be summarized as follows:

(i) Bond strength increased with decreasing diameter of wire.

(ii) Bond strength decreased with increasing L"/D ratios, however, the relationship was not linear.

(iii) The minimum embedment length required to develop ultimate strength in steel increased with an increase in diameter for plain wires.

(iv) The minimum embedment length required for steel fracture for 1/16" diameter wires was found to be between 70 and 75 times diameter.

(v) The minimum embedment length causing steel fracture for 5/32" diameter wires appeared to lie somewhere between 180 and 260 diameters.

1.4 FUTURE APPROACH

(a) The current ACI Code provisions on bond[11] appear to be conservative and limited in scope.

(b) The flexural capacity of a beam is a three-dimensional phenomenon and depends not only on the cross sectional properties at a section along the span but also on the development length in both directions. Also, bond, shear and moment resistances can not be regarded as independent responses to given loads. It is only convenient for calculation purposes to treat each resistance as a separate entity in the ACI Code. There is a need to develop a rational bond theory to account for combined effects of shear and flexure.

(c) The development of a rational bond theory depends on the establishment of the bond stress distribution, the splitting forces developed, and different factors influencing them.

CHAPTER II

INVESTIGATION OF BOND IN REINFORCED CONCRETE MODELS -PULL-OUT TEST

2.1 INTRODUCTION

In the standard concentric pull-out test, either plain or deformed bar is usually embedded in a cylinder or a prism, and pulled out while concrete at the same end is subjected to compressive stresses[1,12,13...21].

The pull-out test has some advantages[1] over the bond beam test. It is economic, simple and less time consuming. It is of interest to find out the reliability of pull-out tests in predicting bond action in beams. However, the weakness of the simple pull-out test as a standard is that the concrete at the loaded end is in compression and eliminates transverse tension cracking. Perry and Thompson[18] studied the bond stress distribution using the eccentric pull-out test and correlated these stresses with those existing in reinforced concrete beams in the neighbourhood of a crack in the constant moment region. They found little similarities in the bond stress distributions, however the magnitude of the maximum bond stress was approximately the same. Mains[13] used beam and pull-out specimens to study bond stress distributions and noted that cracks in beams decisively affected the magnitude and distribution of tensile and bond stresses. He calculated the tensile force in reinforcing bars and observed that the calculated values were usually lower than measured values for loads near the ultimate when shear as well as moment acted on the beam.

The McGill investigations used a modified eccentric pull-out test with the support conditions detailed in Fig. 1. The advantage of this test is the fact that it represents the conditions of flexure and shear which normally exist in the beams subjected to vertical loads. When the author was finalizing the results for Reference 22, Kemp et al[11] published their experimental studies on the effect of rust and scale on the bond characteristics of deformed reinforcing bars using "the new cantilever bond specimen." These specimens like the ones developed by the authors independently are more representative of the strain gradient that exists between a flexural crack and the beam support. Kemp et al[11] used steel conduits over the

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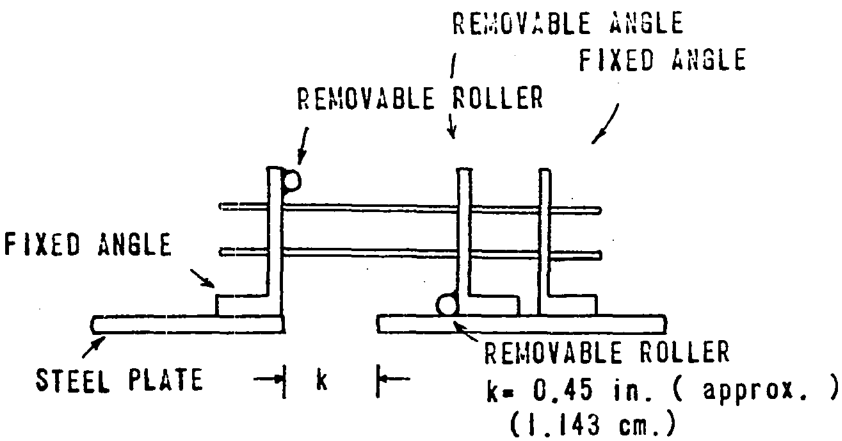

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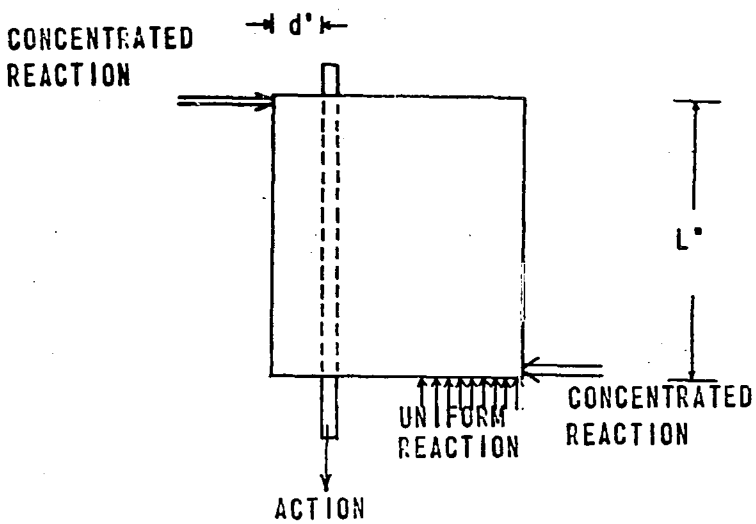


FIG. 1 SUPPORT DESIGN FOR ECCENTRIC PULL-OUTS

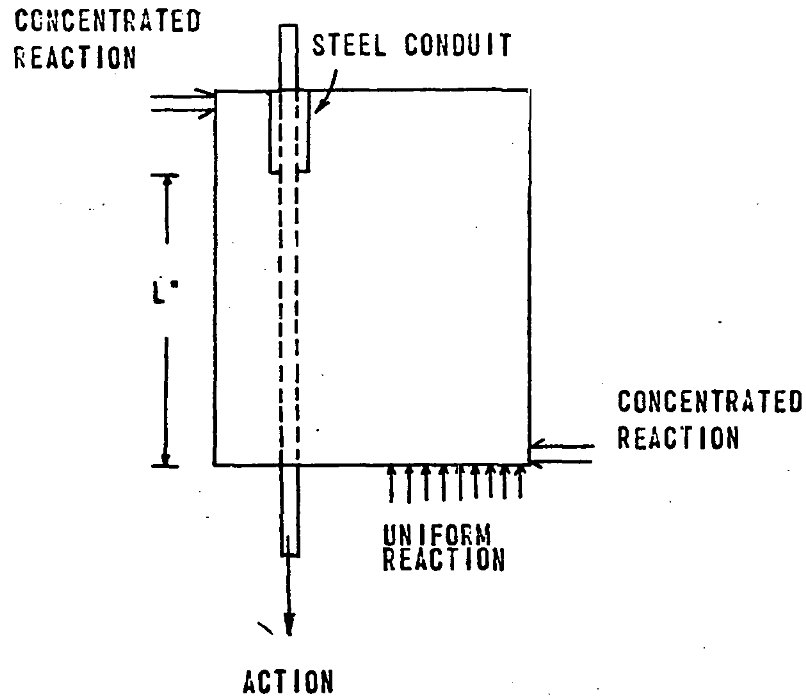
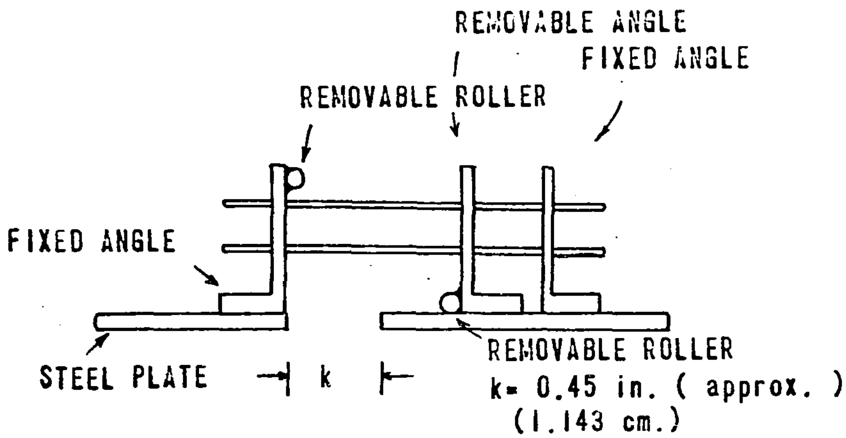


FIG. 2 SUPPORT DESIGN FOR ECCENTRIC PULL-OUTS

(AT THE FREE END BE NOT DISTURBED BY
CONCENTRATED REACTION)



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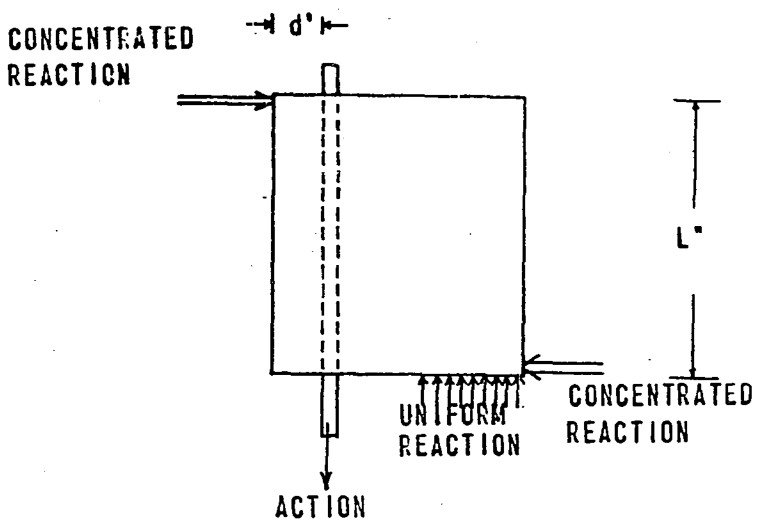

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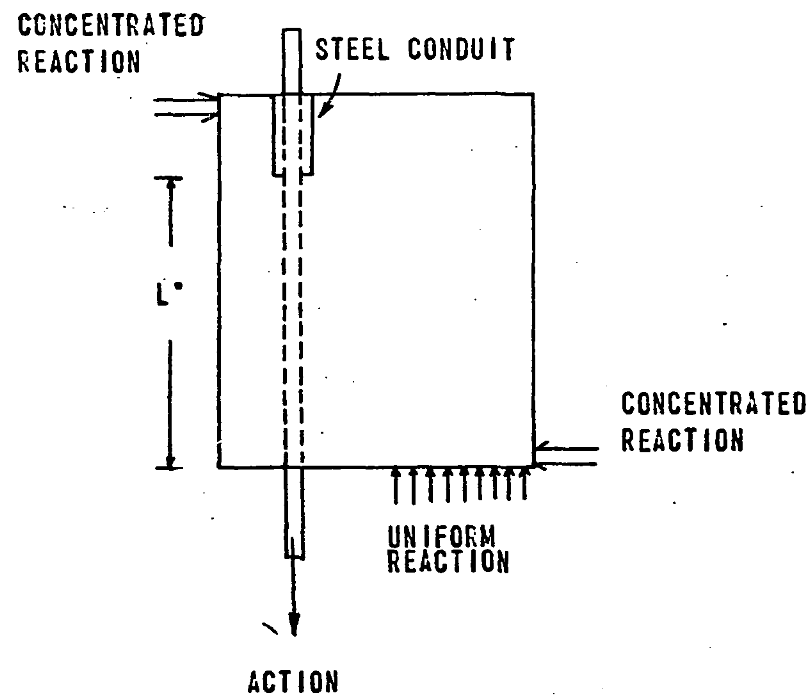

FIG. 2 SUPPORT DESIGN FOR ECCENTRIC PULL-OUTS
(AT THE FREE END BE NOT DISTURBED BY
CONCENTRATED REACTION)

reinforcing bar at both ends which eliminates the local effects of concentrated loads. However their specimens do not provide for the movement of the neutral axis that takes place in flexural specimens as the loads are increased from zero up to the failure load. It is interesting to note that the crack patterns obtained in this investigation are very similar to those obtained by Kemp et al in their specimens with conduits at both bar ends. The merits of using a conduit at the simple support can be appreciated and it is recommended that future investigators consider its use (Fig. 2).

2.2 SPECIMENS AND TEST PROCEDURE

All test specimens were 1 x 1.94 in. (25.4 x 49.3 m.m.) in cross-section and had varying lengths to suit the development length L" of the wire being tested. The details of the specimen dimensions, wire diameter, clear cover were listed in Tables 1 through 12 in Appendix A. The specimens were cast from micro-concrete prepared from a graded mixture of crushed quartz sand of five different grades (all passing Sieve No. 10) and high early strength cement. Three different nominal concrete strengths were used, e.g. 3000 psi (211 kgm/cm^2), 4000 psi and 5,000 psi. The strength of concrete was obtained from compression tests on 3 x 6 in. (76 x 152 m.m.) cylinders and are detailed in Tables 1 through 12 in Appendix A. Two types of steel were used - (i) low carbon soft steel plain wire and (ii) low carbon steel deformed wire conforming to the ASTM A496 - 64* specifications (with indentations projecting inwards instead of outward projections or protrusions as in normal deformed steel bars). Six different diameters of both plain and deformed bars were used. The details are given in Tables 1 through 12 along with the embedment lengths L". The properties of steel wires used are listed in Tables 17 in Appendix C.

A total of 223 eccentric pull-out specimens and 35 concentric pull-out specimens were tested using the support details shown in Fig. 1 in a 60,000 lb. (4210 kgm.) Riehle Testing Machine (least count = 2 lb. (0.14 kgm)). The details of the end anchorages used in this investigation are given in Fig. 3. The specimen nomenclature was derived as follows:

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* See Appendix C.

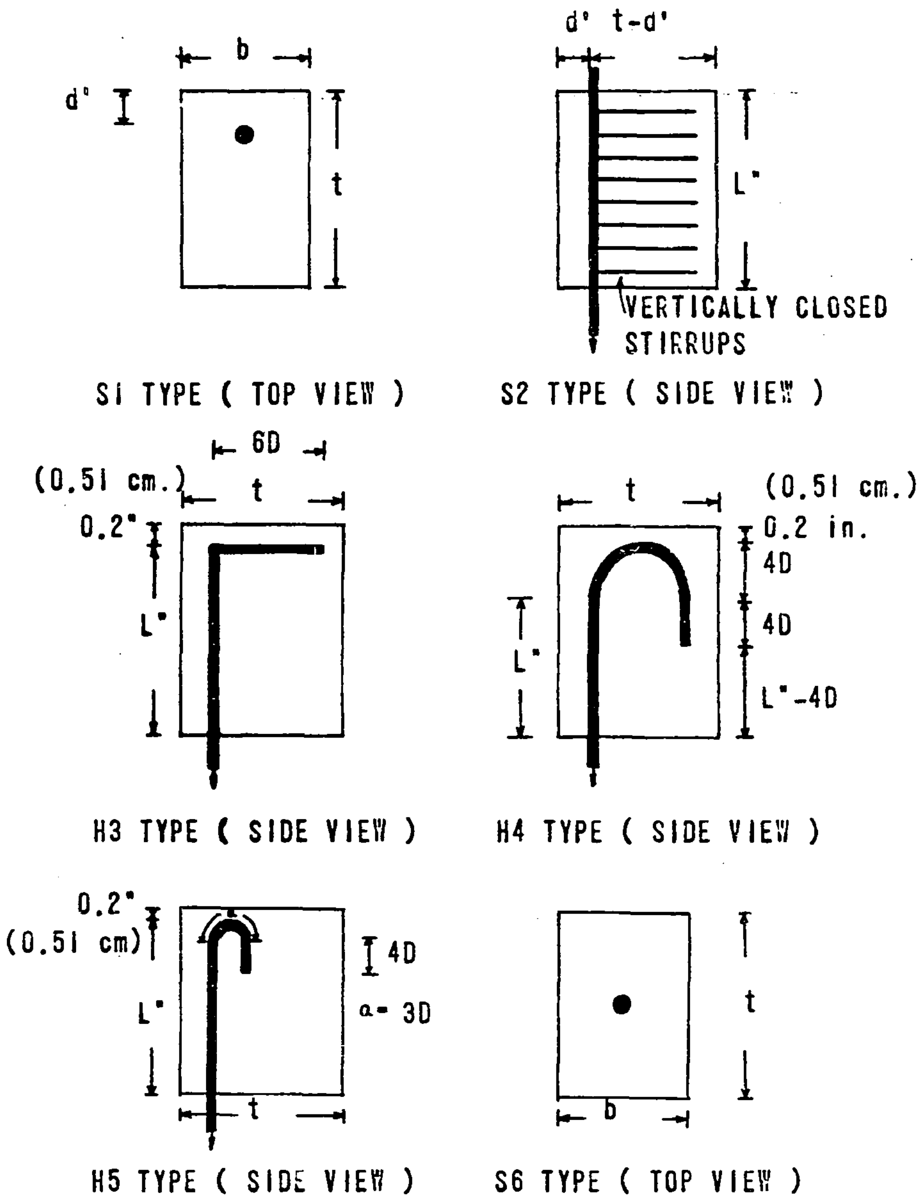


FIG. 3 VARIOUS KINDS OF PULL-OUT SPECIMENS

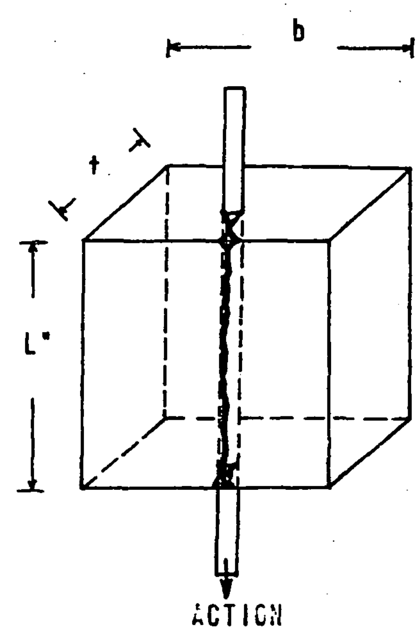


FIG. 4
C. S. & B. F.
(DEFORMED BAR)

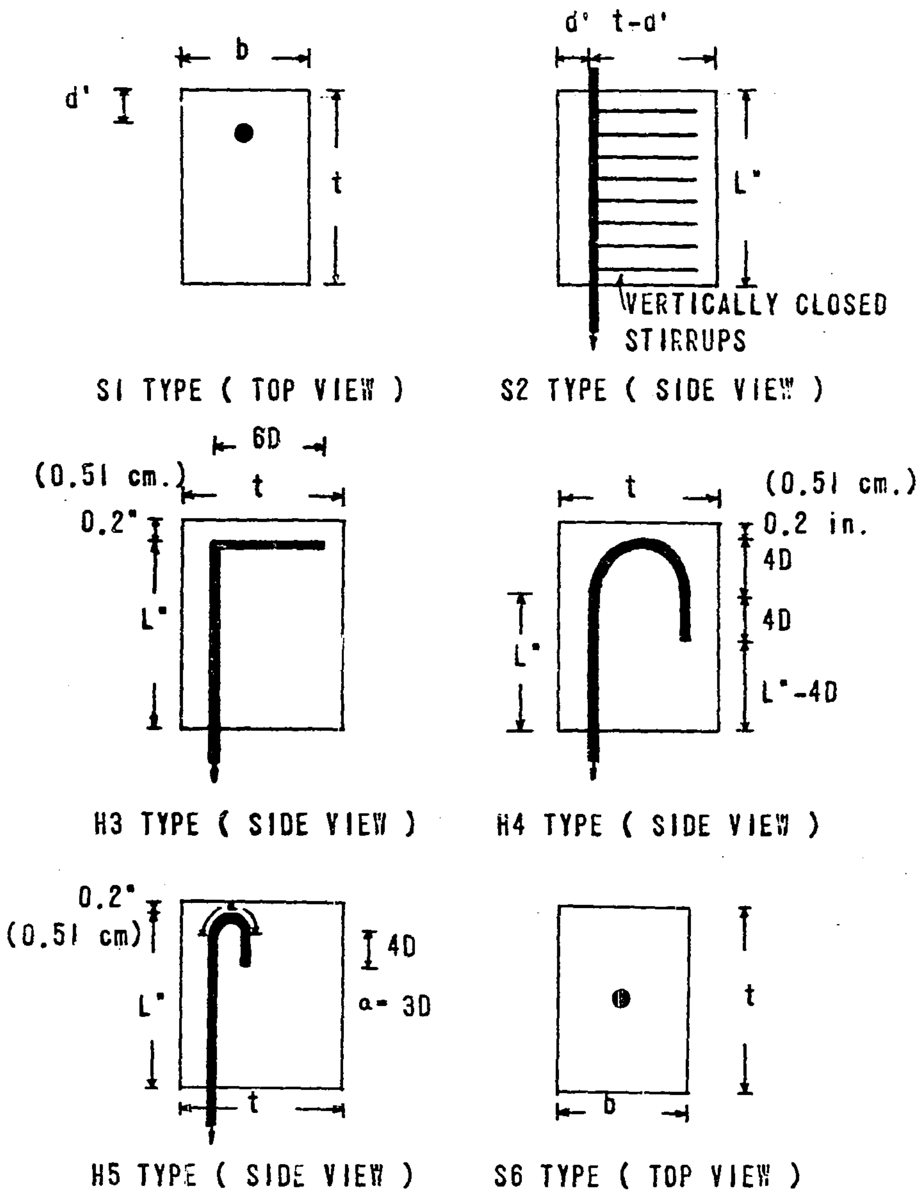


FIG. 3 VARIOUS KINDS OF PULL-OUT SPECIMENS

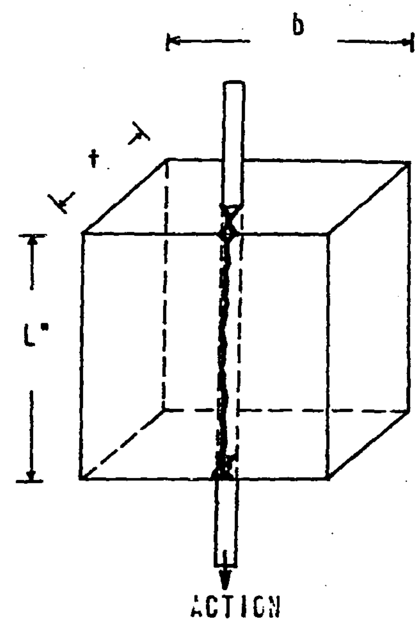


FIG. 4
C. S. & B. F.
(DEFORMED BAR)

1. The first letter P indicates the pull-out test.
2. The second letter D indicates the use of deformed bars while P indicates the use of plain bars.
3. The third letter and the fourth numeral indicate the type of end anchorage used in the specimens (Fig. 3).
4. The fifth letter indicates the type of concrete used. Concrete properties for each specimen are detailed in Tables 1 through 12 in Appendix A and C.
5. The sixth numeral indicates the sequential numbers of the development length being used.

The name PDS1G-6 indicates a pull-out test on a specimen containing a deformed steel wire with no end anchorages and concrete of type G (e.g. 3000 psi (211 kgm/cm^2) compressive strength) and sixth embedment length-diameter (L"/D) ratio is being used with all other variables being maintained constant.

The details of test results are given in Tables 1 through 12 in Appendix A. The tables also show the calculated values of the average ultimate bond stress u and the bond coefficient X calculated as follows:

$$u = \frac{P}{DL''} = \frac{f_s}{4x(L''/D)} \qquad (1)$$

and $$X = u/(f'_c)^{0.5} \qquad (2)$$

2.3 CRACK PATTERNS AND MECHANISMS OF FAILURE

The following behaviour and mechanisms of failure were noted for the specimens reinforced with plain and deformed bars with no end anchorages or stirrups.

(A) PLAIN BARS: There were two modes of failure

(i) The bar was pulled out of concrete without any distress in concrete e.g. tension splitting (denoted as B.F. - bond failure in the tables of results).

(ii) If the development length was sufficient, a rupture of steel bar was obtained at ultimate load (denoted as S.F. - steel failure).

(B) DEFORMED BARS: There were five different mechanisms of failure observed during the eccentric pull-out tests on specimens with deformed wires.

(i) Longitudinal Splitting:

A longitudinal bond crack first appeared at the loaded end directly under the bar on the clear cover side. As the loads increased, this crack slowly propagated along the bar length to the unloaded end when failure occurred. (denoted as B.F.- bond failure and C.S. - concrete splitting). (Fig. 4)

(ii) Flexural Cracking and Longitudinal Splitting:

A flexural crack first appeared across the face of the specimen on the clear cover side. This was followed in some cases by more flexural cracks and longitudinal splitting along the bar length as detailed in (i) above. (also denoted as B.F. and C.S.). (Fig. 5)

(iii) Bond Failure Accompanied by Diagonal Tension Cracking:

The longitudinal bond cracks first developed along a horizontal plane at the reinforcement level and were noticed as horizontal cracks. As the applied load was increased, the bars could not redistribute the forces any further and finally failure occurred due to diagonal tension as shown in Fig. 6.(denoted as B.F., C.S. and D.T. - diagonal tension)

(iv) Steel Failure:

In this case the embedment length of steel was sufficient to develop the ultimate tensile strength of the wires and failure occurred due to failure of steel. No transverse or longitudinal bond cracks or any other type of distress was noticed in concrete. (denoted as S.F. - steel failure) (Fig. 7)

(v) Steel Failure with Flexural Cracking:

Steel wires exhibited behaviour identical to that in (iv) above, however, transverse flexural cracks were noticed as soon as steel fractured. (Fig. 8)

The following behaviour and mechanisms of failure were observed for specimens with end anchorages or vertical stirrups.

(i) Deformed Bars with Hooked End Anchorage:

Longitudinal splitting of concrete at reinforcement level and transverse flexural cracking were noticed as for the specimens reinforced with deformed wires with no end anchorages. However, local distress was noticed in concrete around the hook, with

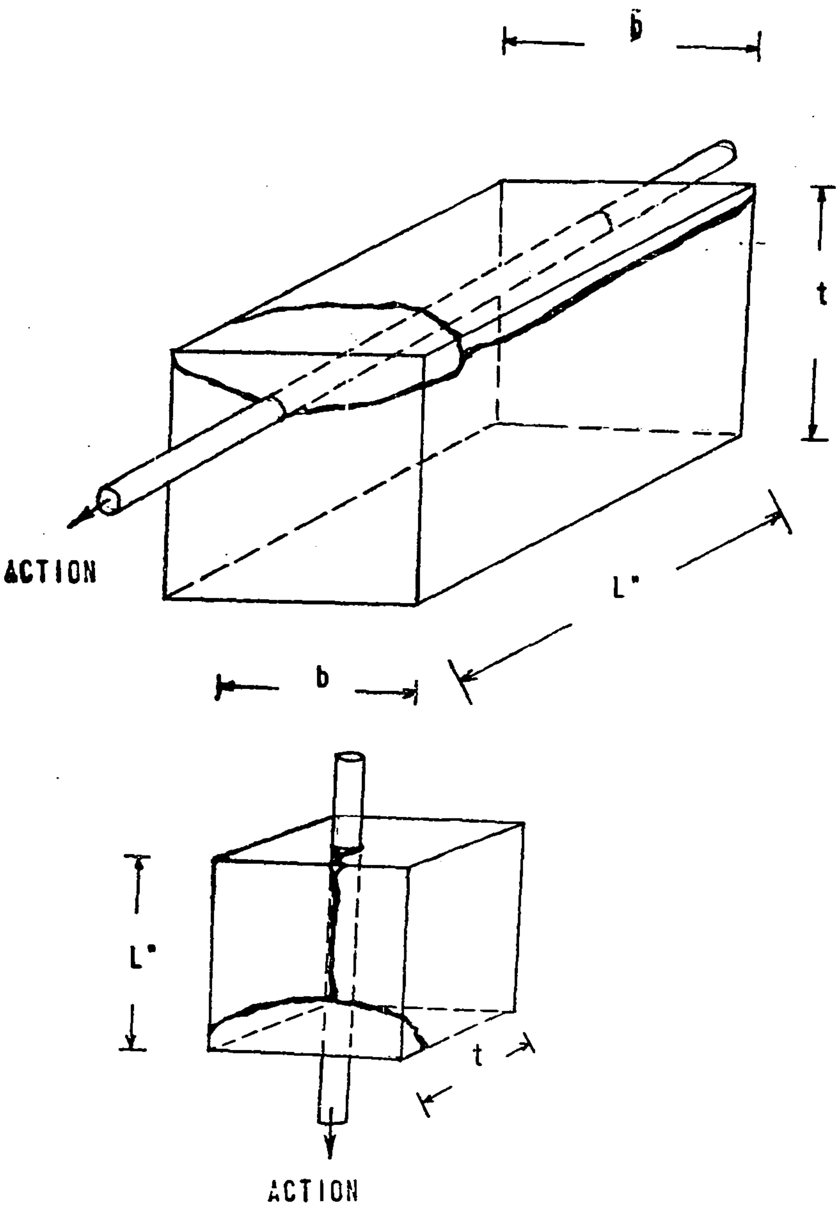


FIG. 5 C. S. & B. F. (DEFORMED BAR)

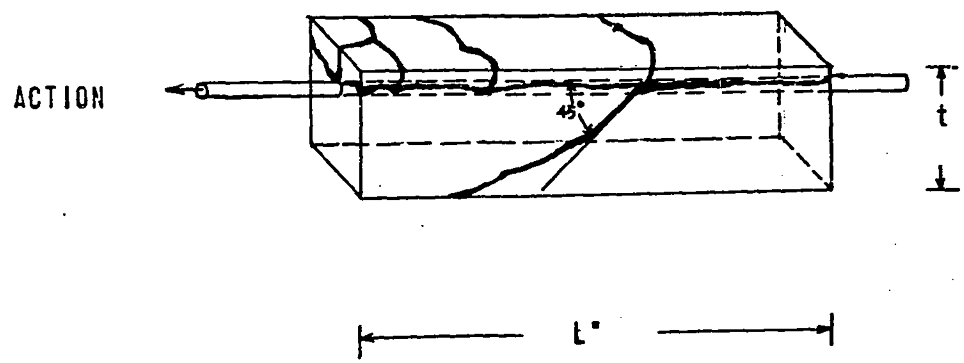


FIG. 6 B. F. & C. S. (D. T.) (DEFORMED BAR)

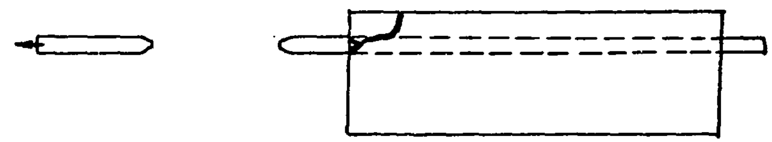


FIG. 8 S. F. & C. S. (DEFORMED BAR)

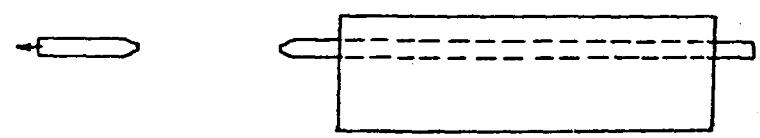


FIG. 7. S. F. (DEFORMED BAR)

concrete on the inside of the hook showing signs of crushing. Provision of hooks caused a considerable improvement in the apparent bond strength.

(ii) Deformed Bars with Stirrups:
These specimens failed at slightly higher loads than their counterparts without any stirrups. However, the width of the longitudinal and transverse cracks was much smaller than those noticed in specimens without stirrups.

2.4 TEST RESULTS AND DISCUSSION

The specimens details and the test results are indicated in Tables 1 through 12 in Appendix A. The effects of different parameters were noted as follows:

(A) EFFECT OF CONCRETE STRENGTH (f'_c)

There are two schools of thought on the effect of concrete strength on the bond resistance[8]. There are those who suggest that bond is frictional in nature and therefore the compressive strength of concrete has very little effect on it. According to Glanville,[24] the shrinkage and temperature changes in an unloaded reinforced concrete beam produce relative strains and hence bond stresses between steel and concrete. Since the shrinkage of concrete depends on the water-cement ratio and the environmental conditions and since the water-cement ratio also controls the concrete strengths, it can be argued that there would be some relationship between the concrete strength and the bond resistance. The ACI Committee 408 states: "When plain bars without surface deformations were used, bond was often thought of as chemical adhesion between concrete paste and bar surface." In the case of deformed bars they stated: "The 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 pull-out 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". Since splitting was a tension phenomenon, the Committee 408 considered the ultimate bond stress to vary approximately as the square root of f'_c as did the modulus of rupture. Dr. Chinn[1] had informally reported a better correlation with $(f'_c)^{0.7}$.

The M.I.T. results indicate an increase in the wire bond resistance with an increase in the concrete strength[8]. Figure 9 indicates the variation of the bond coefficient X with the concrete compressive strength for D2 type reinforcing bars while Figure 10 represents a plot of the coefficient $Y = u/(f'_c)^{0.7}$ against f'_c for the same steel bars. It appears that the parameter X is fairly constant with f'_c for different embedment length-diameter (L"/D) ratios and therefore indicates that the average ultimate bond stress u is more closely a linear function of $(f'_c)^{0.5}$ rather than $(f'_c)^{0.7}$.

(B) EFFECT OF DEVELOPMENT LENGTH (L")

The variation of bond coefficient X with embedment length (L") is shown in Figures 11 through 16 for different wire sizes, and leads to the following observations:

(i) The bond strength varies non-linearly with the increase in the development length.

(ii) The apparent average ultimate bond stress for deformed wires appears to increase with the increase in development length L" up to L" values listed below:

Type D-2, critical L" = 2.24 in. (57.0 mm) (L"/D = 14.1)
Type D-2.5, critical L" = 2.5 in.(63.5 mm) L"/D) = 14.1)
Type D-3, critical L" = 3.5 in. (89.0 mm) (L"/D = 17.9)
Type D-3.5, critical L" = 4.9 in. (124.4 mm) (L"/D = 22.8)
Type D-4, critical L" = 3.4 in. (86.4 mm) (L"/D = 15.1)

The variation of steel tensile stress with the L"/D ratio at ultimate load is shown in Fig. 17. It appears that the bar D-3.5 has a higher yield point and ultimate strength which matches the tensile stress obtained at ultimate load in the eccentric pull-out tests. (Fig. 18,19) This may perhaps explain the high critical L"/D ratio obtained for D-3.5 wires in these pull-out tests. These observations do not generally agree with the findings of other investigators[1,8,12,13,15,17,25,26]. These variations can possibly be explained as follows:

(i) Very short development length for steel bars in prototype bond specimens have generally not been investigated.

(ii) Investigation of bond in reinforced concrete models with the exception of the Cornell tests[9] have generally been on specimens

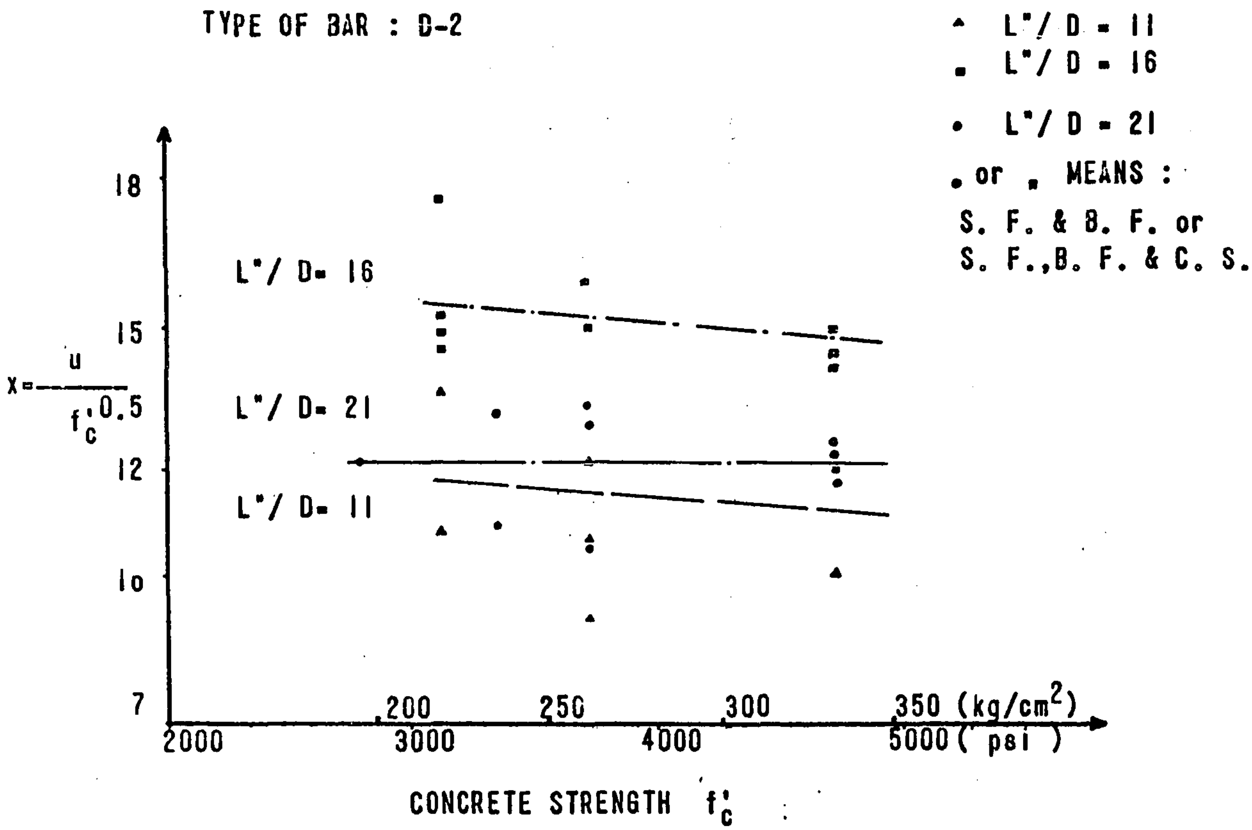


FIG. 9 VARIATION OF BOND COEFFICIENT WITH CONCRETE STRENGTH IN PULL-OUTS

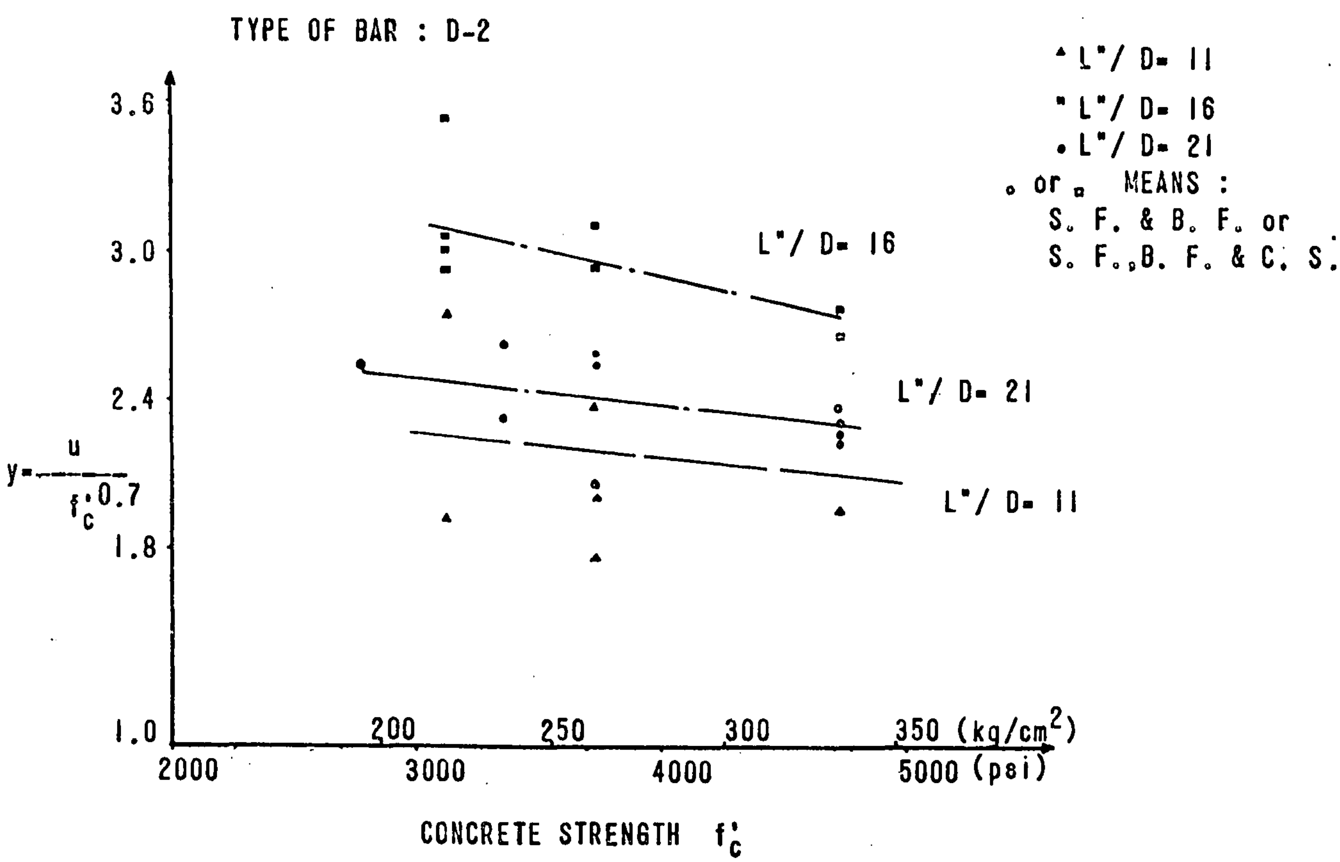


FIG. 10 VARIATION OF BOND COEFFICIENT WITH CONCRETE STRENGTH

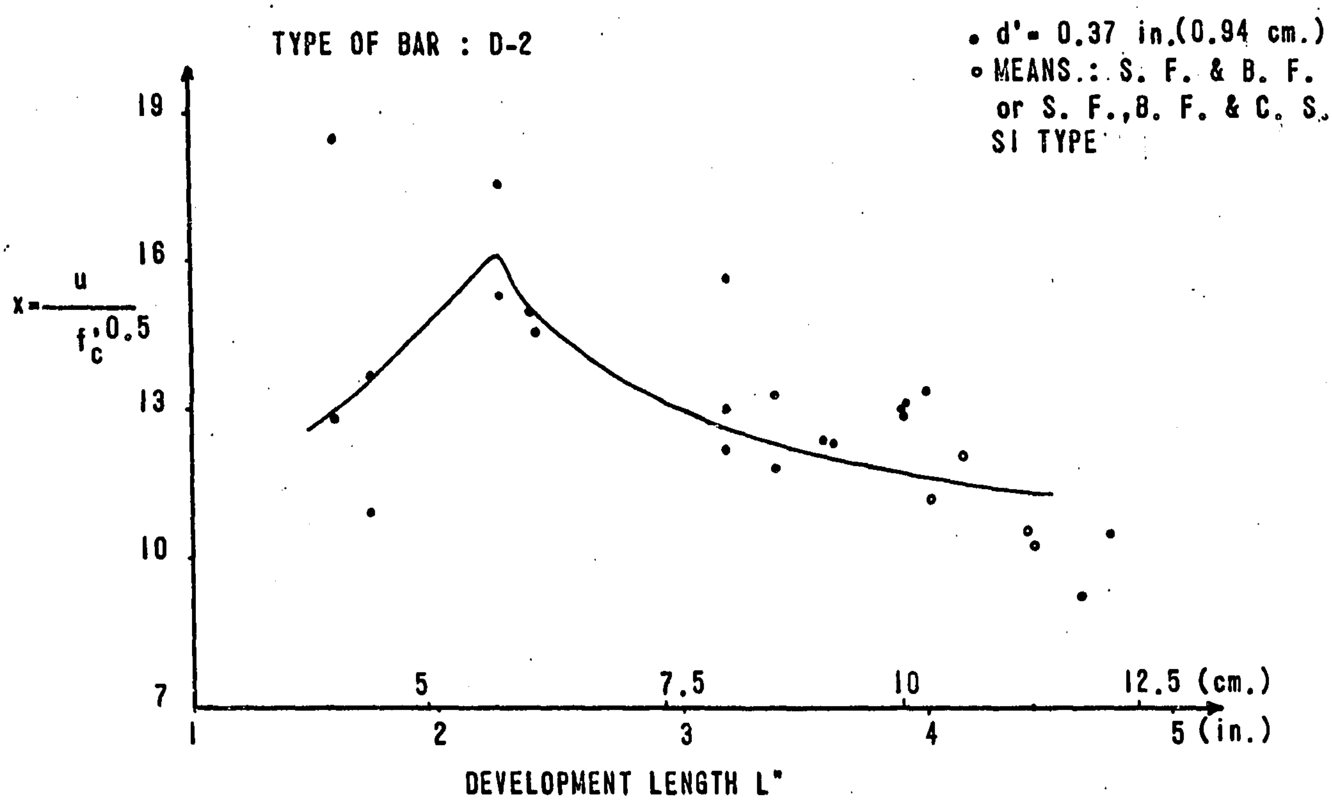


FIG. II VARIATION OF BOND COEFFICIENT WITH DEVELOPMENT LENGTH IN PULL-OUTS

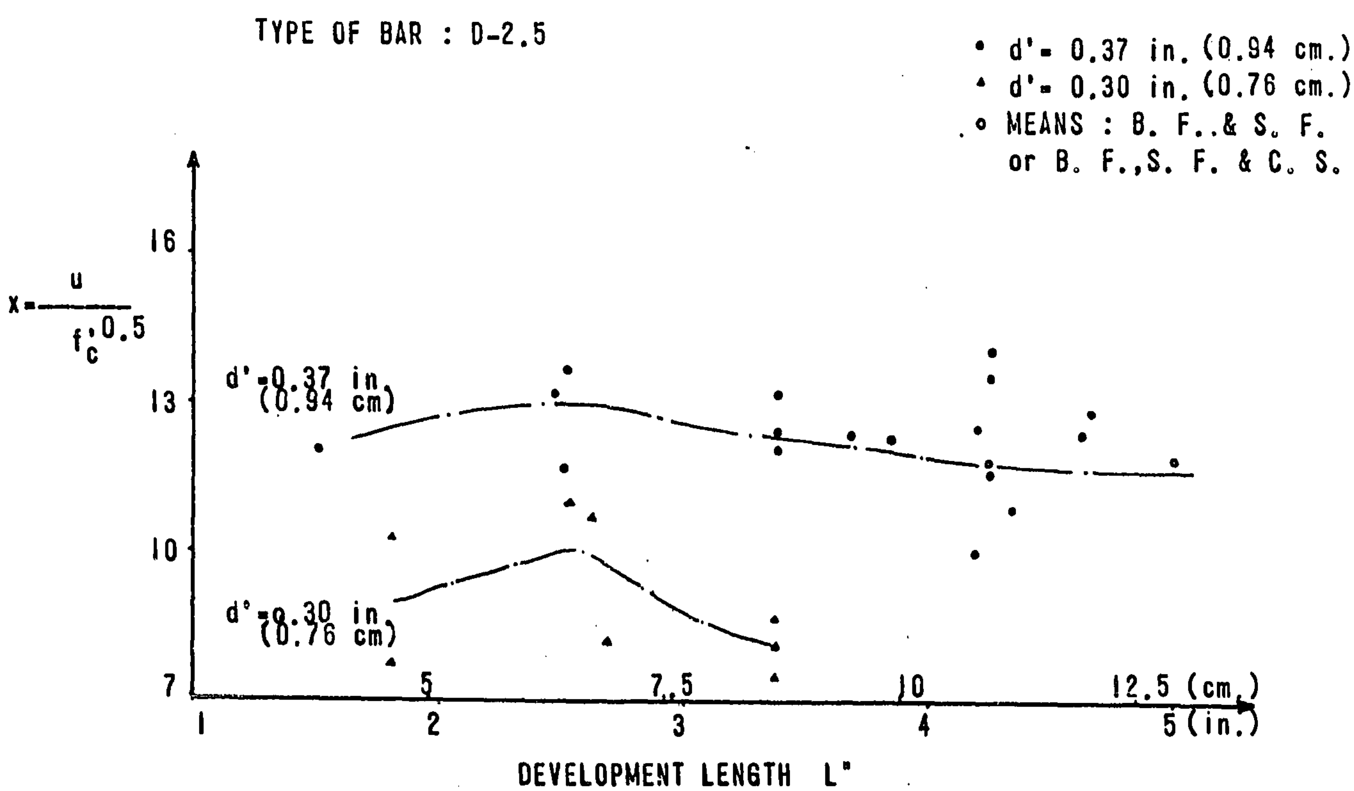


FIG. 12 VARIATION OF BOND COEFFICIENT WITH DEVELOPMENT LENGTH IN PULL-OUTS

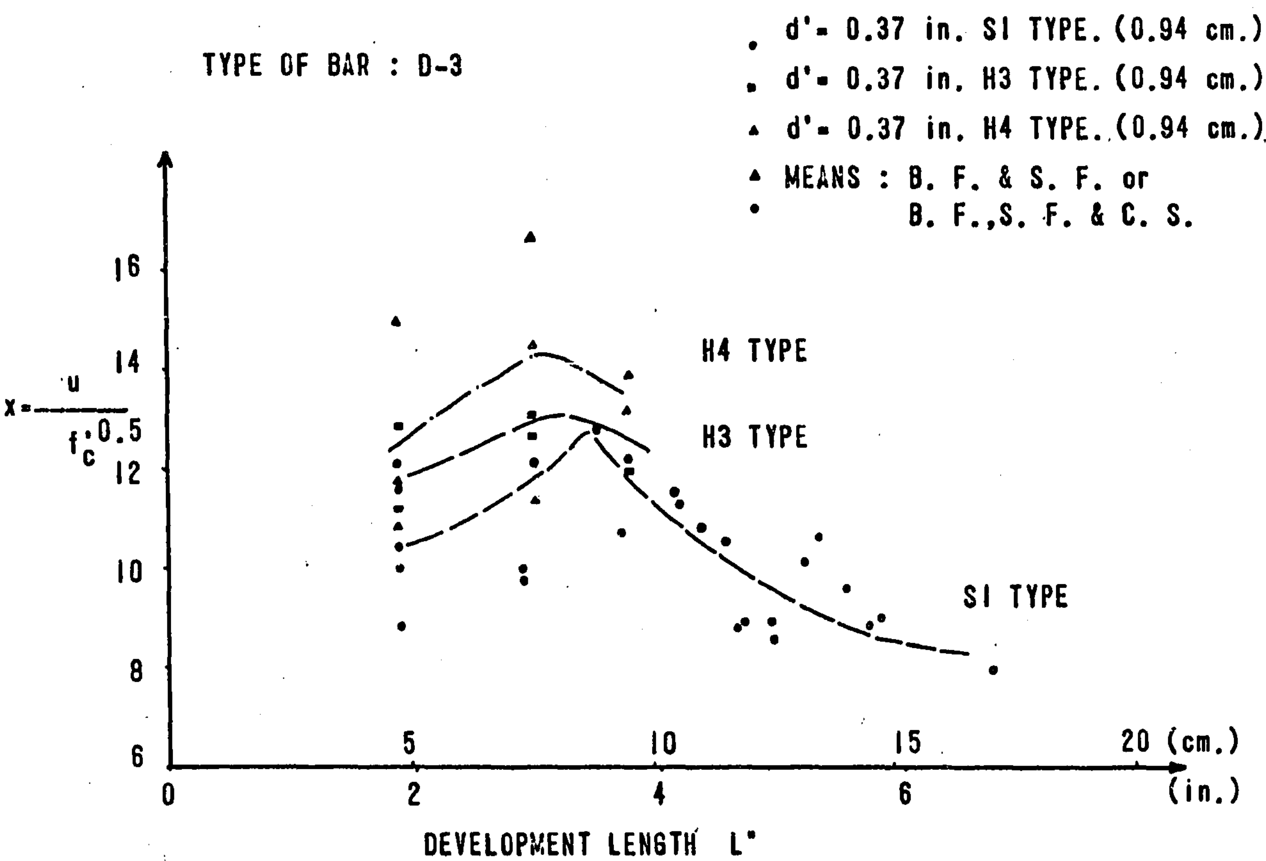


FIG. 13 VARIATION OF BOND COEFFICIENT WITH DEVELOPMENT LENGTH IN PULL-OUTS

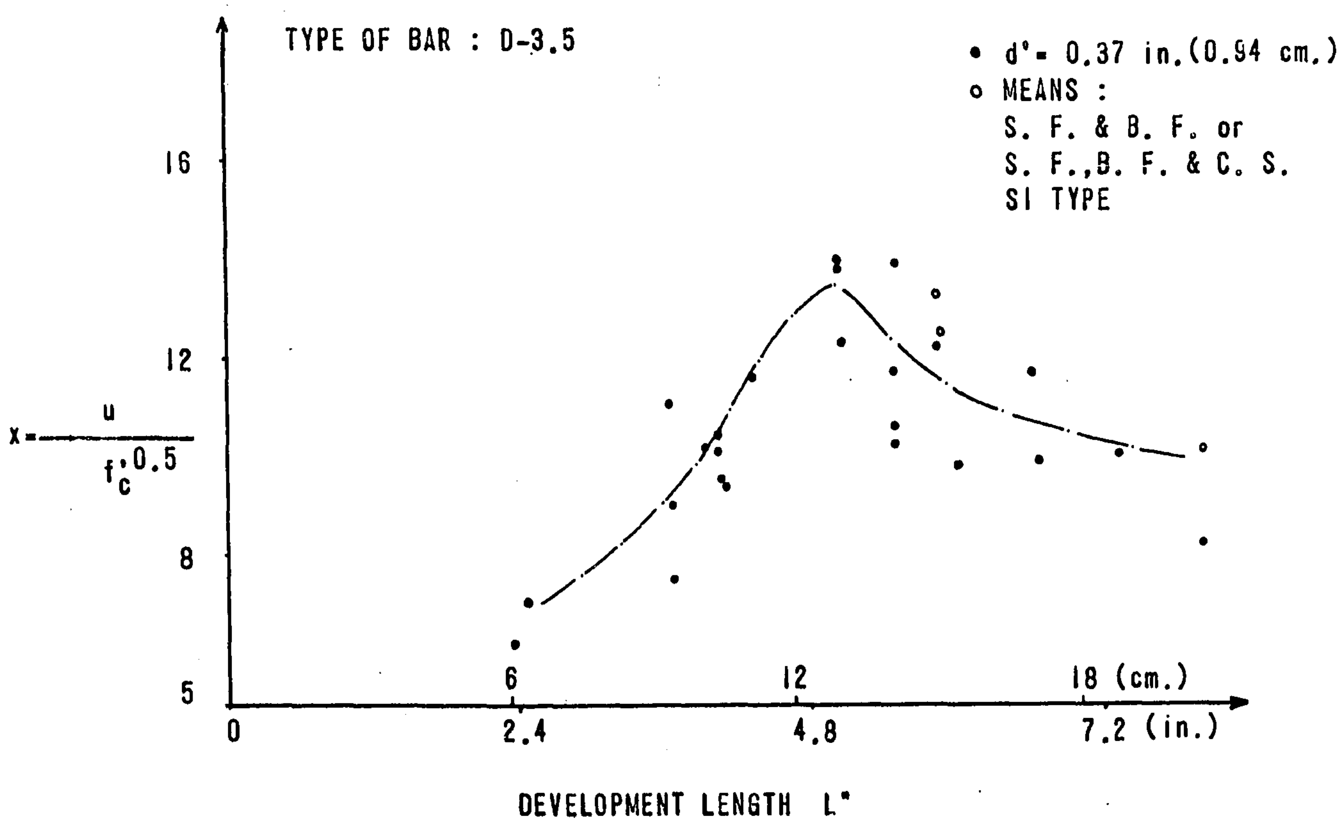


FIG. 14 VARIATION OF BOND COEFFICIENT WITH DEVELOPMENT LENGTH L" IN PULL-OUTS

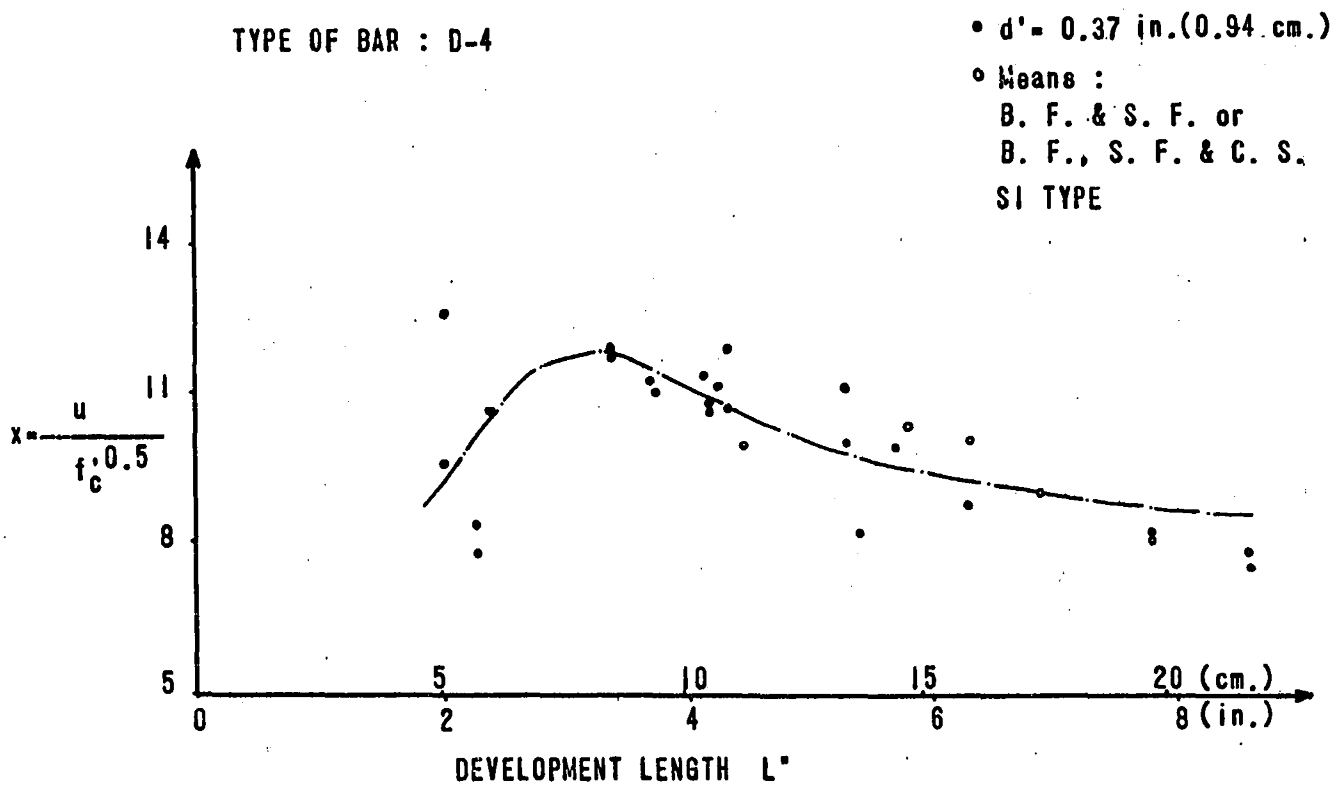


FIG. 15 VARIATION OF BOND COEFFICIENT WITH DEVELOPMENT LENGTH IN PULL-OUTS

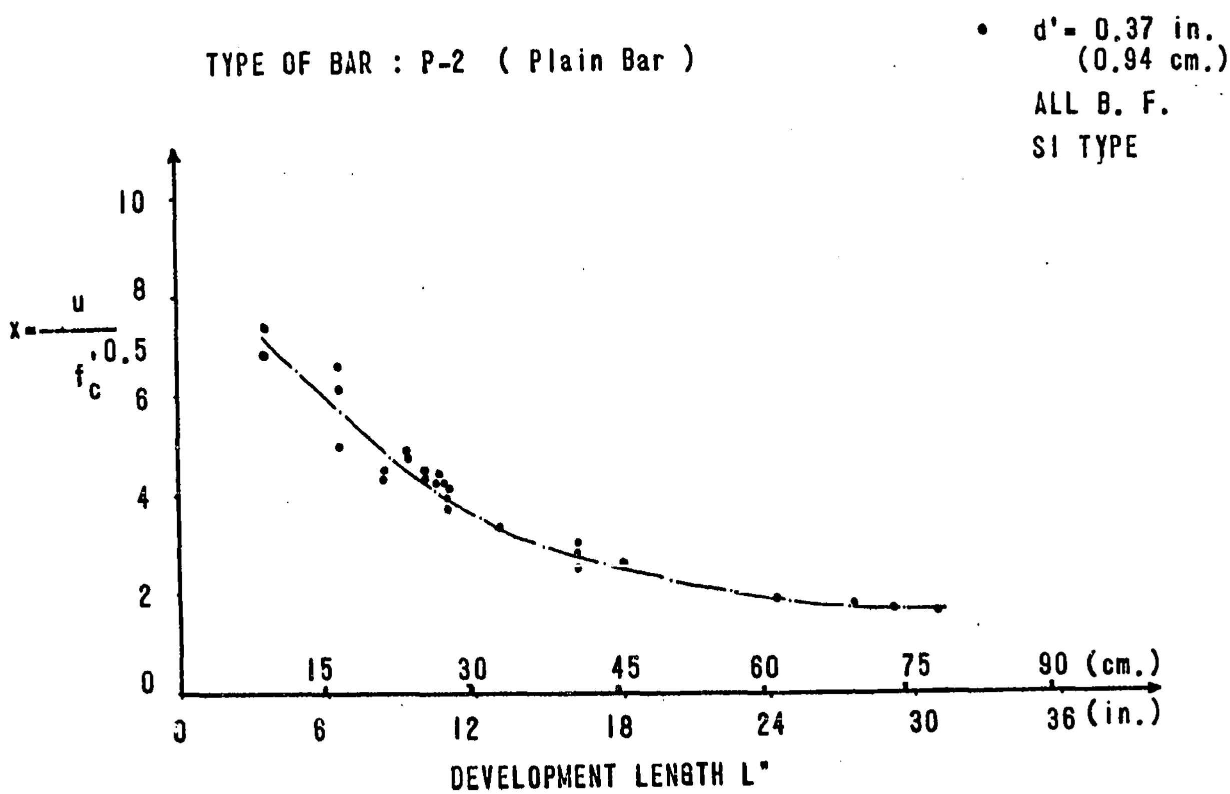


FIG. 16 VARIATION OF BOND COEFFICIENT WITH L" IN PULL-OUTS

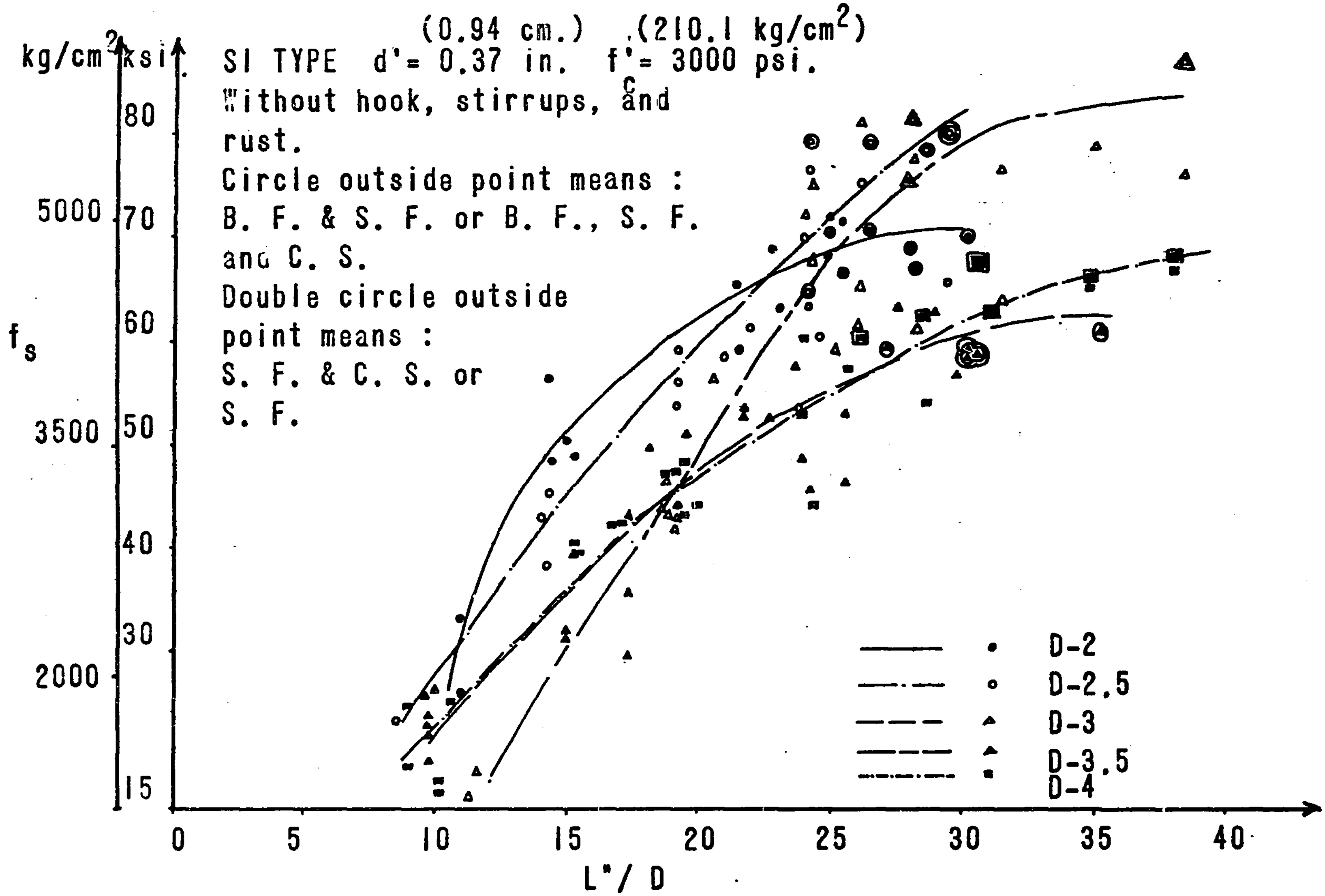


FIG. 17 VARIATION OF STEEL STRESS WITH L"/D IN PULL-OUTS

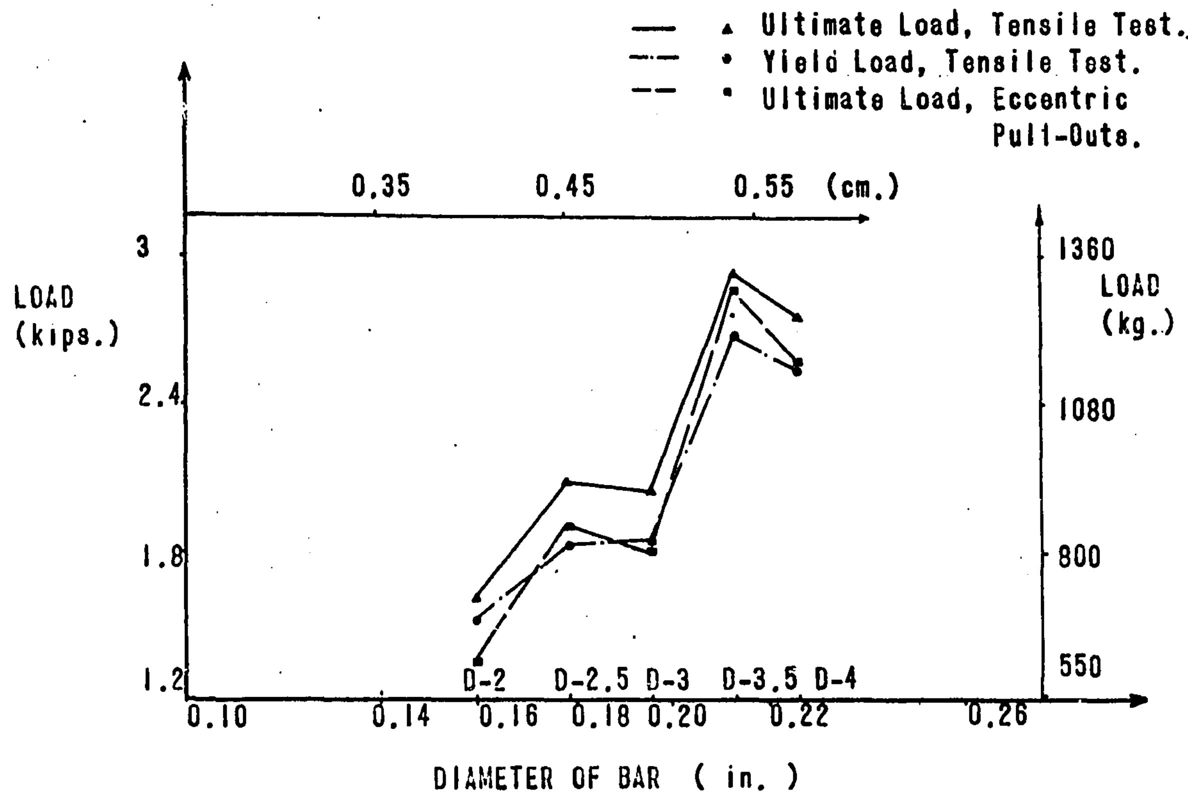


FIG. 18 THE LOAD COMPARSION OF STEEL FRACTURED BY TENSILE TEST AND BY PULL-OUT TEST

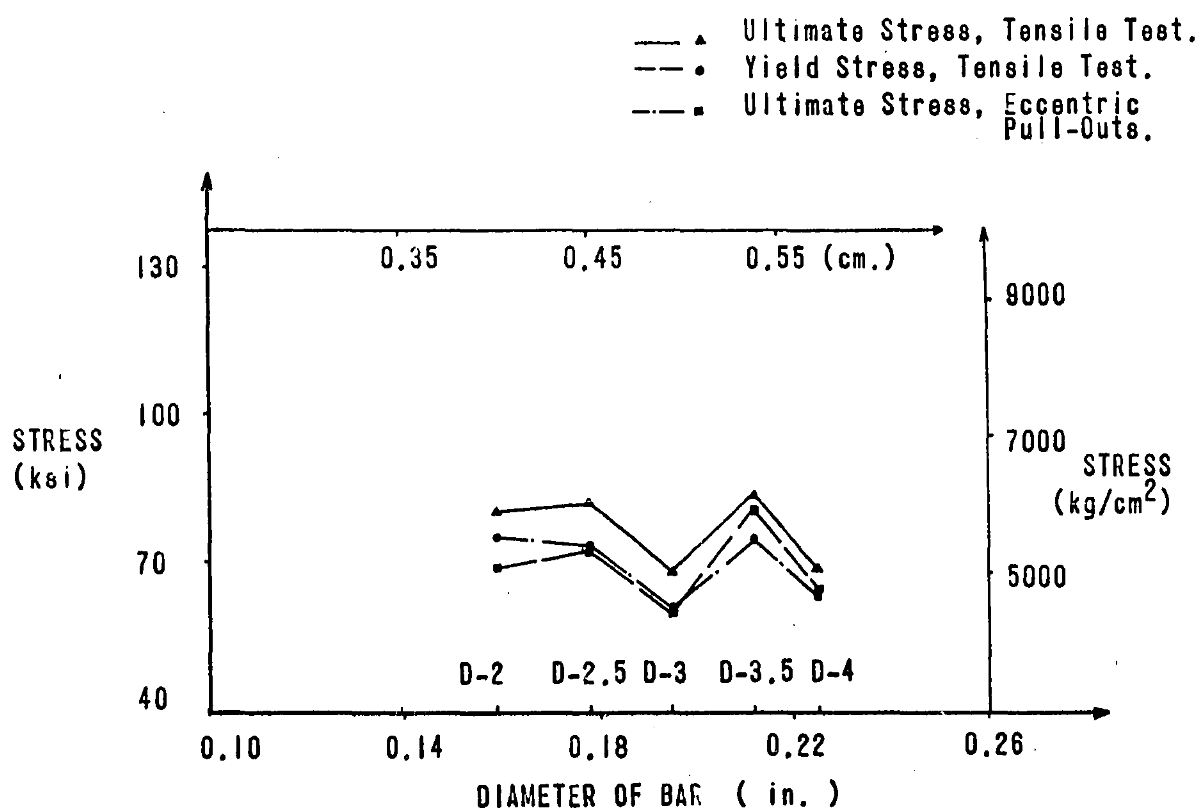


FIG. 19 THE STRESS COMPARSION OF STEEL FRACTURED BY TENSILE TEST AND BY PULL-OUT TEST

with L"/D ratios equal to and greater than 20.

(iii)According to the 1963 ACI Code, the ultimate bond stress u is given by

$$u = \frac{f_s}{4(L''/D)} = \frac{Df_s}{4L''} \qquad (3)$$

This equation suggests that the ultimate bond stress would increase with a decrease in the development length and would be theoretically infinite as the embedment length becomes smaller and smaller and approaches zero. However, this is incompatible with the physics of the problem - it is not possible to develop any bond resistance which for this case i.e. L"=0 must be zero. The non-linear variation of the average ultimate bond stress with the development length can be expressed by the equation

$$u = \sum_{k=1}^{k=n} A_k (L'')^k \qquad (4)$$

or

$$\frac{u}{\sqrt{f'_c}} = \sum_{k=1}^{k=n} B_k \left(\frac{L''}{D}\right)^k \qquad (5)$$

where A_k and B_k are real constants and n is a positive integer. Both these equations lead to curves passing through the origin. It may be noted that the flexural moment capacity of a given section is a problem in three dimensions with the tensile and compressive forces being developed at the cross-section. The tensile force is gradually developed along the length of the bar from zero value at the support to the required value at the section. Therefore, the equations for bond stress should in general represent an interaction between the bond forces, the shearing forces and the bending moments. These equations can be expected to be quite complicated and would represent a completely different philosophical approach to the problem.

(C) EFFECT OF BAR DIAMETER (D)

A plot of bond coefficient X against the bar diameter (D) is shown in Fig. 20 for different L"/D ratios. The coefficient X is observed to decrease linearly as the bar size increases from D-2 to D-3.5. The slope of these lines appears to increase with a decrease in the L"/D ratio (Fig. 20). A similar decrease though non-linear was observed in the M.I.T.

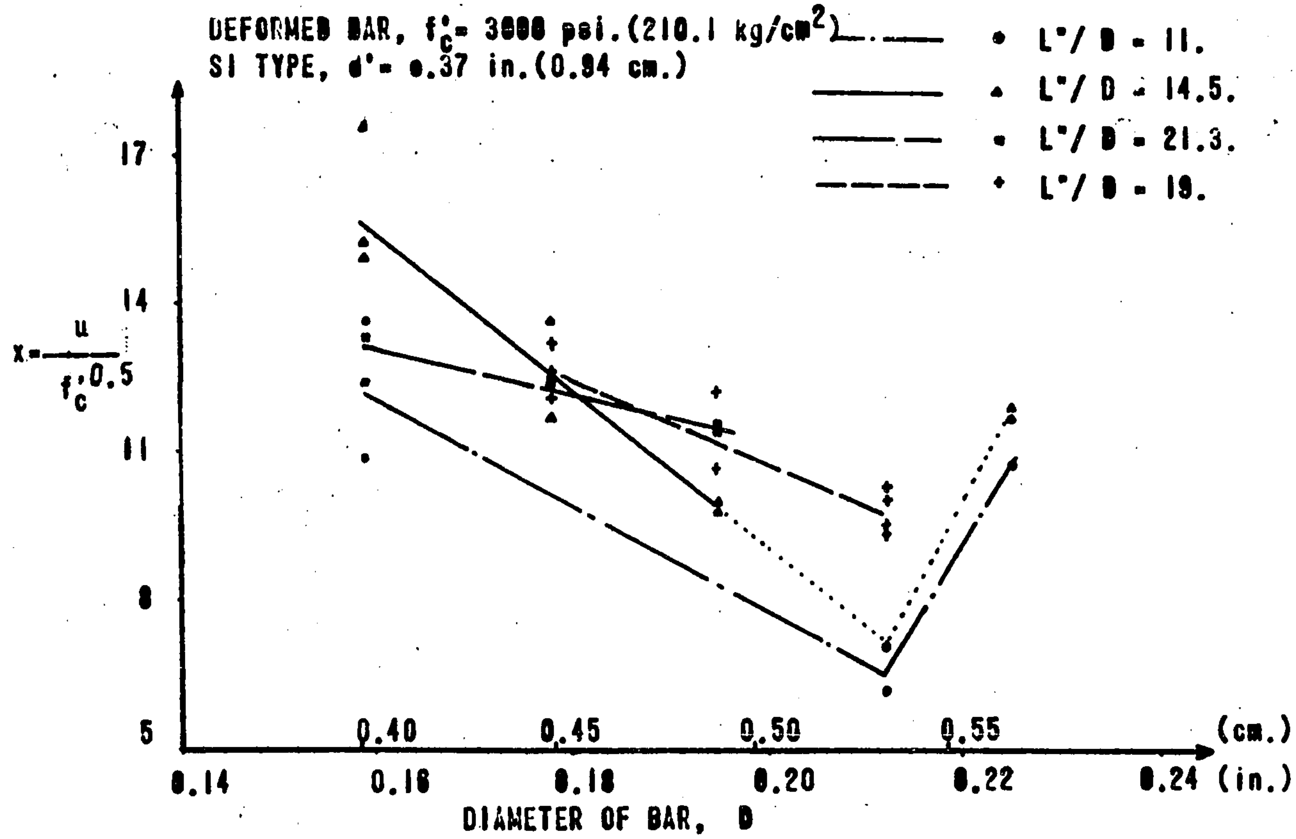
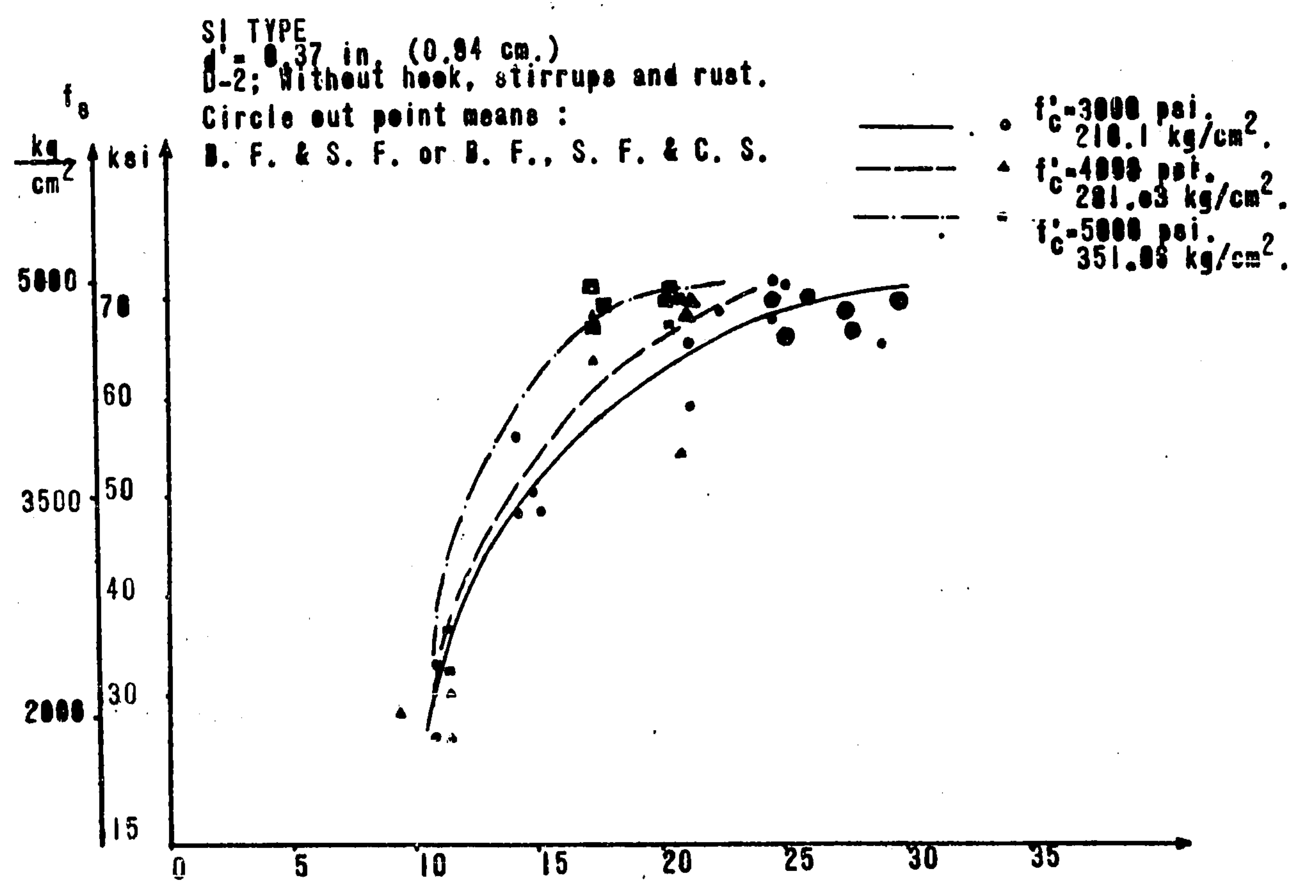


FIG. 20 VARIATION OF BOND COEFFICIENT WITH BAR DIAMETER IN PULL-OUTS



bond beam experiments with wire size varying between 0.063 in. (1.60 mm) and 0.135 in. (3.43 mm) diameters. Three specimens reinforced with D-4 bars tested for L"/D values of 14.5 and 11 indicate the pattern of decreasing values of coefficient X with an increase in bar size up to D-3.5 and then show a sudden increase as the bar size changes from D-3.5 to D-4. This variation cannot be explained from three tests and more experiments with bar sizes of D-4 and above will have to be undertaken to establish any possible trends.

(D) EFFECT OF STEEL STRENGTHS AND TYPE OF BARS (DEFORMED AND PLAIN)

The ultimate tensile strength, the yield strength and the load in the bar at failure are plotted against bar diameter in Fig. 18. Figure 19 shows the same data plotted in terms of stresses against the bar diameter. It is seen that in specimens which showed steel rupture the tensile strength at ultimate load was generally lower than the ultimate tensile strength of the steel wire. This is because corrosion of steel decreased the cross-sectional area of the bar but increased its surface roughness.

The tensile stress in the test specimens at ultimate load are plotted against L"/D ratio in Fig. 17. It is seen that

(i) The steel stress that can be developed in the wires at ultimate load appears to increase with the L"/D ratios.

(ii) The development length-bar diameter ratio at which steel wires ruptured appears to lie between 25 and 30.

Figure 21 indicates variation of steel stress at failure against L"/D ratios for different concrete strengths. It appears that a decrease in the concrete compressive strength from 5000 psi (352 kgm/cm^2) to 3000 psi (211 kgm/cm^2) appears to increase the L"/D ratio at which steel rupture appears to increase from approximately 12 to about 25.

Figure 22 indicates the variation of steel stress with L"/D ratio for plain bars. It appears that beyond L"/D=60 there is very little increase in the steel stress even for L"/D ratios greater than 190. No steel rupture was obtained in any of the tests on specimens with plain bars. In case of plain bars, bond is normally considered to consist of chemical adhesion between concrete paste and the bar surface which is generally not rough with cold drawn steel wires. As suggested by the ACI Committee 408, low bar stress causes slip sufficient to break the adhesion immediately adjacent

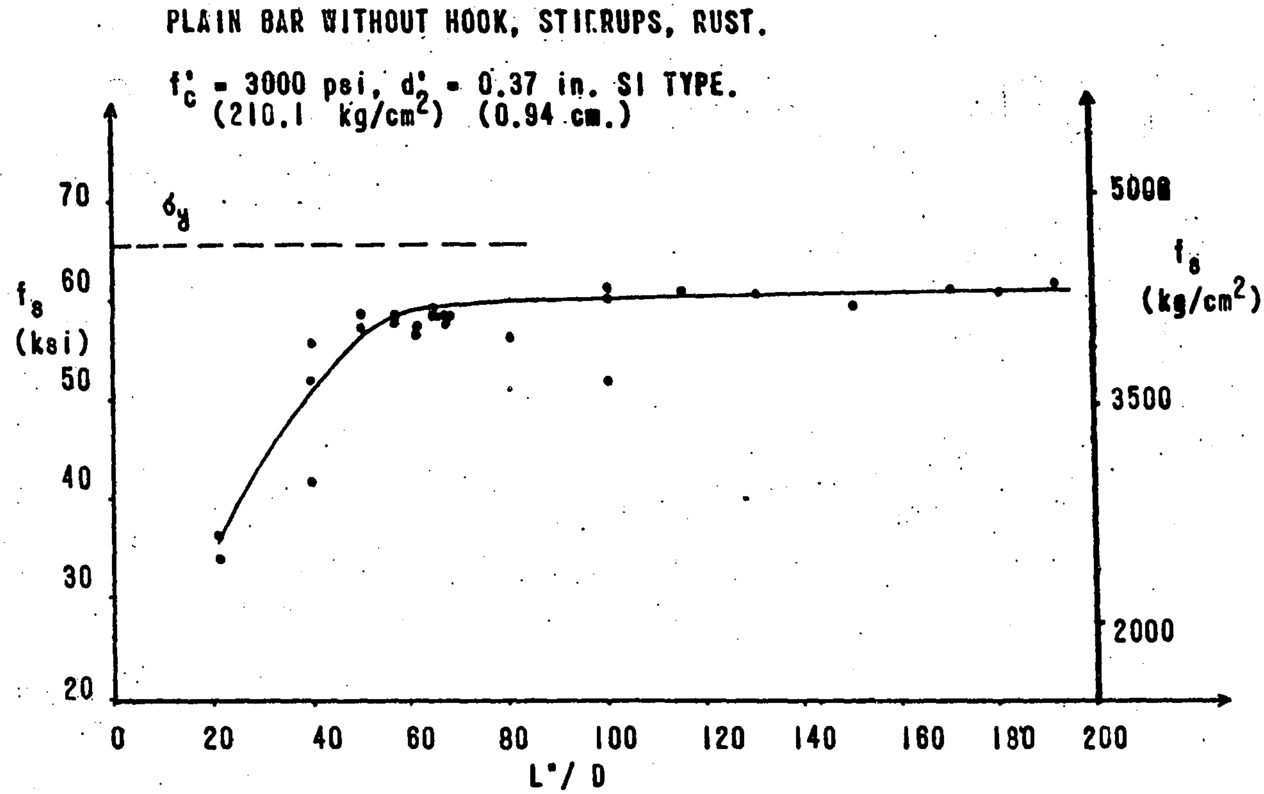


FIG. 22 VARIATION OF STEEL STRESS WITH L"/ D IN PULL-OUTS

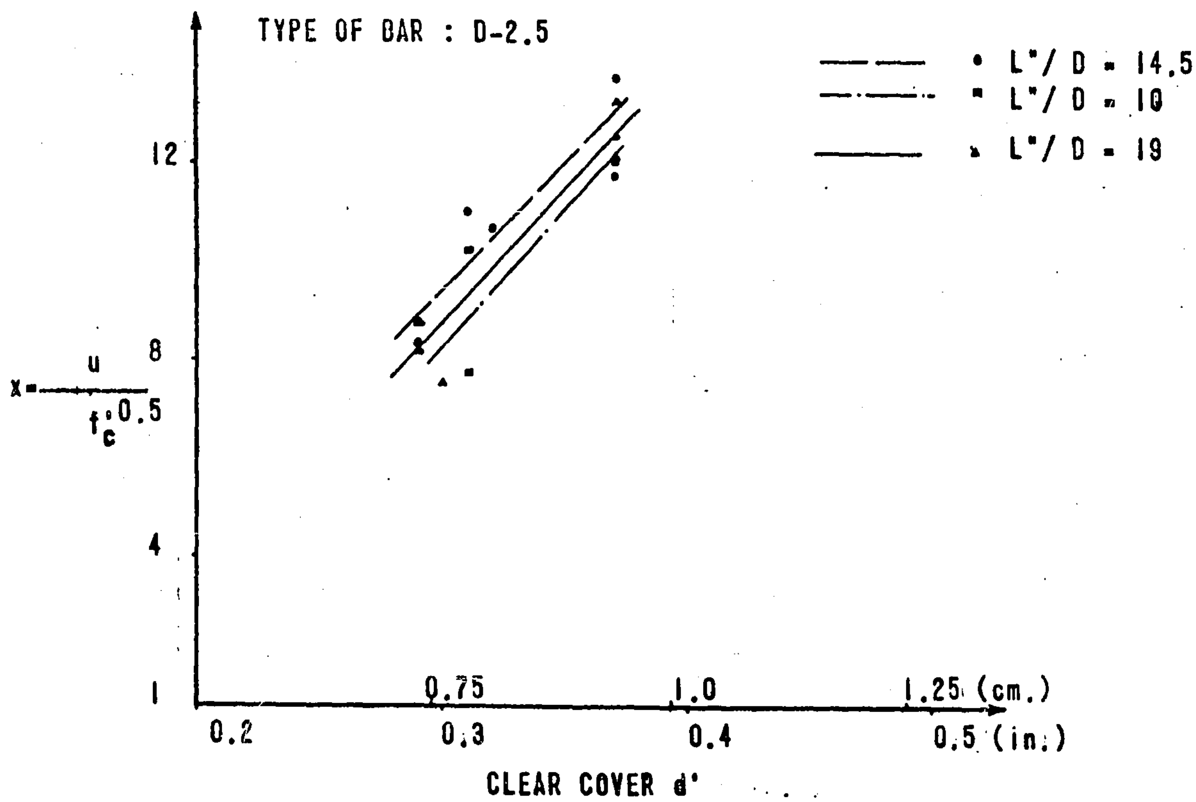


FIG. 23 VARIATION OF BOND COEFFICIENT WITH d' IN PULL-OUTS

to the loaded end. This slip is further assisted by the increased steel strains near the ultimate load and traverses the entire embedment length of the bar resulting in the bar pull-out for any embedment length.

(E) EFFECT OF CLEAR COVER (d')

Figure 23 indicates the variation of bond coefficient X with the clear cover d' for different L"/D ratios for D-2.5 deformed bars. Significant increases in the coefficient X are noted for L"/D ratios of 10, 14.5 and 19 with an increase in the clear cover.

(F) EFFECT OF END ANCHORAGE (HOOK)

Figure 13 presents the variation of bond coefficient X with the development length L" for bars with no end anchorage and the H3 and H4 type anchorages (Fig. 3.). The end anchorages appear to increase the bond strength for a constant development length. As expected, the H4 type hook appears to cause a greater improvement in bond strength than the simple right angled H3 type hooks. Tables II.1 and II.2 present the values of the ultimate bond stress u and the bond coefficient X for different embedment lengths and different types of hooks. H4 type hooks are noted to be most efficient in increasing the bond strength followed by H3 and H5 type hooks. The calculations of the average ultimate bond strength were based on an embedded length L" without the hook. As expected, the percentage efficiency of the hooks in increasing the bond strength decreased with an increase in the development length.

(G) EFFECT OF VERTICAL STIRRUPS

The addition of vertical closed stirrups to the main reinforcement slowed the propogation of tension cracks, and also decreased the crack width besides augmenting the apparent bond strength. This phenomenon had previously been noted by other investigators[1,25,27]. The bond strength increase is apparent from the results listed in Table II.3.

(H) EFFECT OF RUST

Table II.4 shows that for a constant development length L" and a constant clear cover d', the rusted bars exhibit strength increases of about 12 to 17 per cent over the unrusted bars. The comparison of ultimate loads in specimens exhibiting steel rupture with the ultimate tensile strength of steel has already been discussed in section (D) (Effect of Steel Strength).

TABLE II.1

COMPARISON OF BOND STRENGTHS

STRAIGHT AND HOOKED BARS

(DEFORMED BAR D-3)

L" approx. in.	Type of Specimen	d' in.	u psi	$X = \frac{u}{f'^{0.5}_c}$	Average X	Average X-S1=K	K/S1 %
1.88	S1	0.37	704 685 514 609 635 583	12.10 11.79 8.84 10.46 10.92 10.01	10.890	0	0
	S1*	0.35	518 474	9.60 8.78	9.190	-1.700	-15.60
	H5	0.37	661 748 418	12.24 13.86 7.75	11.280	+0.390	+ 3.60
	H3	0.37	707 615	12.97 11.30	12.140	+1.250	+11.50
	H4	0.37	644 875 648	11.81 15.05 11.16	12.673	+1.783	+16.40
2.90	S1	0.37	537 541 666	9.84 9.92 12.23	10.660	0	0
	S1*	0.35	556 549 549	10.30 10.16 10.16	10.310	-0.350	- 3.30
	H3	0.37	700 712	12.83 13.05	12.940	+2.280	+21.40
	H4	0.37	814 639 940	14.52 11.41 16.80	14.243	+3.573	+33.51
3.74	S1	0.37	697 664 582	12.90 12.29 10.78	11.960	0	0
	H3	0.37	590 685	13.24 12.22	12.730	+0.780	+ 6.52
	H4	0.37	776 672	13.86 13.13	13.495	+1.535	+12.80

TABLE II.2

COMPARISON OF BOND STRENGTHS

STRAIGHT AND HOOKED BARS

(DEFORMED BAR D-4)

L" approx. in.	Type of Specimen	d' in.	u psi	$X = \frac{u}{f_c'^{0.5}}$	Average X	Average X-S1=K	K/S1 %
4.25	S1 PDS1N-8 PDS1O-8 PDS1Z-8	0.37	607 561 632 464 611 632 567	10.81 10.00 11.24 11.48 10.85 12.10 10.85	11.05	0	0
	H4 PDH4Z-18	0.37	615 845	11.78 16.18	13.98	+2.93	+26.5

TABLE II.3

COMPARISON OF BOND STRENGTHS

STRAIGHT BARS WITH OR WITHOUT

VERTICAL CLOSED STIRRUPS

Type of bar	L" approx. in.	Type of Specimen	d' in.	u psi	$X = \frac{u}{f_c'^{0.5}}$	Average X	Average X-S1=K	K/S1 %	No of Stirrups
D-4	4.25	S1 PDS1N-8 PDS1O-8 PDS1Z-8	0.37	607	10.81	11.05	0	0	0
				561	10.00				
				632	11.24				
				646	11.48				
				611	10.85				
				632	12.10				
				567	10.85				
		PDS2Z-16 S2	0.37	704	13.50	13.54	+2.49	+22.5	6
				710	13.58				
	1.59	S1 PDS1G-6 PDS1Y-6	0.37	612	10.95	14.01	0	0	0
				765	13.70				
				706	12.90				
				1006	18.50				
		S2 PDS2Y-21	0.37	1013	18.65	19.33	+5.32	+37.9	2
				1088	20.00				
D-2	3.18	S1 PDS1Y-8	0.37	663	12.20	13.69	0	0	0
				709	13.02				
				856	15.75				
		S2 PDS2Y-22	0.37	825	15.16	14.83	1.14	+ 8.3	5
				869	15.95				
				728	13.38				

TABLE II.4

COMPARISON OF BOND STRENGTHS
STRAIGHT BARS
(RUSTED AND UNRUSTED)

Type of bar	L" approx. in.	Type of Specimen	d' in.	u psi	$X = \frac{u}{f_c'^{0.5}}$	Average X	Average X-S1=K	K/S1 %	No of Stirrups
D-4	4.25	S1 PDS1N-8 PDS1O-8 PDS1Z-8	0.37	607	10.81	11.05	0	0	-
				561	10.00				-
				632	11.24				-
				646	11.48				-
				611	10.85				-
				632	12.10				-
				567	10.85				-
		S1 PDS1Y-17 (R)	0.37	728	13.40	12.91	+1.86	+16.83	-
				666	12.24				-
				713	13.10				-
D-2	3.4	S1 PDS1A-9 PDS1D-9 PDS1Y-8	0.37	655	12.40	13.08	0	0	-
				684	11.80				-
				770	13.30				-
				663	12.20				-
				709	13.02				-
				856	15.75				-
		S1 PDS1Y-23	0.37	852	15.63	14.64	+1.58	+12.1	-
				875	16.07				-
				665	12.22				-

CHAPTER III

INVESTIGATION OF BOND IN REINFORCED CONCRETE MODELS

-BOND BEAM TEST-

3.1 INTRODUCTION

Bond beam specimens were first used to investigate bond in flexural members at the National Bureau of Standards[1,12,14,28] and the University of Texas[1,12,25,26]. The National Bureau specimens (called BS) consisted of a simply supported beam with a constant moment region while those of University of Texas (called UT) provided for overhanging beams with resulting negative moments over the supports. A recent development in this field is the "Cantilever Test Beam"[29], which is a modification of the eccentric pull-out test. Splitting of concrete is a common mode of failure in bond and is significantly influenced by support reactions. Therefore, the National Bureau of Standards developed special supports for beams which eliminated direct pressure from reactions on the bars under test. Ferguson[1] developed a negative bending moment region in their test beams and studied the bond characteristics of the tension at top thus eliminating the local pressure effects. ACI Committee 408[1] found that the results from the National Bureau of Standards and University of Texas tests agree very closely, in spite of the fact that the former used heavy stirrups while the latter used either no stirrups or very light ones.

The M.I.T. team used small sized specimens similar to University of Texas beams to investigate bond behaviour in reinforced concrete models[8]. McGill investigations of bond in reinforced concrete models besides using the concentric and eccentric pull-out tests (Chapter II), also employed bond specimens similar to those of Ferguson[1]. Along with the above test specimens, McGill also developed the symmetrical bond beam shown in Fig. 24. It indicates that the ends of the bars are subjected to local pressures from the concentrated loads P_u. This is a deviation from the National Bureau of Standards and University of Texas beams, however in practice most of the beams are subjected to concentrated reactions at the supports which perhaps represent the poorest bond conditions. The principal characteristics of the symmetrical bond beam (called SB) are as follows:

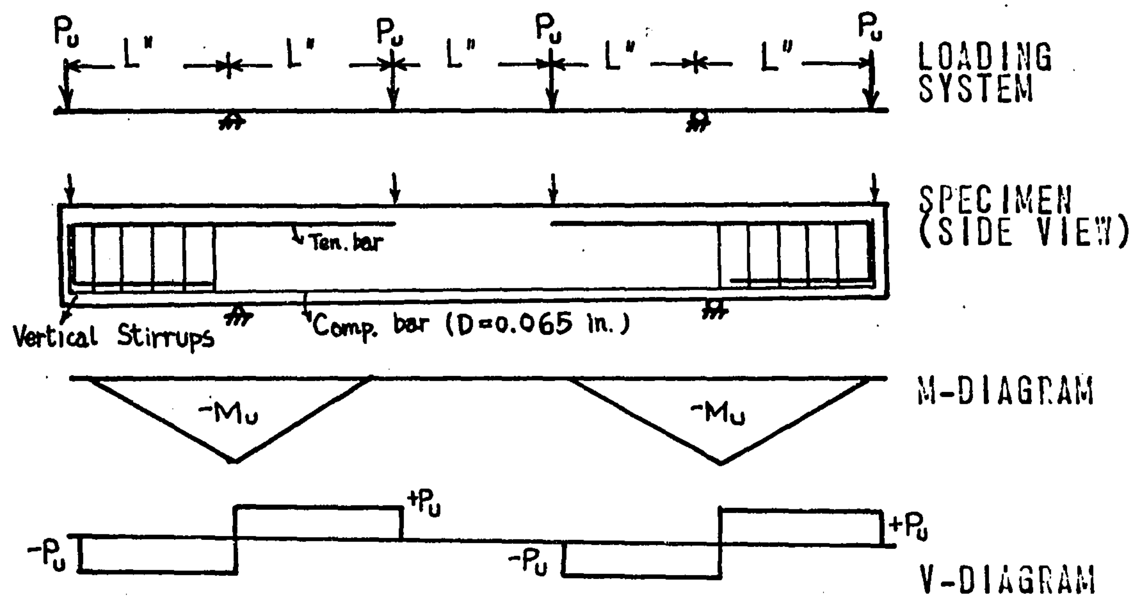


FIG. 24 SYMMETRICAL BOND BEAM SPECIMEN

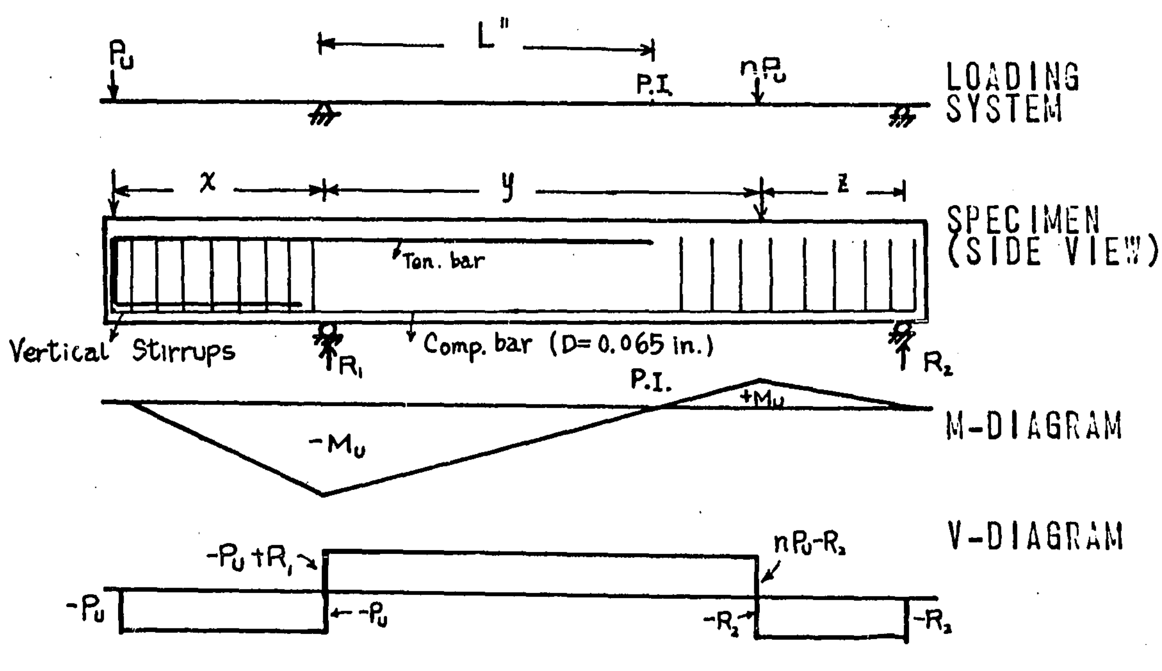


FIG. 25 UNIVERSITY OF TEXAS BEAM

(i) The SB specimens yield a region of known length L" over which the bending moment and shear force are zero. This resembles the free end conditions in the eccentric pull-out test.

(ii) The SB specimens provide for a symmetrical specimen and loading condition. It is easier to apply and control the four equal concentrated loads during the test unlike the UT beams which require evaluation of distances x and y for the two concentrated loads (which are generally not equal).

(iii)The SB specimens could possibly be used to study bond characteristics of reinforcing bars anchored in zones of zero moment and zero shear. This feature is not available in any other test specimens.

(iv) The cracks appear at the supports in the SB specimens and the UT specimens while there is not control over the location of cracks in the constant moment region of the BS beams. It is therefore possible to determine the actual development length used in a particular test beams.

3.2 THEORETICAL ANALYSIS

The details of models of UT beams is shown in Fig. 25 along with the bending moment and the shear force diagrams. The reactions R_1 and R_2 and the distance x were calculated from known values of ultimate load P_u, the constant n, the development length L" and distances y and z using the following equations:

$$R_2 = \frac{P_u(-x + ny)}{(y + z)} \tag{6}$$

$$R_1 = \frac{P_u\left(x+y+(n+1)z\right)}{(y + z)} \tag{7}$$

$$x = \frac{nzL''}{y + z - L''} \tag{8}$$

3.3 BOND STRESS EVALUATION
(Existing and Proposed Calculation Methods)

Equations for evaluation of bond stress are derived from two basic concepts:

(i) Anchorage or development bond (ACI Committee 408 suggested that anchorage and development bond are identical concepts).

(ii) Flexural bond.

According to the 1963 ACI Code[30] the average development bond stress u (or the average anchorage bond stress) is given by

$$u = \frac{f_s}{4(L''/D)} \qquad (3)$$

If the steel stress f_s were to approach the ultimate tensile strength f_u then equation (3) gives the average ultimate bond stress u_u.

$$u_u = \frac{f_u}{4(L''/D)} \qquad (3A)$$

The flexural bond stress which represents the local shear stress at the concrete-steel bar interface at a section is given by[29,30]

$$u = \frac{V}{\sum ojd} \qquad (9)$$

According to the ACI Committee 408[1] the average ultimate bond stress appears to be more significant than the local value at any specified point, particularly where the specimen has undergone flexural cracking. Each flexural crack creates points of bond stress concentration which influence the average usable stress. This investigation was aimed at finding the average ultimate bond stress rather than the flexural bond stress.

(A) EXISTING CALCULATION METHODS

An examination of equation (3) indicates that the average bond stress u is directly dependent on the steel stress f_s. The existing methods for evaluation of steel stress f_s are as follows:

(a) Current Method in WSD and USD:

The Working Stress Design method (WSD) has been used by several investigators and is based on the following assumptions:

(i) Strains are distributed linearly across the beam section.

(ii) Concrete and steel strains are within the elastic limit.

(iii) The modular ratio $n = \frac{E_s}{1000\ f'_c}$

(iv) Concrete does not resist any tension.

The steel stress f_s is then given by

$$f_s = \frac{M}{A_s\,jd}$$

Ferguson and Thompson[26] used a value of $\frac{7}{8}$ for the lever-arm factor j.

In the Ultimate Strength Design (USD) method, the steel stress is assumed to be equal to f_y[12]. Ferguson[12,26] suggested an approximate value of 0.9 for j for USD method which appeared to be justified for practical calculations. It may be noted that many of the beams in practice or experimental works are designed with nominal compression steel to support the stirrups, however the bond stress calculations ignore the compression reinforcement[8]. The area of compression steel used in test beams was kept as small as possible to decrease any influence of compression steel on calculated bond stresses.

(b) Strain Gauge Method:

Mains[13] cut a longitudinal slice using a precision band saw. The larger slice was then milled to provide a channel for strain gauges and lead wires. The channel was so cut as to allow the gauges to be mounted as near the centroid of the finished bar section as possible. Mains also provided the following procedure to construct bond stress curve[13]

(i) Plot force-in-bar values to a convenient scale and draw in a reasonably smooth curve between values.

(ii) Determine the difference in force-in-bar between two successive gauge points A and B. This difference was the total force developed in the interval A-B, or

$$\Delta F = F_B - F_A \qquad (10)$$

(iii)The average unit bond stress in the interval A-B was then

$$u_{ave} = \frac{F_B - F_A}{\pi D.\Delta L}$$

where ΔL is the length of the interval AB.

(iv) Determine unit bond stress values at points A and B (and intermediate maxima or minima)from the slope of curve in (i).

(v) Plot values of u_A, u_B, u_{ave} (and intermediate points when necessary).

(vi) Draw in the bond stress curve so that the particular points, u_A, u_B (and intermediate points) are contained and so that u_{ave} is achieved for the interval.

Perry and Thompson[18] used a test procedure similar to Mains, however they milled the bar into two semi-circular sections and then cut a groove in the flat surface of each half and installed electric resistance strain gauges. They obtained the distribution of steel stress and bond stress along the reinforcing bar in the eccentric pull-out test and in beams at a crack and at a bar cut-off.

(B) PROPOSED CALCULATION METHOD

The following two methods were used at McGill University to evaluate the bond stresses in the test beams:

(a) The effect of compression reinforcement was ignored in evaluating the bond stresses: According to the 1963 ACI Code[30],

$$c = 0.85 f'_c ba$$

$$T = A_s f_s \qquad (12)$$

where $$a = \frac{A_s f_s}{0.85 f'_c b}$$

Therefore

$$M = T(d - \frac{a}{2}) = A_s f_s (d - \frac{A_s\ f_s}{1.7\ f'_c\ b}) \tag{13}$$

$$f_s = \frac{1.7\ f'_c\ d - \sqrt{(1.7\ f'_c d)^2 - 6.8\ f'_c\ M}}{2\ A_s} \tag{13A}$$

The experimental values of M are substituted into equation (13) or (13A) which yields the value of steel stress f_s. This value of $\dot{f}_s$ is then substituted into equation (3) to obtain the average bond stress u.

(b) The effect of compression reinforcement was included: Author's previous investigation at College of Chinese Culture led to the following equation:

$$c = \frac{M}{bd^2} = pf_s(1 - \frac{f_s p - f'_s p'}{fc'} \cdot \frac{\Phi \mathcal{E}_{cn} - \Psi}{\Phi^2}) + p'f'_s(-\frac{d'}{d} + \frac{f_s\ p - f'_s p'}{f'_c} \cdot \frac{\Phi \mathcal{E}_{cn} - \Psi}{\Phi^2}) \tag{14}$$

where

$\mathcal{E}_c$ = Compressive strain in concrete from compression test.

$\mathcal{E}_{cu}$ = Compressive strain corresponding to concrete compressive stress f'_c.

$\mathcal{E}_{cn}$ = $\mathcal{E}_c / \mathcal{E}_{cu}$

Φ = $\int_0^{\mathcal{E}_{cn}} f(\mathcal{E}_{cn}) d\mathcal{E}_{cn}$ = Area of the normalized stress-strain curve for concrete.

Ψ = $\int_0^{\mathcal{E}_{cn}} f(\mathcal{E}_{cn})\, \mathcal{E}_{cn} d\mathcal{E}_{cn}$ = First moment of the normalized stress-strain curve area about the stress axis.

Note that if the compression reinforcement is absent or is to be ignored, the value of p' would be zero in equation (14)

Then

$$c = \frac{M}{bd^2} = pf_s\left(1 - \frac{f_s p}{f'_c} \cdot \frac{\Phi\varepsilon_{cn} - \Psi}{\Phi^2}\right) \qquad (15)$$

The value of M and the stress-strain characteristic of concrete leading to values of Φ , ε_{cn} and Ψ are obtained experimentally and substituted in equation (14) or (15) which yields the steel stress value f_s.

The 1963 ACI Code[30] gives the following equation for the flexural strength of beams with both tension and compression reinforcements,

$$M = (A_s f_s - A'_s f'_s)\left(d - \frac{a}{2}\right) + A'_s f'_s (d - d') \qquad (16)$$

where

$$a = \frac{A_s f_s - A'_s f'_s}{0.85\ f'_c b}$$

Equations (14) and (16) can be solved simultaneously to obtain the values of f'_s, however these calculations are generally very complicated, and "trial and error" approaches may be used to solve these equations.

(C) PROPOSED TRIAL AND ERROR METHODS

(i) Exact method: The following steps are involved in determining the steel stress values f_s and f'_s (see Fig. 26).

(1) Assume a suitable value for c. (This locates the neutral axis).

(2) From the experimentally known value of the ultimate compressive strain ε_u, the values of steel strains ε_s and ε'_s can be obtained from similar triangles.

(3) The values of steel stresses f_s and f'_s can be calculated from the strain values in (2).

(4) Using the equilibrium of moments and forces obtain the values of the concrete compressive force C and z as shown in Fig. 26.

(5) The location of the resultant compressive force is then determined which yields the value of the lever arm jd.

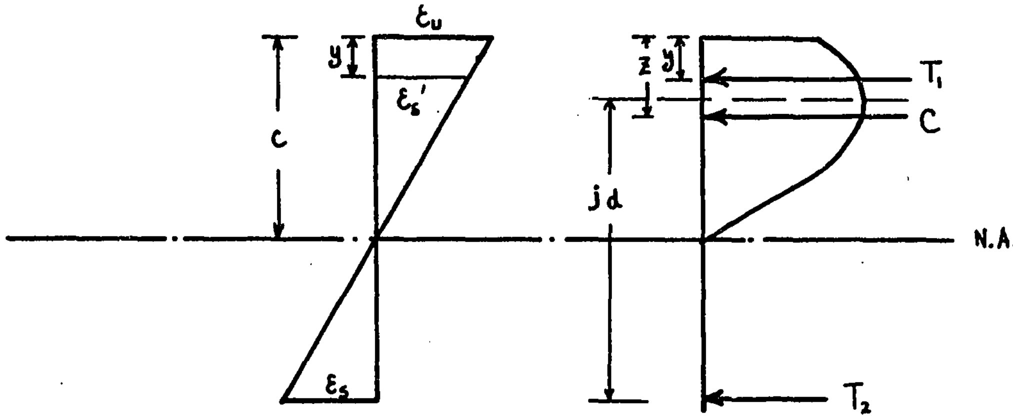


FIG. 26 STRAIN AND STRESS DISTRIBUTIONS:EXACT METHOD

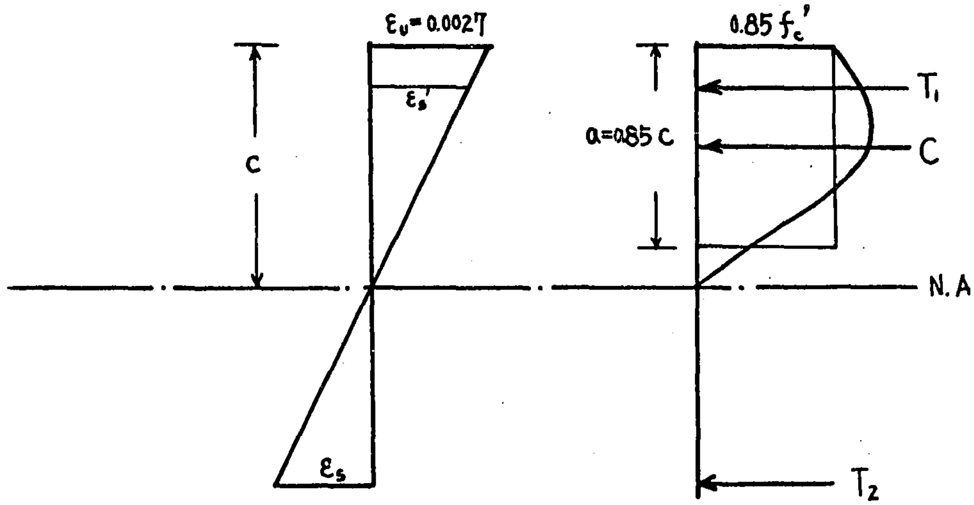


FIG. 27 STRAIN AND STRESS DISTRIBUTIONS:APPROX. METHOD

(6) The flexural strength of the section is then evaluated using the equation

$$M_{cal} = A_s f_s jd$$

(7) This value M_{cal} is then compared with the experimental value M_{test}. If the two values agree, then the assumed value of c was satisfactory. If not, another trial can be made with a new value of c. The f_s value can then be used for bond stress evaluation.

(ii) Approximate method: This method is identical to the exact method in (i) above with the following exceptions:

(1) The stress block is assumed to be rectangular in accordance with the 1963 ACI Code[30].

(2) The ultimate concrete compressive strain is assumed to be 0.0027[7] from the evaluation of microconcrete compressive test.

3.4 TEST PROGRAMME

This investigation consists of tests on 48 test specimens (30 specimens were similar to University of Texas Beams and 18 Symmetrical Bond Beams). Some of the beams failed in diagonal tension (called SDT in notations) before any bond distress was noticed. These specimens were not analysed for the bond stress value because the primary cause of failure was diagonal tension which was followed by an internal redistribution of forces. The bond stress in this case therefore depended on the effectiveness of the shear reinforcement. For such cases Taub and Neville[32] have suggested the use of a fixed permissible bond stress value. Ferguson, Thompson and Matlook[25,33] also concluded that diagonal tension cracking lowered the bond stress values. However in some test beams bond failure preceded the diagonal tension cracking. These cases were analysed and the test result presented in Appendix B.

It is suggested that in future "open vertical stirrups" be used in the region where diagonal tension crackings occur to prevent such a failure. The author performed one test using this reinforcement scheme and the results are reported in Appendix B.

The details of the symmetrical bond beams and the beams designed after the University of Texas specimens are shown in Fig. 28 to 31.

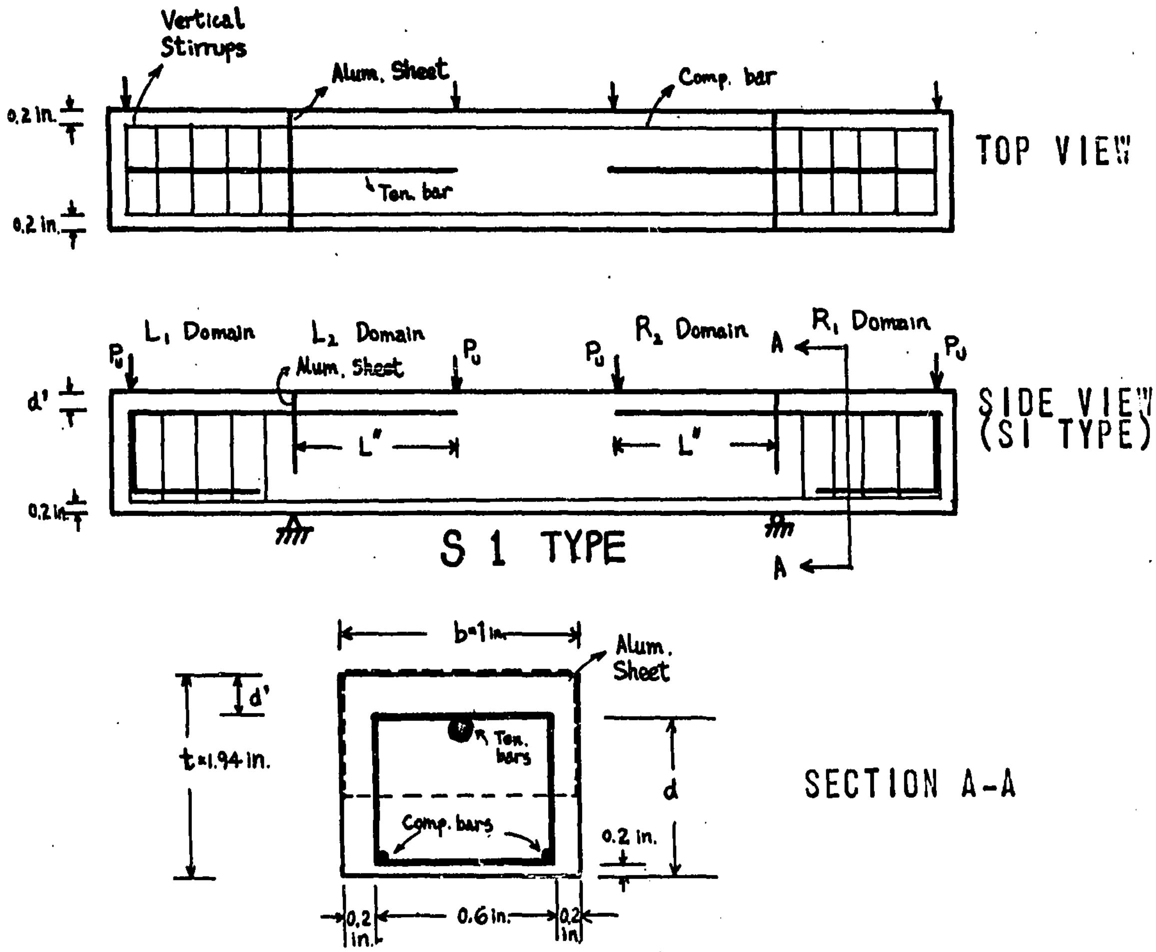


FIG. 28 SPECIMEN DETAILS:SYMMETRICAL BOND BEAM TEST

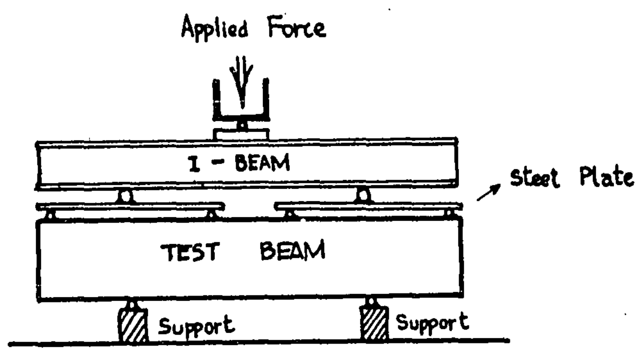


FIG. 29 EXPERIMENTAL SET-UP:SYMMETRICAL BOND BEAM TEST

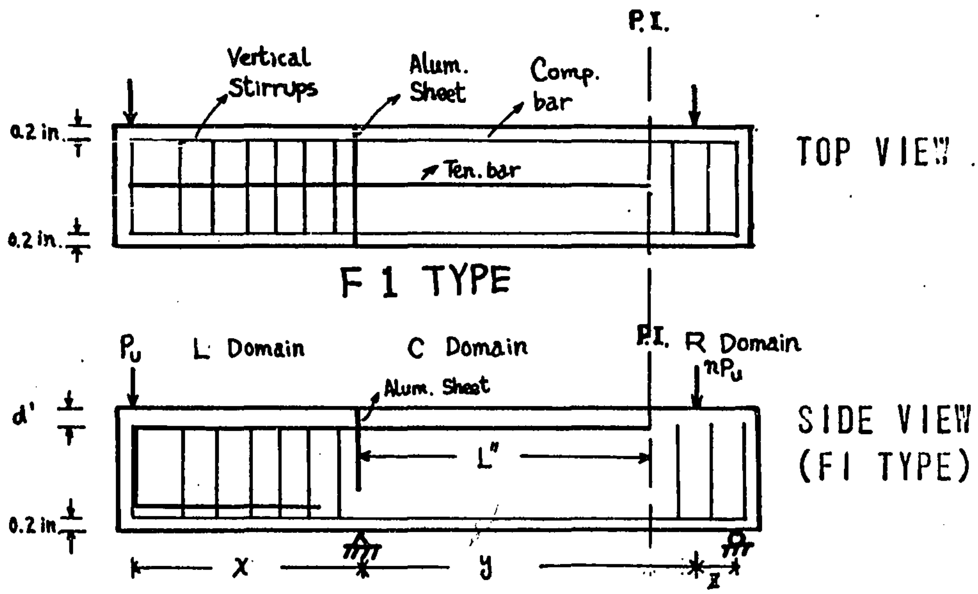


FIG. 30 SPECIMEN DETAILS:UNIVERSITY OF TEXAS BEAM

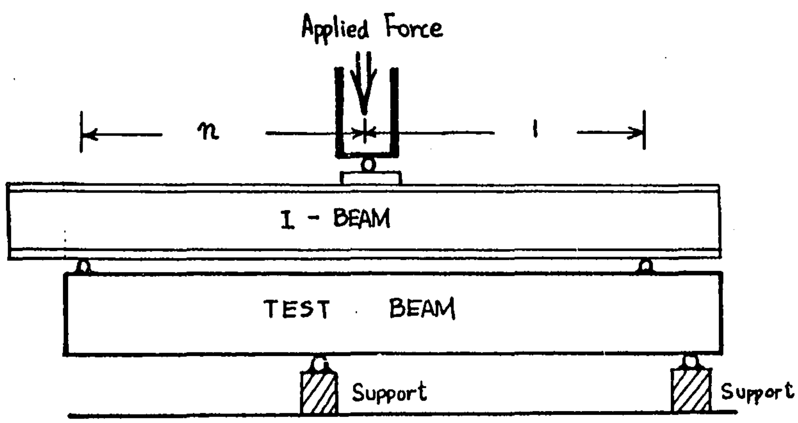


FIG. 31 EXPERIMENTAL SET-UP:UNIVERSITY OF TEXAS BEAM

The following points have to be stressed in the design of model bond beam specimens.

(1) The use of open and closed vertical stirrups is suggested to prevent permature diagonal tension failures.

(2) Artificial crack was inserted on the tension of the beam over the support using an aluminium sheet. This immediately fixes the development length L" to be used in bond stress calculations. Moreover concrete does not contribute in resisting any tension (which normally occurs in beams in practice in spite of the fact that concrete is assumed to carry no tension).

(3) The bars under bond study are at top and therefore free from localized support reaction pressures. Hence there is no need for special supports as in National Bureau of Standards beams.

3.5 COMPARISON AND DISCUSSION OF UNIVERSITY OF TEXAS BEAM AND SYMMETRICAL BOND BEAM SPECIMENS

Ferguson studied the bar cut-off characteristics in the negative moment region of the University of Texas beams. Along with Matlook[33], he also investigated the effect of bar cut-off on bond. McGill Symmetrical Bond specimens also use the negative moment region and the zero moment and zero shear at the free end of the tension bars to study the bond characteristics at bar cut-off points which form points of stress concentration. Effect of concentrated loads at bar cut-off points was also studied in the investigation being reported. Table III.1 indicates that bond stress in UT series is generally higher than that in SB series for the same cross-section, development length, concrete strength and clear cover because of the concentrated loads at the bar cut-off points in the SB beams. It appears from Table III.1 that the UT beam series reinforced with plain steel bars exhibit approximately 19 per cent apparent bond stress increase over the corresponding SB beams when the calculations are based on Working Stress Design theory. However Ultimate Strength Design considerations show that UT beams give approximately 22 per cent higher apparent bond stress than the corresponding SB beams. For deformed bar reinforcing, similar increases were of the order of 45 and 49 per cent respectively using the WSD and USD criteria respectively.

TABLE III.1

COMPARISON OF UNIVERSITY OF TEXAS BEAM AND SYMMERICAL

BOND BEAM SPECIMENS

Type of Bar	Test Specimen	Specimen No.	d' (in.)	L"/D	Types of Failure	u**+	u***+	(b)-(a) (psi)	(d)-(c) (psi)
P-2 (plain bar)	Symmetrical bond beams	BPS1B'-1	0.37	40	B.F.	$285^{(a)}$	$270^{(c)}$		
		BPS1B'-1	0.37	40	B.F.				
		BPS1B'-1	0.37	40	S.D.T.*				
	University of Texas beams	BPF1A'-1	0.37	40	B.F.				
		BPF1A'-1	0.37	40	S.D.T*	$338^{(b)}$	$332^{(d)}$	53	62
		BPF1A'-1	0.37	40	B.F.&P.T.				
D-2 (deformed bar)	Symmetrical bond beams	BDS1G'-3	0.37	10.19	C.S.&B.F.	$742^{(a)}$	$702^{(c)}$		
		BDS1G'-3	0.37	10.19	C.S.&B.F.				
	University of Texas Beams	BDF1D'-3	0.37	10.19	C.S.&B.F.	$1074^{(b)}$	$1049^{(d)}$	332	347

* Indicates that average bond stresses were not calculated for these cases.

+ The notations refer to 4.2, Chapter IV.

TABLE III.2

(UNIVERSITY OF TEXAS BEAM, d' = 0.37 in., f'_c = 3000 psi)

COMPARISON OF EFFICIENCY OF RUSTED AND UNRUSTED BARS

Types of Bar	Specimen No.	L"/D	Type of Failure	u***+ (psi)
	BPF1A'-1	40	B.F.	332
P-2	BPF1A'-1	40	S.D.T.*	
(Plain bar)	BPF1A'-1	40	B.F. & P.T.	
	BPF1F'-2(R)	40	B.F.	326
	BPF1F'-2(R)	40	B.F.	
	BPF1F'-2(R)	40	B.F.	
D-2	BDF1D'-3	10.19	C.S. & B.F.	1049
(Deformed	BDF1F'-6(R)	10.19	C.S. & B.F.	1017
Bar)	BDF1F'-6(R)	10.19	C.S. & B.F.	

* Indicates that average bond stresses were not calculated for these cases.

\+ The notations refer to 4.2, Chapter IV.

3.6 RUSTED AND UNRUSTED BARS

(Comparison and Discussion of Bond Efficiency)

From Table III.2, it appears that the rusted bars do not appreciably affect the bond strength in beam specimens. However in case of eccentric pull-out specimens, the rusted bars were found to increase the bond strength a little (refer 2.4.(h), Chapter II). These findings in beam specimens seem to agree with Johnston and Cox[35] and Kemp et al[11].

3.7 EFFECT OF CLEAR COVER IN BOND BEAM SPECIMENS

Only three specimens were used to investigate the effect of clear cover in model beams and are therefore insufficient to detect any trends However, Table III.3 indicates the significant increase in the apparent bond strength as the clear cover is increased from 0.30 in.,to 0.37 in., all other variables being held constant. These findings appear to agree with the eccentric pull-out test findings.

TABLE III.3 EFFECT OF CLEAR COVER IN BEAM TEST

(University of Texas Beam, D-2.5)

Specimen No.	L"/D	d' (in.)	u*** (psi)	Type of Failure
BDF1E'-18	10	0.30	599	B.F. & C.S.
BDF1D'- 5	10	0.37	1067.1	B.F. & C.S
BDF1D'- 5	10	0.37		B.F. & D.T.

3.8 CRACK PATTERNS AND MECHANISMS OF FAILURE

The crack patterns of test beams in this investigation (University of Texas beam and Symmetrical Bond beam) were observed to be similar to the modes suggested by ACI Committee 408[1]. The cases of bond failure (B.F.), concrete splitting and bond failure (B.F. & C.S), and bond failure and diagonal tension failure (B.F. & D.T.), indicate that flexural crack first occurred at points of maximum negative bending moment (compare with the loaded end in pull-out specimens). The diagonal tension cracks or longitudinal or transverse concrete splitting, or further flexural cracking then followed.

It may be noted that for plain bar reinforcing, simple bond failures were observed without any concrete splitting. However for the beams reinforced with deformed bars concrete splitting always accompanied bond failure. The modes of failure listed as B.F. & D.T. and B.F., C.S., and D.T. were governed by bond failure which was then followed by diagonal tension cracking or concrete splitting.

For the specimens failing in diagonal tension mode (S.D.T.), flexural cracks first appeared and were followed by diagonal tension cracks. The behaviour of these specimens was governed by diagonal tension as mentioned previously, the bond failure in these specimens is a secondary phenomenon and therefore these test results were not analysed. The crack patterns and the overall mechanisms of failure for different modes are shown in Fig. 32-35. The mechanisms of failure and the propagation of splitting cracks were found to be similar to the modes suggested by ACI Committee 408[1]. Furthermore as indicated by the ACI Committee 408, vertical closed stirrups not only slowed the splitting crack propagation but also helped decrease the crack width.

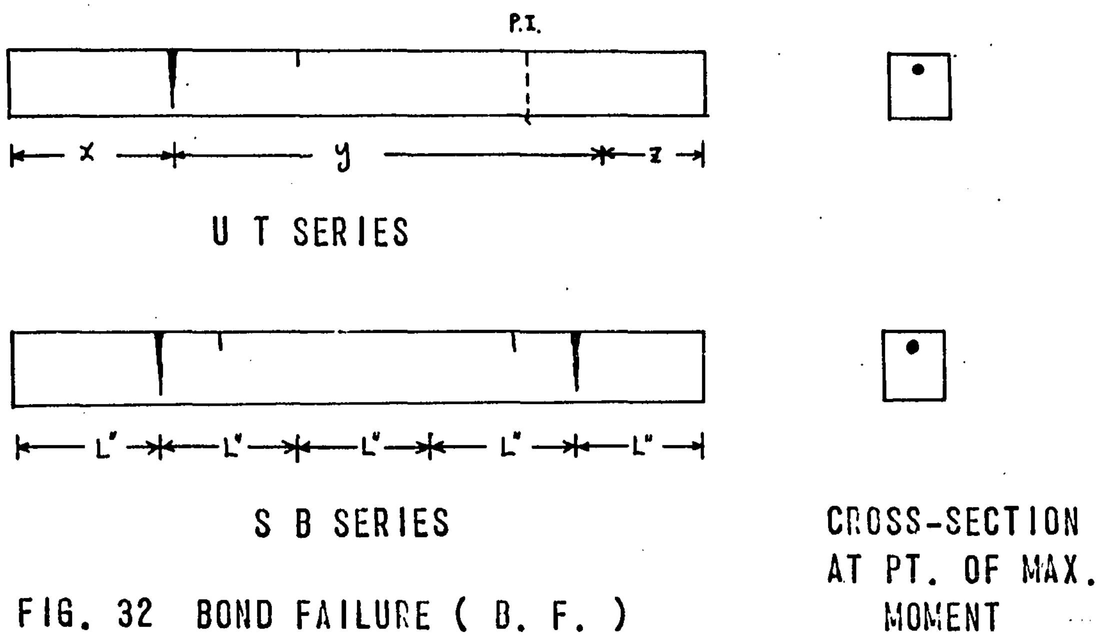


FIG. 32 BOND FAILURE (B. F.)

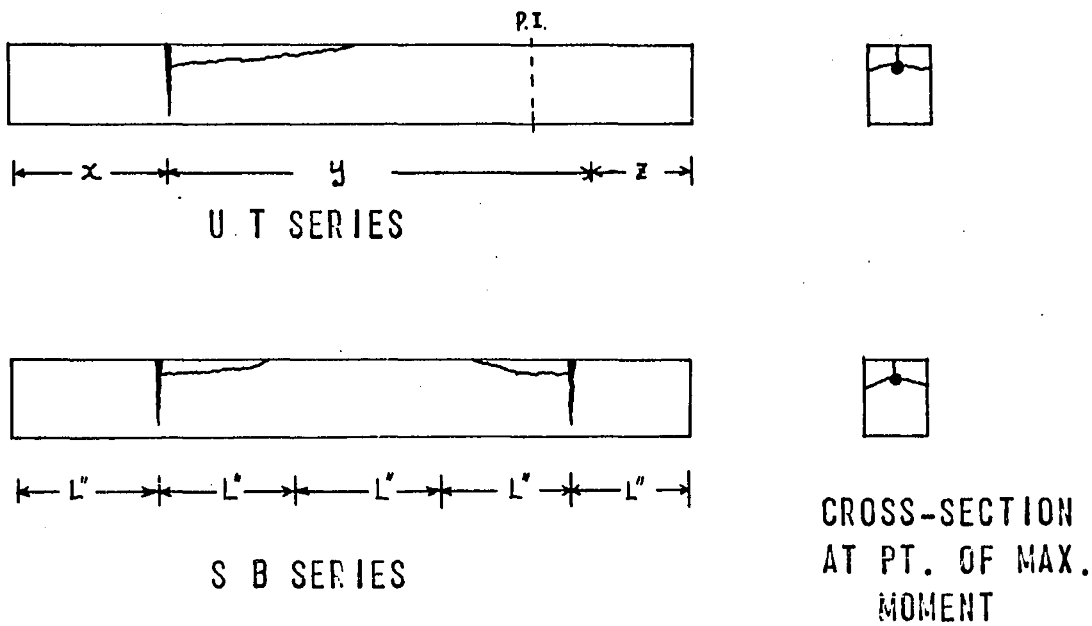


FIG. 33 BOND FAILURE AND CONCRETE SPLITTING
(B. F. & C. S.)

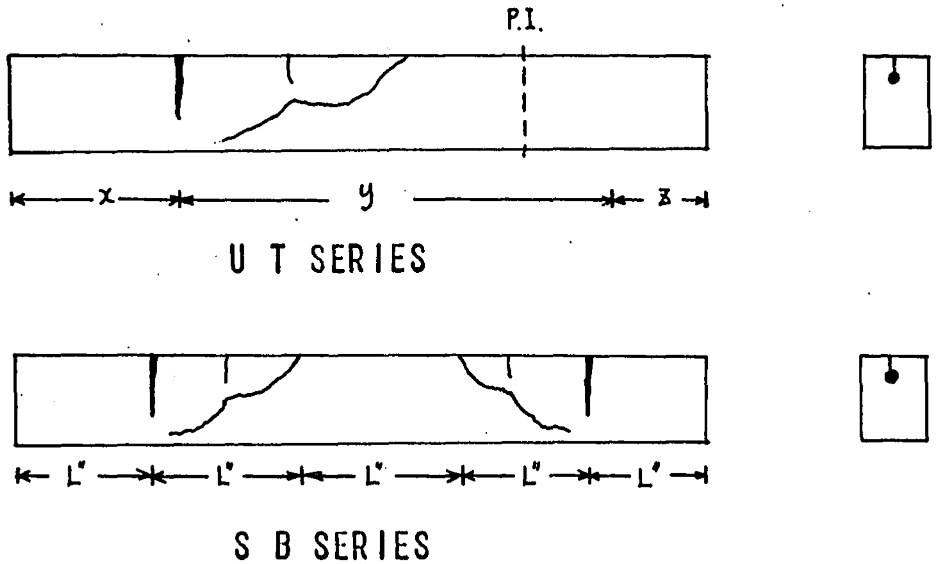


FIG. 34 BOND FAILURE, CONCRETE SPLITTING AND DIAGONAL TENSION (B.F.,C.S.&D.T.)

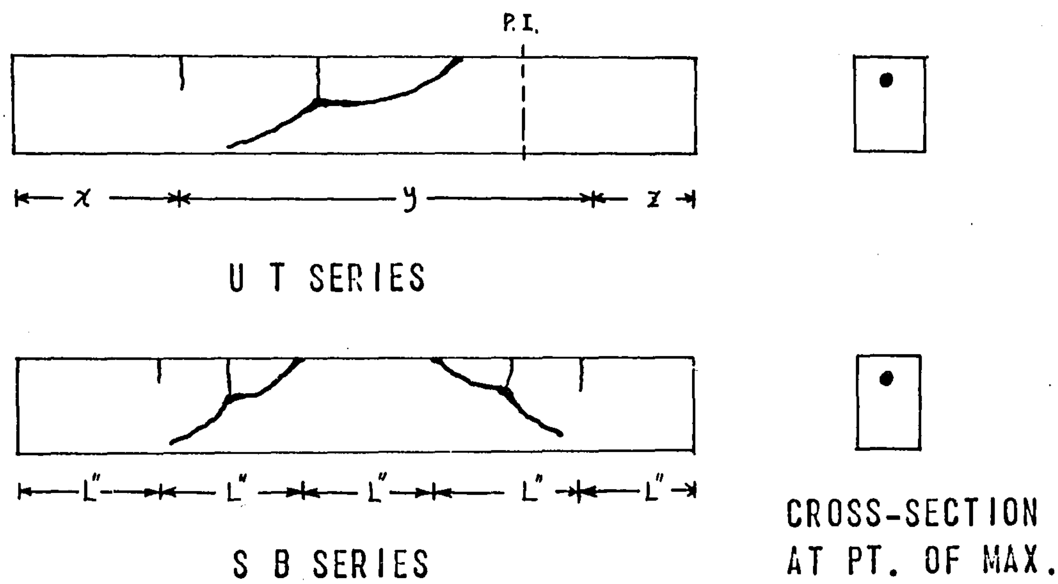


FIG. 35 SERIOUS DIAGONAL TENSION (S. D. T.)

CHAPTER IV

THE RELATIONSHIP BETWEEN PULL-OUT AND BOND BEAM TESTS

4.1 INTRODUCTION

Bond characteristics in prototype or model specimens can be obtained experimentally using either the pull-out or the bond beam test. Investigators differ in their preference of either test. However the pull-out test is usually inexpensive, simple and less time consuming compared with the bond beam test. It must be pointed out that the pull-out test with the exception of the eccentric pull-out test with special support design does not generally represent the actual loading conditions existing in beams in practice while the bond beams do represent these conditions. It is therefore necessary to find out the correlation between the bond stress prediction from the pull-out and the bond beam tests. Summary of findings of investigators in this field is shown in Table IV.1.

4.2 CORRELATION OF AVERAGE ULTIMATE BOND STRESS IN PULL-OUT AND BOND BEAM SPECIMENS

Table IV.2 and Fig. 36 indicate that the average ultimate bond stress values obtained in the eccentric pull-out tests agreed reasonably with the corresponding values from the symmetrical bond beam tests for both plain and deformed bars. This behaviour was anticipated because the overall mechanical behaviour of the two types of specimens was identical with the free ends of the bar under test being subjected to concentrated loads in both cases. Average ultimate bond stress values calculated for the University of Texas beams were generally higher than the results from the eccentric pull-out or the symmetrical bond beam tests. It may be noted that the models of the University of Texas specimens provided for a point of contraflexure at the free end of the bar under test, thus eliminating the disturbance caused by the concentrated loads in other types of specimens. The notations used in Table IV.2 are as follows:

U* = Average ultimate bond stress in the pull-out specimen calculated from equation (3).

U** = Average ultimate bond stress in the beam specimens (symmetrical bond beam and University of Texas beam specimens), calculated using working stress analysis equation. Value of j was taken as 0.875.

U***=Average ultimate bond stress in beam specimens, calculated using the ultimate strength criteria from equation (13) or (13A).

Kemp et al[11] developed the cantilever bond test on reinforced concrete prototype specimens with special supports. (Note that the University of West Virginia specimens used steel conduits around the reinforcement). Some of the experimental results were compared with the National Bureau of Standards test results on bond beams. Symmetrical bond beam and the eccentric pull-out tests on reinforced concrete models were developed independently at McGill University during this investigation. Special support conditions were used in the eccentric pull-out test to simulate the loading conditions in beams in practice (Fig. 1). The current investigation established that the model eccentric pull-out test results agreed well with the corresponding test results from the model bond beam tests. Table IV.3 indicates the present status of research work on relationship between pull-out and bond beam tests.

Bond similitude between prototype pull-out and beam specimens and the corresponding models still remain to be investigated. Some work on bond similitude studies using models of prototypes studied by other investigators is in progress at McGill University.

TABLE IV.1

THE RELATIONSHIP BETWEEN PULL-OUT AND BOND BEAM TESTS

(1) Investigators	(2) Reference No.	(3) Pull-Out Specimen	(4) Bond Beam Specimen	(5) Principal Conclusions	(6) Model or Prototype
Harris Schwindt Taher Werner Hansen Sturnan (M.I.T. Report) (1963)	8	The specimen consisted of a fixed length of wire embedded in a cylinder of mortar. (Diameter of cylinder = 2 in.)	University of Texas Beam (Ferguson's Bond Beams)	(i) Bond beam and pull-out specimens were of the same D and L"/D. In this case, the beam specimens showed twice as much bond resistance as the pull-out specimen. (ii) The limited scope of the pull-out tests did not allow elaborate study of the different variables affecting bond strength.	Model
Mains (1951)	13	Eccentric pull-out specimen (Rectangular cross-section)	National Bureau of Standards Beams	The shapes of the two curves-force in bar, bond stress distribution, were markedly similar when these were plotted to the same scale (with the free end of the beam bar being coincident with the free end of the bar being pulled out). However, this similarity was limited to the portion of the beam between the support and the nearest crack.	Prototype.
Ferguson Breen and Thompson (1965)	27	Eccentric pull-out specimen (with or without spirals)	None. (Compare with Clark's works)[37,38]	(i) There was some doubt that a pull-out specimen could reflect the mode of failure in a beam specimen. (ii) Flexural crack width (in pull-outs) at a given steel stress was speculated to be less than the slip at the loaded end of a pull-out specimen.	Prototype.

TABLE IV.1 CONTINUED

(1) Investigators	(2) Reference No.	(3) Pull-Out Specimen	(4) Bond Beam Specimen	(5) Principal Conclusions	(6) Model or Prototype
Mathey, Watstein (1961)	14	Concentric pull-out specimen (square cross-section)	National Bureau of Standards Beams	(i) The ultimate bond stresses in the pull-out specimens agreed in general with the values obtained in beam with no. 4 bars. However, for no. 8 bars the bond strengths in pull-out specimens were significantly greater than the values obtained with beams. (ii) There was some doubt that a pull-out specimen could reflect the mode of failure in a beam specimen.	Prototype
Perry, Thompson (1966)	18	Eccentric pull-out specimen (special support design).	National Bureau of Standards Beams, Univ. of Texas Beams	(i) There was little similarity between the bond stress distribution in pull-out specimen and bond stress distribution adjacent to a crack in a beam. However the magnitude of the maximum bond stress for each were approximately the same. (ii) The three types of specimens developed about the same maximum bond stress for equivalent steel stresses, but the point of maximum bond stress occurred at a different location in each type.	Prototype

TABLE IV.2

THE RELATIONSHIP BETWEEN ECCENTRIC PULL-OUT TEST,

SYMMETRICAL BOND BEAM TEST AND UNIVERSITY OF TEXAS BEAM TEST

Type of Bar	Test Specimen	Specimen No.	d' in	L"/D	Type of Failure	u* psi	u** psi	u*** psi
P-2 plain bar	Pull-outs	PPS1B-2	0.37	40	B.F.	313	-	-
		PPS1B-2	0.37	40	B.F.			
		PPS1B-2	0.37	40	B.F.			
	Symmetrical bond beams	BPS1B'-1	0.37	40	B.F.	-	285	270
		BPS1B'-1	0.37	40	B.F.			
		BPS1B'-1	0.37	40	S.D.T.+			
	U. of Texas beams	BPF1A'-1	0.37	40	B.F.	-	338	332
		BPF1A'-1	0.37	40	S.D.T.+			
		BPF1A'-1	0.37	40	B.F. & D.T.			
D-2	Pull-outs	PDS1G-6	0.37	10.88	C.S. & B.F.	772	-	-
		PDS1G-6	0.37	10.88	C.S. & B.F.			
		PDS1Y-6	0.37	10	C.S. & B.F.			
		PDS1Y-6	0.37	10	C.S. & B.F.			
	Symmetrical bond beams	BDS1G'-3	0.37	10.19	C.S. & B.F.	-	742	702
		BDS1G'-3	0.37	10.19	C.S. & B.F.			
	U. of Texas beams	BDF1D'-3	0.37	10.19	C.S. & B.F.	-	1074	1049

\+ Indicates that the average ultimate bond stresses were not calculated for these cases.

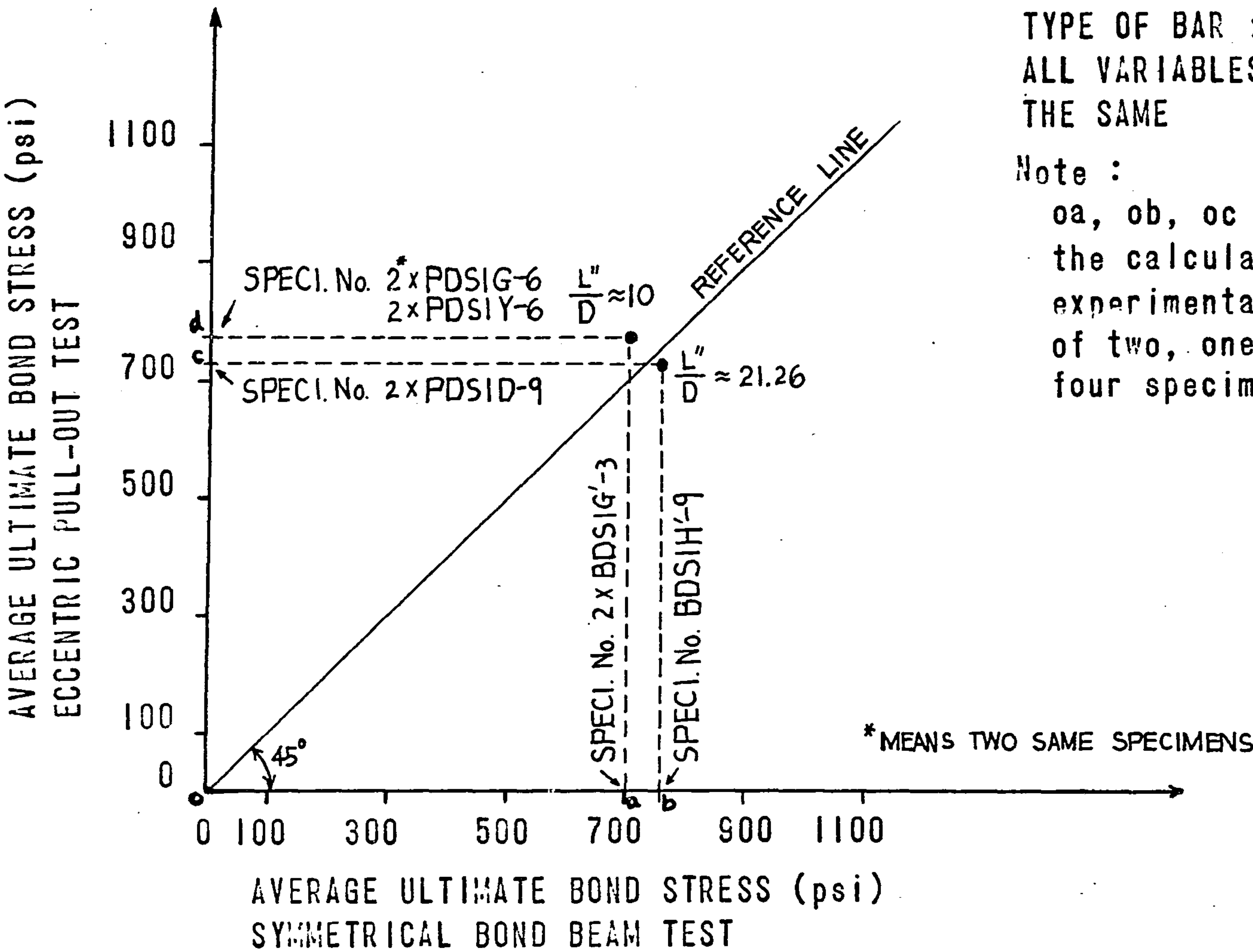
TABLE IV.2 (CONTINUED)

Type of Bar	Test Specimen	Specimen No.	d' in.	L"/D	Type of Failure	u* psi	u** psi	u*** psi
D-2	Pull-outs	PDS1D-9	0.37	21.26	C.S. & B.F.	727	-	-
		PDS1D-9	0.37	21.26	C.S. & B.F.			
	Symmetrical bond beams	BDS1H'-9	0.37	21.26	C.S. & B.F.	-	-	756
D-2.5	Pull-outs	PDS1H-1	0.37	8.43	C.S. & B.F.	708	-	-
	U. of Texas beams	BDF1D'-5	0.37	9.84	C.S. & B.F.	-	1070	1067
		BDF1D'-5	0.37	9.84	B.F. & D.T.			
	Pull-outs	PDS1C-17	0.31	10.27	C.S. & B.F.	516	-	-
		PDS1C-17	0.31	10.27	C.S. & B.F.			
	U. of Texas beams	BDF1E'-18	0.30	9.59	C.S. & B.F.	-	658	599
D-3	Pull-outs	PDH4L-24	0.37	9.64	C.S. & B.F.	737	-	-
		PDH4L-24	0.37	9.64	C.S. & B.F.			
	U. of Texas beams	BDF1E'-17	0.37	9.59	C.S. & B.F.	-	957	988
		BDF1E'-17	0.37	9.59	C.S. & B.F.			
D-3.5	Pull-outs	PDS1D-3	0.37	11.08	C.S. & B.F.	386	-	-
		PDS1D-3	0.37	11.56	C.S. & B.F.			
	U. of Texas beams	BDF1E'-7(R)	0.37	9.75	C.S. & B.F.	-	474	452
		BDF1E'-7(R)	0.37	9.75	C.S. & B.F.			

TABLE IV.3

PRESENT WORK ON RELATIONSHIP BETWEEN PULL-OUT AND BOND BEAM TESTS

Investigators	Reference	Pull-out Specimen	Bond Beam Specimen	Principal Conclusions	Model or Prototype
Kemp, Brezny and Unterspan (1968)	11	Cantilever bond specimen (note that the formula for calculating the bond stress was the same as for the pull-out specimens).	None. (Compared with the work in reference 39).	(1) The average ultimate bond stress of beam test results (National Bureau of Standards beams) were in general agreement with the cantilever test data. (2) The cantilever bond specimens were based on ultimate strength criteria for both shear and flexure.	Prototype
Hsu and Mirza (1967-1969)	22,23	Eccentric pull-out test with special support conditions	(1) Symmetrical bond beam specimen (2) University of Texas beam specimen	(1) The eccentric pull-out specimens were based on ultimate strength criteria for both shear and flexure. (2) The average ultimate bond stresses in the symmetrical bond beam tests were in general agreement with the eccentric pull-out test data (note that the free end of the bar in both specimens was subjected to concentrated loads). (3) The disturbance due to concentrated load caused a reduction in the observed bond strength.	Model



TYPE OF BAR : D-2
ALL VARIABLES ARE
THE SAME

Note :
oa, ob, oc and od are the calculated average experimental values of two, one, two and four specimens respectively

FIG. 36 THE RELATIONSHIP BETWEEN ECCENTRIC PULL-OUT AND SYMMETRICAL BOND BEAM TESTS

CHAPTER V

DESIGN RECOMMENDATIONS FOR REINFORCED CONCRETE MODELS

M.I.T. Report[8] suggested the following points for bond consideration in reinforced concrete models (note that M.I.T. used the University of Texas Beam specimens with SWG No. 10, No. 14 and No. 16 steel wires):

(a) The present ACI Code limits the allowable bond stress to 0.1 f'_c with a maximum of 350 psi for top bars (W.S.D.). This is well within the bond stresses that can be developed by the wires of this investigation.

(b) Since the wire diameter does not enter as variable in this investigation, the relationship between $\frac{u}{\sqrt{f'_c}}$ and embedment length L" is suggested as

$$\frac{u}{\sqrt{f'_c}} = 9.8 - 0.667\, L''$$

(c) It can safely be concluded that the wires can be used to simulate large deformed reinforcing bars.

McGill investigations studied several parameters of the steel bars and some recommendations can be made regarding the use of suitable reinforcement in structural concrete models. The following conclusions are based on the eccentric pull-out test results on specimens with the free end affected by a concentrated load. The specimens with stirrups and hooks gave higher apparent bond strengths and have been ignored. However the effectiveness of stirrups, hooks and bars unaffected by a concentrated load has been discussed in Chapters II and III. The design approach developed from this investigation can be applied at this stage only to reinforced concrete models or small sized specimens using steel wire reinforcement. Then criteria cannot be extrapolated to prototypes unless bond similitude problem between the models and the prototypes is resolved. The value of bond stress - $\sqrt{f'_c}$ ratio against the ratio of development length (or embedment length) to bar diameter (L"/D) is indicated in Fig. 37. A conservative bilinear relationship has been suggested

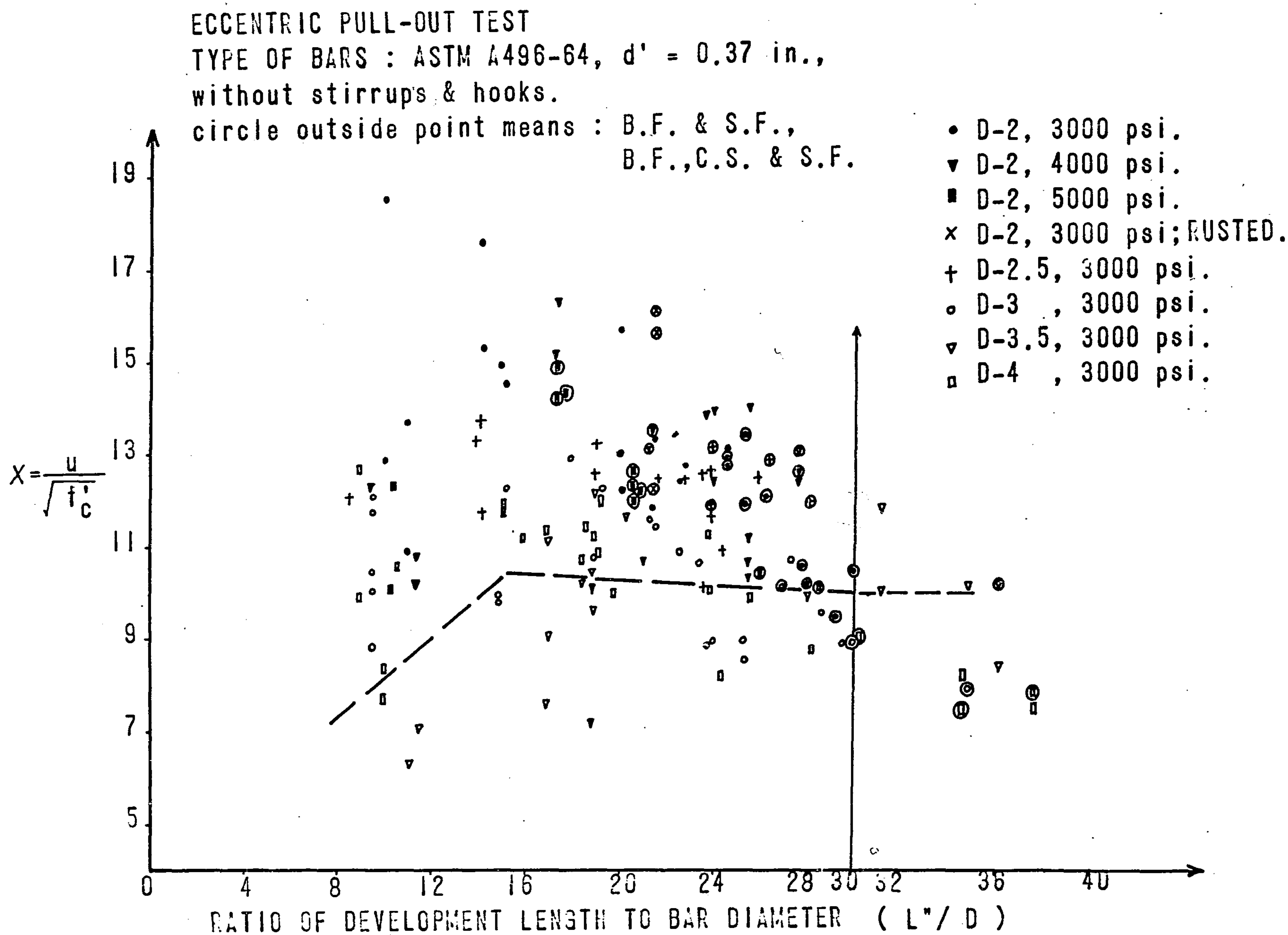


FIG.37 RELATIONSHIP USED TO DEVELOP DESIGN CRITERIA

for use in reinforced concrete model design.

The significant points in Fig. 37 for the purpose of design are that:

(a) The value of $\frac{u}{\sqrt{f'_c}}$ appears to increase with the development length - bar diameter ratio (L"/D) up to L"/D values in the neighbourhood of 15. For L"/D values greater than 15, the bond resistance appears to decrease exponentially with an increase in L"/D ratios and finally becomes asymptotic at a point where steel failure occurs (Chapter II). For design purposes the conservative bilinear proposal is given by the following equations:

$$\text{(i)} \quad 0 \leq \frac{L''}{D} < 15 \; ; \quad \frac{u}{\sqrt{f'_c}} = 0.4 \frac{L''}{D} + 4 \tag{19}$$

$$\text{(ii)} \quad 15 \leq \frac{L''}{D} \leq 30 \; ; \quad \frac{u}{\sqrt{f'_c}} = 11 - 0.033 \frac{L''}{D} \tag{20}$$

(b) The $\frac{L''}{D}$ ratio which leads to steel failures (as noted from Fig.17 and 37) appears to be in the neighbourhood of 30.

(c) The maximum bond stress for reinforced concrete model design (Fig. 37) appears to be

$$u = 10 \sqrt{f'_c} \tag{21}$$

However, variation of $\frac{L''}{D}$ should be taken into consideration as given in equations (19) and (20). Using equation (21), it is seen that

$$\text{for} \quad f'_c = 3000 \text{ psi} \; ; \quad u = 548 \text{ psi} \tag{22}$$

$$f'_c = 4000 \text{ psi} \; ; \quad u = 633 \text{ psi} \tag{23}$$

$$f'_c = 5000 \text{ psi} \; ; \quad u = 706 \text{ psi} \tag{24}$$

The present ACI Code[30] limites the permissible bond stress for ultimate strength design to a maximum of 560 psi for top bars. These permissible bond values indicated in equations (22), (23) and (24) are well within the Code permissible value for $f'_c \geq 3136$ psi.

for use in reinforced concrete model design.

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$$u = 10 \sqrt{f'_c} \qquad (21)$$

However, variation of $\frac{L''}{D}$ should be taken into consideration as given in equations (19) and (20). Using equation (21), it is seen that

$$\text{for} \quad f'_c = 3000 \text{ psi} \; ; \quad u = 548 \text{ psi} \qquad (22)$$

$$f'_c = 4000 \text{ psi} \; ; \quad u = 633 \text{ psi} \qquad (23)$$

$$f'_c = 5000 \text{ psi} \; ; \quad u = 706 \text{ psi} \qquad (24)$$

The present ACI Code[30] limites the permissible bond stress for ultimate strength design to a maximum of 560 psi for top bars. These permissible bond values indicated in equations (22), (23) and (24) are well within the Code permissible value for $f'_c \geq 3136$ psi.

(d) The data used in Fig. 37 indicates that test results of smaller diameter bars (D-2 and D-2.5) leads to a band in the upper part of the diagram while the experimental bond stress values of larger diameter bars (D-3, D-3.5, and D-4) suggest a band in lower part of the same diagram. The suggested equations are therefore quite conservative for smaller sized wires and show reasonable agreement with the larger sized ones.

(e) If the bar free end is not influenced by concentrated load as noted in the University of Texas beam specimens, the average ultimate bond strength appears to increase by approximately 40 to 50 per cent. The suggested equations would therefore be considerably conservative for this case.

(f) The bond criteria suggested in equations (19) and (20) (shown in fig. 37) are applicable only to deformed wires. For small sized plain wires, the test data is shown in Fig. 16 and 22 and could lead to a suitable criteria for bond in models reinforced with plain wires.

CHAPTER VI

CONCLUSIONS

1. The eccentric pull-out test with the specially designed support conditions used in this investigation reflects the loading conditions of combined flexure and shear existing in reinforced concrete beams used in practice. Many of the existing bond tests e.g. some eccentric pull-out tests having the bars under examination subjected to pure flexure do not account for interaction between bond, shear and flexure that normally exists in concrete elements. The eccentric pull-out test is therefore recommended for use in any prototype or model bond investigations. (Also refer to Kemp's work[11]).

2. Symmetrical bond beam specimens (developed at McGill) and the University of Texas beam models were used for the bond beam tests. The symmetrical bond beams have the following characteristics:

   (i) a controlled negative moment region.

   (ii) a controlled zero moment and zero shear region (near the centre of the beam.).

   (iii) a controlled bar cut-off point with the concentrated load at the free end of the test bar.

3. Both the eccentric pull-out and the symmetrical bond beam specimens were subjected to similar loading combinations (flexure and shear) and yielded very close values of average ultimate bond stress. Bond conditions in beam specimens were analysed using both the working stress and ultimate strength design equations. In the pull-out specimens, bond stress was averaged over the entire development length of the bar using the equation

$$u = \frac{f_s}{4\left(\frac{L''}{D}\right)}$$

4. The average ultimate bond stress appears to be closely related to the square root of f'_c. Bond between steel and concrete tends to improve with increasing concrete strengths.

5. The apparent average ultimate bond stress appears to increase with the embedment length-bar diameter ratio up to L"/D values in the neighbourhood of 15. For L"/D values greater than 15, the bond resistance appears to decrease exponentially with an increase in L"/D ratios and finally becomes asymptotic at a point where steel failure occurs.

6. Deformed bars show bond resistance comparable with the prototype reinforcing bars. As expected, deformed bars indicate better bond characteristics than plain bars.

7. An increase in the clear cover to the deformed wire reinforcing increases the apparent bond resistance.

8. Suitable end anchorages increase the average ultimate bond strength developed between steel and concrete.

9. Provision of stirrups increases the apparent bond strength, and slows the propagation of cracks besides causing a decrease in the crack width.

10. Specimens reinforced with rusted bars appear to be insignificantly more efficient in bond development than the specimens with unrusted bars.

11. Some of the findings in the "lower range" of L"/D ratios do not appear to agree with those of other investigators and more experimental work will have to be undertaken to confirm these trends.

12. Concentrated loads at the free end of test bars in the symmetrical bond beam specimens appeared to decrease the bond strength to values lower than those observed in the University of Texas beam models which were not influenced by any load concentrations.

13. A suitable design approach for bond characteristics has been suggested for reinforced concrete model investigations.

14. The suggested bond design criteria for models can be extrapolated for use in prototype reinforced concrete design, once bond similitude between prototypes and models has been established.

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APPENDICES

TEST RESULTS AND MATERIALS

APPENDIX (A)	PULL-OUT TEST RESULTS
	Table 1 to Table 12
APPENDIX (B)	BOND BEAM TEST RESULTS
	University of Texas Beam Test Results
	Table 13 to Table 14
	Symmetrical Bond Beam Test Results
	Table 15 to Table 16
APPENDIX (C)	MATERIALS
	1. Steel Bars
	Fig. A to Fig. G and Table 17
	2. Concrete

APPENDIX (A)

PULL-OUT TEST RESULTS

TABLE 1

CONCENTRIC PULL-OUT TEST RESULTS

Type of Bar: D-2, Deformed Bar
Bar Diameter: 0.159 in.
Area of Cross Section: 0.02 in.^{2}

Specimen No.	b in.	t in.	d' in.	L" in.	$\frac{L"}{D}$	f'_c psi	$f_c'^{0.5}$	$f_c'^{0.7}$	P lb.	$f=\frac{P}{A_s}$ psi	u psi	$X=\frac{u}{f_c'^{0.5}}$	Type of Failure
PDS6A-1	1	1.94	0.89	1.45	9.12	2789	52.8	258	500	25000	685	12.94	C.S.&B.F.
PDS6C-1	1	1.94	0.89	1.60	10.06	3121	55.8	279	580	29000	725	12.95	C.S.&B.F.
PDS6C-1	1	1.94	0.89	1.80	11.31	3121	55.8	279	523	26150	581	10.41	C.S.&B.F.
PDS6A-2	1	1.94	0.89	2.34	14.71	2789	52.8	258	760	38000	645	12.41	C.S.&B.F.
PDS6C-2	1	1.94	0.89	2.38	14.96	3121	55.8	279	1210	60500	1030	18.43	C.S.&B.F.
PDS6G-2	1	1.94	0.89	2.38	14.96	3121	55.8	279	1210	60500	1030	18.43	C.S.&B.F.
PDS6A-3	1	1.94	0.89	3.30	20.76	2789	52.8	258	1320	66000	744	14.07	C.S.&B.F.
PDS6A-3	1	1.94	0.89	3.38	21.27	2789	52.8	258	1460	73000	858	16.23	C.S.&B.F.

TABLE 3

CONCENTRIC PULL-OUT TEST RESULTS

Type of Bar: D-2.5 Deformed Bar
Bar Diameter: 0.178 in.
Area of Cross Section: 0.025in.^{2}

Specimen No.	b in.	t in.	d' in.	L" in.	$\frac{L"}{D}$	f'_c psi	$f_c'^{0.5}$	$f_c'^{0.7}$	P lb.	$f=\frac{P}{A_s}$ psi	u psi	$X=\frac{u}{f_c'^{0.5}}$	Type of Failure
PDS6C-14	1	1.94	0.89	1.85	10.40	3227	56.8	288	535	21400	515	9.06	C.S.&B.F.
PDS6C-14	1	1.94	0.89	1.75	9.65	3227	56.8	288	730	29200	757	13.30	C.S.&B.F.
PDS6C-15	1	1.94	0.89	2.61	14.65	3227	56.8	288	1020	44810	765	13.44	C.S.&B.F.
PDS6C-15	1	1.94	0.89	2.55	14.32	3227	56.8	288	950	38000	663	11.65	C.S.&B.F.
PDS6C-16	1	1.94	0.89	4.28	24.02	3227	56.8	288	1730	69230	728	12.80	B.F.&S.F.
PDS6C-16	1	1.94	0.89	4.29	24.10	3227	56.8	288	1670	66800	693	12.20	B.F.&S.F.
PDS6C-16	1	1.94	0.89	4.31	24.20	3227	56.8	288	1635	65490	676	11.90	B.F.&S.F.

TABLE 2

ECCENTRIC PULL-OUT TEST RESULTS

Type of Bar: D-2, Deformed Bar
Bar Diameter: 0.159 in.
Area of Cross Section: 0.02 in.^{2}

Specimen No.	b in.	t in.	d' in.	L" in.	$\frac{L"}{D}$	f'_c psi	$f'_c{}^{0.5}$	$f'_c{}^{0.7}$	P lb.	$f=\frac{P}{A_s}$ psi	u psi	$X=\frac{u}{f'_c{}^{0.5}}$	$y=\frac{u}{f'_c{}^{0.7}}$	Type of Failure
PDS1G-6	1	1.94	0.37	1.73	10.88	3121	55.8	279	532	26600	612	10.95	1.907	C.S.&B.F.
PDS1G-6	1	1.94	0.37	1.73	10.88	3121	55.8	279	665	33250	765	13.70	2.740	C.S.&B.F.
PDS1Y-6	1	1.94	0.37	1.59	10.00	3001	54.4	272	605	30250	706	12.90	2.590	C.S.&B.F.
PDS1Y-6	1	1.94	0.37	1.59	10.00	3001	54.4	272	805	40250	1006	18.50	3.690	C.S.&B.F.
PDS1G-7	1	1.94	0.37	2.40	15.10	3121	55.8	279	980	49000	813	14.55	2.915	C.S.&B.F.
PDS1G-7	1	1.94	0.37	2.25	14.15	3121	55.8	279	969	48450	855	15.30	3.060	C.S.&B.F.
PDS1G-7	1	1.94	0.37	2.38	14.96	3121	55.8	279	1003	50150	836	14.97	3.000	C.S.&B.F.
PDS1G-7	1	1.94	0.37	2.25	14.15	3121	55.8	279	1136	56800	984	17.60	3.522	C.S.&B.F.
PDS1Y-8	1	1.94	0.37	3.18	20.00	3001	54.4	272	1060	53000	663	12.20	-	C.S.&B.F.
PDS1Y-8	1	1.94	0.37	3.18	20.00	3001	54.4	272	1135	56750	709	13.02	-	C.S.&B.F.
PDS1Y-8	1	1.94	0.37	3.18	20.00	3001	54.4	272	1370	68500	856	15.75	-	C.S.&B.F.
PDS1A-9	1	1.94	0.37	3.58	22.50	2789	52.8	258	1180	59000	655	12.40	2.540	C.S.&B.F.
PDS1D-9	1	1.94	0.37	3.38	21.26	3358	57.9	293	1162	58100	684	11.80	2.330	C.S.&B.F.
PDS1D-9	1	1.94	0.37	3.38	21.26	3358	57.9	293	1310	65500	770	13.30	2.620	C.S.&B.F.
PDS1A-10	1	1.94	0.37	3.91	24.60	2789	52.8	258	1365	68250	694	13.12	-	C.S.&B.F.
PDS1F-10	1	1.94	0.37	3.62	22.75	3134	55.9	280	1260	63000	692	12.76	-	C.S.&B.F.
PDS1F-10	1	1.94	0.37	3.90	24.53	3134	55.9	280	1410	70500	718	12.86	-	B.F.&S.F.&C.S.
PDS1F-10	1	1.94	0.37	3.90	24.53	3134	55.9	280	1430	71500	728	13.01	-	B.F.&S.F.
PDS1E-11	1	1.94	0.37	4.01	25.17	3483	59.0	302	1330	66500	660	11.90	-	B.F.&S.F.
PDS1F-11	1	1.94	0.37	4.15	26.10	3134	55.9	280	1410	70500	675	12.08	-	B.F.&S.F.
PDS1A-11	1	1.94	0.37	4.00	25.16	2789	52.8	258	1435	71750	710	13.42	-	C.S.&B.F.
PDS1E-12	1	1.94	0.37	4.41	27.74	3483	59.0	302	1380	69000	622	10.54	-	B.F.&S.F.&C.S.
PDS1E-12	1	1.94	0.37	4.44	27.91	3483	59.0	302	1345	67250	603	10.22	-	B.F.&S.F.
PDS1E-13	1	1.94	0.37	4.63	29.10	3483	59.0	302	1310	65500	562	9.52	-	C.S.&B.F.

TABLE 2
(CONTINUED)

ECCENTRIC PULL-OUT TEST RESULTS

Specimen No.	b in.	t in.	d' in.	L" in.	$\frac{L"}{D}$	f'_c psi	$f'^{0.5}_c$	$f'^{0.7}_c$	P lb.	$f=\frac{P}{A_s}$ psi	u psi	$X=\frac{u}{f'^{0.5}_c}$	$y=\frac{u}{f'^{0.7}_c}$	Type of Failure
PDS1F-13	1	1.94	0.37	4.75	29.88	3134	55.9	280	1400	70000	585	10.46	-	S.F.&C.S.
PDS1P-14	1	1.94	0.37	1.47	9.38	3723	61.1	316	564	28200	752	12.31	2.38	B.F.&C.S.
PDS1P-15	1	1.94	0.37	1.81	11.39	3723	61.1	316	601	30050	660	10.80	2.09	B.F.&C.S.
PDS1P-15	1	1.94	0.37	1.81	11.39	3723	61.1	316	512	25600	562	9.20	1.78	B.F.&C.S.
PDS1P-16	1	1.94	0.37	2.75	17.30	3723	61.1	316	1280	64000	925	15.12	2.93	B.F.&C.S.
PDS1P-16	1	1.94	0.37	2.75	17.30	3723	61.1	316	1370	68500	980	16.30	3.10	B.F.&C.S.
PDS1P-17	1	1.94	0.37	3.43	21.30	3723	61.1	316	1400	70000	822	13.45	2.60	S.F.&B.F.
PDS1P-17	1	1.94	0.37	3.31	20.85	3723	61.1	316	1084	54200	650	10.64	2.06	B.F.&C.S.
PDS1P-17	1	1.94	0.37	3.35	21.08	3723	61.1	316	1350	67500	801	13.11	2.54	S.F.&B.F.
PDS1Q-18	1	1.94	0.37	1.81	11.39	4727	69.0	370	650	32500	714	10.07	1.93	B.F.&C.S.
PDS1Q-18	1	1.94	0.37	1.81	11.39	4727	69.0	370	729	36448	850	12.32	2.30	B.F.&C.S.
PDS1Q-19	1	1.94	0.37	2.75	17.30	4727	69.0	370	1350	67500	975	14.23	2.64	S.F.&B.F.
PDS1Q-19	1	1.94	0.37	2.75	17.30	4727	69.0	370	1430	71500	1032	15.00	2.79	S.F.&B.F.
PDS1Q-19	1	1.94	0.37	2.80	17.69	4727	69.0	370	1395	69750	985	14.30	2.66	S.F.&B.F.
PDS1Q-20	1	1.94	0.37	3.25	20.44	4727	69.0	370	1420	71000	870	12.60	2.36	S.F.&B.F.
PDS1Q-20	1	1.94	0.37	3.25	20.44	4727	69.0	370	1400	70000	857	12.42	2.32	S.F.&B.F.
PDS1Q-20	1	1.94	0.37	3.30	20.75	4727	69.0	370	1400	70000	844	12.23	2.28	S.F.&C.S.
PDS1Q-20	1	1.94	0.37	3.25	20.44	4727	69.0	370	1350	67500	827	12.00	2.24	S.F.&C.S.(D.T.)
PDS2Y-21 *	1	1.94	0.37	1.59	10.00	3001	54.4	272	810	40500	1013	18.65	-	C.S.&B.F.
PDS2Y-21 *	1	1.94	0.37	1.5[illegible]	10.00	3001	54.4	272	870	43500	1088	20.00	-	C.S.&B.F.
PDS2Y-22 **	1	1.94	0.37	3.18	20.00	3001	54.4	272	1320	66000	825	15.16	-	C.S.&B.F.
PDS2Y-22 **	1	1.94	0.37	3.18	20.00	3001	54.4	272	1390	69500	869	15.95	-	C.S.&B.F.
PDS2Y-22 **	1	1.94	0.37	3.18	20.00	3001	54.4	272	1165	58250	728	13.38	-	C.S.&B.F.
PDS1Y-23 (R)	1	1.94	0.37	3.40	21.42	3001	54.4	272	1460	73000	852	15.65	-	S.F.&B.F.
PDS1Y-23 (R)	1	1.94	0.37	3.40	21.42	3001	54.4	272	1500	75000	875	16.07	-	S.F.
PDS1Y-23 (R)	1	1.94	0.37	3.40	21.42	3001	54.4	272	1140	57000	665	12.22	-	C.S.&B.F.

* Using 2 vertically closed stirrups. ** Using 5 vertically closed stirrups.

TABLE 4

ECCENTRIC PULL-OUT TEST RESULTS

Type of Bar: D-2.5, Deformed Bar
Bar Diameter: 0.178 in.
Area of Cross Section: 0.025 in.^{2}

Specimen No.	b in.	t in.	d' in.	L" in.	$\frac{L"}{D}$	f'_c psi	$f'^{0.5}_c$	$f'^{0.7}_c$	P lb.	$f = \frac{P}{A_s}$ psi	u psi	$X = \frac{u}{f'^{0.5}_c}$	Type of Failure
PDS1H-1	1	1.94	0.37	1.50	8.43	3428	58.8	300	598	23920	708	12.09	C.S.&B.F.
PDS1H-2.	1	1.94	0.37	2.50	14.04	3428	58.8	300	970	38800	690	11.72	C.S.&B.F.
PDS1H-2	1	1.94	0.37	2.51	14.10	3428	58.8	300	1135	45410	806	13.71	C.S.&B.F.
PDS1H-2	1	1.94	0.37	2.46	13.81	3428	58.8	300	1080	43210	782	13.29	C.S.&B.F.
PDS1H-3	1	1.94	0.37	3.38	19.00	3428	58.8	300	1400	56050	737	12.52	C.S.&B.F.
PDS1H-3	1	1.94	0.37	3.38	19.00	3428	58.8	300	1350	54000	710	12.12	C.S.&B.F.
PDS1H-3	1	1.94	0.37	3.38	19.00	3428	58.8	300	1480	59250	780	13.25	C.S.&B.F.
PDS1C-4	1	1.94	0.37	3.69	20.73	3227	56.8	288	1465	58600	707	12.43	C.S.&B.F.
PDS1C-5	1	1.94	0.37	3.84	21.57	3227	56.8	288	1530	61250	709	12.47	C.S.&B.F.
PDS1C-6	1	1.94	0.37	4.19	23.52	3227	56.8	288	1345	53800	572	10.05	C.S.&B.F.
PDS1H-6	1	1.94	0.37	4.20	23.59	3428	58.8	300	1750	70000	742	12.60	C.S.&B.F.
PDS1C-7	1	1.94	0.37	4.25	23.86	3227	56.8	288	1580	63250	664	11.66	C.S.&B.F.
PDS1C-7	1	1.94	0.37	4.25	23.86	3227	56.8	288	1610	64490	676	11.89	B.F.&S.F.
PDS1H-7	1	1.94	0.37	4.25	23.86	3428	58.8	300	1910	76500	802	13.62	C.S.&B.F.
PDS1H-7	1	1.94	0.37	4.25	23.86	3428	58.8	300	1980	79250	831	14.20	B.F.&S.F.
PDS1C-8	1	1.94	0.37	4.34	24.39	3227	56.8	288	1520	60800	623	10.92	C.S.&B.F.
PDS1H-9	1	1.94	0.37	4.62	25.92	3428	58.8	300	1890	75600	730	12.41	C.S.&B.F.
PDS1H-9	1	1.94	0.37	4.66	26.18	3428	58.8	300	1980	79250	756	12.85	B.F.&S.F.&C.S.
PDS1H-10	1	1.94	0.37	5.00	28.08	3428	58.8	300	1970	78800	701	11.92	B.F.&S.F.
PDS1H-11	1	1.94	0.37	5.20	29.20	3428	58.8	300	2010	80190	-	-	S.F.
PDS1C-17	1	1.94	0.31	1.81	10.27	3227	56.8	288	600	24000	585	10.30	C.S.&B.F.
PDS1C-17	1	1.94	0.31	1.81	10.27	3227	56.8	288	456	18300	446	7.85	C.S.&B.F.
PDS1C-18	1	1.94	0.29	2.69	15.10	3227	56.8	288	710	28410	471	8.29	C.S.&B.F.
PDS1C-18	1	1.94	0.32	2.63	14.77	3227	56.8	288	898	35960	610	10.72	C.S.&B.F.
PDS1C-18	1	1.94	0.31	2.53	14.21	3227	56.8	288	890	35600	627	11.02	C.S.&B.F.
PDS1C-19	1	1.94	0.30	3.38	19.00	3227	56.8	288	815	32600	429	7.55	C.S.&B.F.
PDS1C-19	1	1.94	0.29	3.38	19.00	3227	56.8	288	944	37750	497	8.74	C.S.&B.F.
PDS1C-19	1	1.94	0.29	3.38	19.00	3227	56.8	288	877	35100	462	8.13	C.S.&B.F.

TABLE 5

CONCENTRIC PULL-OUT TEST RESULTS

Type of Bar: D-3,Deformed Bar
Bar Diameter: 0.195 in.
Area of Cross Section: 0.03 in.^{2}

Specimen No.	b in.	t in.	d' in.	L" in.	$\frac{L"}{D}$	f'_c psi	$f'^{0.5}_c$	$f'^{0.7}_c$	P lb.	$f=\frac{P}{A_s}$ psi	u psi	$X=\frac{u}{f'^{0.5}_c}$	Type of Failure
PDS6I-15	1	1.94	0.89	1.95	10.00	2913	54.0	267	492	16400	410	7.60	C.S.&B.F.
PDS6I-15	1	1.94	0.89	2.00	10.25	2913	54.0	267	620	20667	508	9.42	C.S.&B.F.
PDS6I-16	1	1.94	0.89	2.80	14.36	2913	54.0	267	1230	41000	714	13.20	C.S.&B.F.
PDS6I-17	1	1.94	0.89	3.13	15.90	2913	54.0	267	1132	37733	586	10.86	C.S.&B.F.

TABLE 7

CONCENTRIC PULL-OUT TEST RESULTS

Type of Bar: D-3.5, Deformed Bar
Bar Diameter: 0.212 in.
Area of Cross Section: 0.035 in.^{2}

Specimen No.	b in.	t in.	d' in.	L" in.	$\frac{L"}{D}$	f'_c psi	$f'^{0.5}_c$	$f'^{0.7}_c$	P lb.	$f=\frac{P}{A_s}$ psi	u psi	$X=\frac{u}{f'^{0.5}_c}$	Type of Failure
PDS6D-1	1	1.94	0.89	2.91	10.90	3358	57.9	293	1038	29680	681	11.79	C.S.&B.F.
PDS6D-1	1	1.94	0.89	2.19	10.28	3358	57.9	293	1090	31160	755	13.05	C.S.&B.F.
PDS6D-2	1	1.94	0.89	3.50	16.51	3358	57.9	293	1540	44000	673	11.62	C.S.&B.F.
PDS6D-2	1	1.94	0.89	3.50	16.51	3358	57.9	293	1425	40710	623	10.76	C.S.&B.F.

TABLE 6

ECCENTRIC PULL-OUT TEST RESULTS

Type of Bar: D-3, Deformed Bar
Bar Diameter: 0.195 in.
Area of Cross Section: 0.03 in.^{2}

Specimen No.	b in.	t in.	d' in.	L" in.	$\frac{L"}{D}$	f'_c psi	$f'^{0.5}_c$	$f'^{0.7}_c$	P lb.	$f=\frac{P}{A_s}$ psi	u psi	$X=\frac{u}{f'^{0.5}_c}$	Type of Failure
PDS1L-1	1	1.94	0.37	1.85	9.48	3404	58.2	290	800	26667	704	12.10	C.S.&B.F.
PDS1Z-1	1	1.94	0.37	1.88	9.65	3397	58.1	295	790	26333	685	11.79	C.S.&B.F.
PDS1Z-1	1	1.94	0.37	1.88	9.65	3397	58.1	295	595	19833	514	8.84	C.S.&B.F.
PDS1Z-1	1	1.94	0.37	1.88	9.65	3397	58.1	295	705	23500	609	10.46	C.S.&B.F.
PDS1Z-1	1	1.94	0.37	1.88	9.65	3397	58.1	295	735	24500	635	10.92	C.S.&B.F.
PDS1Z-1	1	1.94	0.37	1.88	9.65	3397	58.1	295	675	22500	583	10.01	C.S.&B.F.
PDS1J-2	1	1.94	0.37	2.88	14.78	2973	54.5	270	950	31667	537	9.84	C.S.&B.F.
PDS1J-2	1	1.94	0.37	2.88	14.78	2973	54.5	270	1060	32000	541	9.92	C.S.&B.F.
PDS1J-2	1	1.94	0.37	2.94	15.15	2973	54.5	270	1200	40000	666	12.23	C.S.&B.F.
PDS1I-3	1	1.94	0.37	3.50	17.90	2913	54.0	267	1500	50000	697	12.90	C.S.&B.F.
PDS1I-3	1	1.94	0.38	3.75	19.22	2913	54.0	267	1530	51000	664	12.29	C.S.&B.F.
PDS1I-3	1	1.94	0.27	3.69	18.92	2913	54.0	267	1320	44000	582	10.78	C.S.&B.F.
PDS1I-4	1	1.94	0.37	4.19	21.49	2913	54.0	267	1590	53000	617	11.42	C.S.&B.F.
PDS1I-4	1	1.94	0.37	4.15	21.29	2913	54.0	267	1610	53667	626	11.60	C.S.&B.F.
PDS1I-5	1	1.94	0.37	4.38	22.45	2913	54.0	267	1580	52667	588	10.90	C.S.&B.F.
PDS1L-6	1	1.94	0.37	4.62	23.70	3404	58.2	296	1465	48833	515	8.87	C.S.&B.F.
PDS1L-6	1	1.94	0.37	4.56	23.39	3404	58.2	296	1738	57933	618	10.62	C.S.&B.F.
PDS1I-6	1	1.94	0.36	4.68	24.00	2913	54.0	267	1360	45333	472	8.74	C.S.&B.F.
PDS1L-7	1	1.94	o.37	4.94	25.33	3404	58.2	296	1615	52833	521	8.98	C.S.&B.F.
PDS1I-7	1	1.94	0.36	4.94	25.33	2913	54.0	267	1400	46667	460	8.52	C.S.&B.F.
PDS1J-8	1	1.94	0.37	5.20	26.82	2973	54.5	270	1775	59167	552	10.12	S.F.&B.F.
PDS1J-8	1	1.94	0.37	5.31	27.22	2973	54.4	270	1910	63667	585	10.73	C.S.&B.F.
PDS1L-9	1	1.94	0.37	5.56	28.52	3404	58.2	296	1900	63333	555	9.54	C.S.&B.F.
PDS1I-10	1	1.94	0.36	5.75	29.50	2913	54.0	267	1700	56667	480	8.90	C.S.&B.F.

TABLE 6
(CONTINUED)

ECCENTRIC PULL-OUT TEST RESULTS

Specimen No.	b in.	τ in.	d' in.	L" in.	$\frac{L"}{D}$	f'_c psi	$f'^{0.5}_c$	$f'^{0.7}_c$	P lb.	$f=\frac{P}{A_s}$ psi	u psi	$X=\frac{u}{f'^{0.5}_c}$	Type of Failure
PDS1K-11	1	1.94	0.37	5.81	29.80	3050	55.2	274	1770	59000	494	8.95	S.F.&C.S.
PDS1K-11	1	1.94	0.37	5.86	30.08	3050	55.2	274	1780	59333	-	-	S.F.
PDS1K-12	1	1.94	0.37	6.79	34.80	3050	55.2	274	1840	61333	440	7.98	B.F.&S.F.&C.S.
PDS1I-13	1	1.94	0.35	2.06	10.28	2913	54.0	267	640	21333	518	9.60	C.S.&B.F.
PDS1I-13	1	1.94	0.35	1.89	9.69	2913	54.0	267	550	18333	474	8.78	C.S.&B.F.
PDS1I-14	1	1.94	0.33	2.97	15.24	2913	54.0	267	1018	33933	556	10.30	C.S.&B.F.
PDS1I-14	1	1.94	0.35	3.00	15.79	2913	54.0	267	1040	34667	549	10.16	C.S.&B.F.
PDS1I-14	1	1.94	0.35	3.00	15.79	2913	54.0	267	1040	34667	549	10.16	C.S.&B.F.
PDS6I-18	1	1.94	0.89	3.89	19.90	2913	54.0	267	1430	47667	600	11.11	C.S.&B.F.
PDS6I-19	1	1.94	0.89	4.06	20.86	2913	54.0	267	1590	53000	636	11.78	C.S.&B.F.
PDS6I-19	1	1.94	0.89	4.06	20.86	2913	54.0	267	1590	53000	636	11.78	C.S.&B.F.
PDH5I-20	1	1.94	0.36	1.87	9.59	2913	54.0	267	760	25333	661	12.24	C.S.&B.F.
PDH5I-20	1	1.94	0.37	1.87	9.59	2913	54.0	267	860	28667	748	13.86	C.S.&B.F.
PDH5I-20	1	1.94	0.37	1.87	9.59	2913	54.0	267	580	16000	418	7.75	C.S.&B.F.
PDH3J-21	1	1.94	0.37	1.88	9.64	2973	54.5	270	820	27333	707	12.97	C.S.&B.F.
PDH3J-21	1	1.94	0.37	1.88	9.64	2973	54.5	270	710	23667	615	11.30	C.S.&B.F.
PDH3J-22	1	1.94	0.37	2.98	15.58	2973	54.5	270	1300	43624	700	12.83	C.S.&B.F.
PDH3J-22	1	1.94	0.37	2.98	15.58	2973	54.5	270	1330	44333	605	13.05	C.S.&B.F.
PDH3S-23	1	1.94	0.37	3.74	19.20	3183	56.3	282	1600	55333	742	13.20	C.S.&B.F.
PDH3S-23	1	1.94	0.37	3.74	19.20	3183	56.3	282	1580	52667	685	12.22	C.S.&B.F.
PDH4L-24	1	1.94	0.37	1.88	9.64	3397	58.1	295	800	26667	687	11.81	C.S.&B.F.
PDH4L-24	1	1.94	0.37	1.88	9.64	3397	58.1	295	1010	33667	875	15.05	C.S.&B.F.
PDH4L-24	1	1.94	0.37	1.88	9.64	3397	58.1	295	750	25000	648	11.16	C.S.&B.F.
PDH40-25	1	1.94	0.37	2.992	15.70	3183	56.3	282	1530	51000	814	14.52	C.S.&B.F.
PDH40-25	1	1.94	0.37	2.992	15.70	3183	56.3	282	1220	40667	639	11.41	C.S.&B.F.
PDH40-25	1	1.94	0.37	2.992	15.70	3183	56.3	282	1770	59000	940	16.80	S.F.&B.F.
PDH40-26	1	1.94	0.37	3.740	19.20	3183	56.3	282	1790	59667	776	13.86	C.S.&B.F.
PDH40-26	1	1.94	0.37	3.740	19.20	3183	56.3	282	1700	56667	737	13.13	C.S.&B.F.

TABLE 8

ECCENTRIC PULL-OUT TEST RESULTS

Type of Bar: D-3.5., Deformed Bar
Bar Diameter: 0.212 in.
Area of Cross Section: 0.035 in.^{2}

Specimen No.	b in.	t in.	d' in.	L" in.	$\frac{L''}{D}$	f'_c psi	$f'^{0.5}_c$	$f'^{0.7}_c$	P lb.	$f=\frac{P}{A_s}$ psi	u psi	$X=\frac{u}{f'^{0.5}_c}$	Type of Failure
PDS1D-3	1	1.94	0.37	2.34	11.08	3358	57.9	293	563	16100	363	6.27	C.S.&B.F.
PDS1D-3	1	1.94	0.37	2.45	11.56	3358	57.9	293	646	18500	409	7.06	C.S.&B.F.
PDS1D-4	1	1.94	0.37	3.62	17.08	3358	57.9	293	1040	29700	435	7.52	C.S.&B.F.
PDS1D-4	1	1.94	0.37	3.62	17.08	3358	57.9	293	1253	35830	524	9.05	C.S.&B.F.
PDS1D-4	1	1.94	0.37	3.61	17.02	3358	57.9	293	1530	43730	643	11.11	C.S.&B.F.
PDS1E-5	1	1.94	0.36	3.90	18.40	3483	59.0	302	1550	44300	602	10.20	C.S.&B.F.
PDS1E-6	1	1.94	0.37	4.12	18.92	3483	59.0	302	1470	42000	556	7.18	C.S.&B.F.
PDS1E-6	1	1.94	0.37	4.01	18.92	3483	59.0	302	1550	43000	568	9.64	C.S.&B.F.
PDS1E-6	1	1.94	0.37	4.00	18.86	3483	59.0	302	1572	44910	596	10.10	C.S.&B.F.
PDS1E-6	1	1.94	0.37	4.00	18.86	3483	59.0	302	1630	46600	618	10.48	C.S.&B.F.
PDS1E-7	1	1.94	0.37	4.31	20.32	3483	59.0	302	1965	56200	690	11.69	C.S.&B.F.
PDS1K-8	1	1.94	0.37	5.01	23.63	3050	55.2	274	2520	72000	764	13.82	C.S.&B.F.
PDS1X-8	1	1.94	0.37	5.05	24.00	3184	56.4	282	2360	67410	702	12.42	C.S.&B.F.
PDS1X-8	1	1.94	0.37	5.05	24.00	3184	56.4	282	2650	75750	789	13.96	C.S.&B.F.
PDS1K-9	1	1.94	0.37	5.44	25.63	3050	55.2	274	2110	60250	588	10.65	C.S.&B.F.
PDS1K-9	1	1.94	0.37	5.44	25.63	3050	55.2	274	2070	59150	569	10.30	C.S.&B.F.
PDS1X-9	1	1.94	0.37	5.44	25.83	3184	56.4	282	2285	65200	630	11.17	C.S.&B.F.
PDS1X-9	1	1.94	0.37	5.44	25.83	3184	56.4	282	2860	81750	790	14.00	C.S.&B.F.
PDS1K-10	1	1.94	0.37	5.81	27.51	3050	55.2	274	2680	76500	695	12.58	S.F.&B.F.
PDS1X-10	1	1.94	0.37	5.81	27.62	3184	56.4	282	2840	81200	735	13.01	S.F.&B.F.
PDS1X-10	1	1.94	0.37	5.81	27.62	3184	56.4	282	2710	77440	700	12.41	C.S.&B.F.
PDS1K-11	1	1.94	0.37	5.94	28.00	3050	55.2	274	2150	61400	548	9.94	C.S.&B.F.
PDS10-12	1	1.94	0.37	6.60	31.10	3183	56.3	282	2670	76250	665	11.80	C.S.&B.F.
PDS10-12	1	1.94	0.37	6.62	31.20	3183	56.3	282	2265	64750	564	10.01	C.S.&B.F.
PDS1X-13	1	1.94	0.37	7.31	34.75	3184	56.4	282	2790	79650	572	10.13	C.S.&B.F.
PDS1X-14	1	1.94	0.37	8.00	38.02	3184	56.4	282	2490	71200	468	8.30	C.S.&B.B.
PDS1X-14	1	1.94	0.37	8.00	38.02	3184	56.4	282	3060	87500	575	10.20	S.F.&B.F.

TABLE 9

CONCENTRIC PULL-OUT TEST RESULTS

Type of Bar: D-4, Deformed Bar.
Bar Diameter: 0.225 in.
Area of Cross Section: 0.040 in.^{2}

Specimen No.	b in.	t in.	d' in.	L" in.	$\frac{L"}{D}$	f'_c psi	$f'^{0.5}_c$	$f'^{0.7}_c$	P lb.	$f=\frac{P}{A_s}$ psi	u psi	$X=\frac{u}{f'^{0.5}_c}$	Type of Failure
PDS6H-1	1	1.94	0.89	2.12	9.43	3428	58.8	300	1050	26250	697	11.84	C.S.&B.F.
PDS6H-1	1	1.94	0.89	2.19	9.74	3428	58.8	300	1050	26250	674	11.45	C.S.&B.F.
PDS6H-1	1	1.94	0.89	2.29	10.19	3428	58.8	300	1240	31000	760	12.91	C.S.&B.F.
PDS6H-1	1	1.94	0.89	2.38	10.58	3428	58.8	300	1555	38875	918	15.60	C.S.&B.F.
PDS6H-2	1	1.94	0.89	3.94	17.51	3428	58.8	300	2370	59250	844	14.32	C.S.&B.F.
PDS6N-3	1	1.94	0.89	4.25	18.80	3144	56.1	280	2530	63250	841	15.00	C.S.&B.F.
PDS6N-3	1	1.94	0.89	4.25	18.80	3144	56.1	280	2520	63000	838	14.93	C.S.&B.F.
PDS6N-3	1	1.94	0.89	4.50	20.10	3144	56.1	280	2660	66500	828	14.75	B.F.&S.F.
PDS6N-3	1	1.94	0.89	4.38	19.47	3144	56.1	280	2530	63250	800	14.25	C.S.&B.F.

TABLE 11

CONCENTRIC PULL-OUT TEST RESULTS

Type of Bar: P-2, Plain Bar.
Bar Diameter: 0.162 in.
Area of Cross Section: 0.0206 in.^{2}

Specimen No.	b in.	t in.	d' in.	L" in.	$\frac{L"}{D}$	f'_c psi	$f'^{0.5}_c$	$f'^{0.7}_c$	P lb.	$f=\frac{P}{A_s}$ psi	u psi	$X=\frac{u}{f'^{0.5}_c}$	Type of Failure
PDS6B-17	1	1.94	0.89	6.48	40.00	3132	53.0	280	880	41780	261	4.63	B.F.
PDS6B-17	1	1.94	0.89	6.48	40.00	3132	53.0	280	1095	52000	325	5.76	B.F.
PDS6B-17	1	1.94	0.89	6.48	40.00	3132	53.0	280	965	45800	286	5.07	B.F.

TABLE 10

ECCENTRIC PULL-OUT TEST RESULTS

Type of Bar: D-4, Deformed Bar
Bar Diameter: 0.225 in.
Area of Cross Section: 0.040 in.^{2}

Specimen No.	b in.	t in.	d' in.	L" in.	$\frac{L"}{D}$	f'_c psi	$f'_c{}^{0.5}$	$f'_c{}^{0.7}$	P lb.	$f=\frac{P}{A_s}$ psi	u psi	$X=\frac{u}{f'_c{}^{0.5}}$	Type of Failure
PDS1Z-4	1	1.94	0.37	2.25	10.00	2769	52.2	255	700	17500	438	8.40	C.S.&B.F.
PDS1Z-4	1	1.94	0.37	2.25	10.00	2769	52.2	255	655	16375	404	7.84	C.S.&B.F.
PDS1N-4	1	1.94	0.37	2.00	8.89	3144	56.1	280	770	19250	542	9.66	C.S.&B.F.
PDS1N-4	1	1.94	0.37	2.00	8.89	3144	56.1	280	1013	25325	712	12.68	C.S.&B.F.
PDS1N-5	1	1.94	0.37	2.38	10.58	3144	56.1	280	1020	25500	603	10.75	C.S.&B.F.
PDS1N-6	1	1.94	0.37	3.38	15.01	3144	56.1	280	1610	40250	671	11.95	C.S.&B.F.
PDS1N-6	1	1.94	0.37	3.38	15.01	3144	56.1	280	1600	40000	666	11.87	C.S.&B.F.
PDS1N-7	1	1.94	0.37	3.81	16.95	3144	56.1	280	1700	42500	628	11.20	C.S.&B.F.
PDS1N-7	1	1.94	0.37	3.79	16.85	3144	56.1	280	1710	42750	635	11.32	C.S.&B.F.
PDS1Z-8	1	1.94	0.37	4.30	19.12	2769	52.2	255	1930	48250	632	12.10	C.S.&B.F.
PDS1Z-8	1	1.94	0.37	4.30	19.12	2769	52.2	255	1740	43500	567	10.85	C.S.&B.F.
PDS1N-8	1	1.94	0.37	4.19	18.63	3144	56.1	280	1810	45250	607	10.81	C.S.&B.F.
PDS1N-8	1	1.94	0.37	4.44	19.75	3144	56.1	280	1770	44250	561	10.00	C.S.&B.F.
PDS1N-8	1	1.94	0.37	4.25	18.90	3144	56.1	280	1910	47750	632	11.24	C.S.&B.F.
PDS1O-8	1	1.94	0.37	4.13	18.50	3183	56.3	282	1910	47750	646	11.48	C.S.&B.F.
PDS1O-8	1	1.94	0.37	4.19	18.63	3183	56.3	282	1820	45500	611	10.85	C.S.&B.F.
PDS1O-9	1	1.94	0.37	5.30	23.71	3183	56.3	282	2155	53875	568	10.09	C.S.&B.F.
PDS1O-9	1	1.94	0.37	5.30	23.72	3193	56.3	282	2415	60375	635	11.27	C.S.&B.F.
PDS1O-9	1	1.94	0.37	5.40	24.18	3183	56.3	282	1790	44750	462	8.39	C.S.&B.F.
PDS1O-10	1	1.94	0.37	5.80	25.98	3183	56.3	282	2445	61125	588	10.43	S.F.&B.F.&C.S.(D.T.)
PDS1O-10	1	1.94	0.37	5.70	25.50	3183	56.3	282	2300	57500	562	9.95	B.F.&C.S.
PDS1O-11	1	1.94	0.37	6.34	28.38	3183	56.3	282	2550	63750	573	10.16	S.F.&B.F.
PDS1O-11	1	1.94	0.37	6.31	28.22	3183	56.3	282	2180	54500	492	8.73	C.S.&B.F.
PDS15-12	1	1.94	0.37	6.88	30.80	3255	57.0	289	2530	63250	514	9.02	S.F.&B.F.&C.S.
PDS1X-12	1	1.94	0.37	6.81	30.23	3184	56.4	282	2730	68250	565	-	S.F.
PDS1X-15	1	1.94	0.37	7.80	34.62	3184	56.4	282	2562	64050	463	8.25	B.F.C.S.(D.T.)
PDS1X-15	1	1.94	0.37	7.80	34.62	3184	36.4	282	2542	63440	460	8.20	S.F.&B.F.
PDS1Y-14	1	1.94	0.37	8.50	37.77	3001	54.4	272	2465	61625	408	7.50	C.S.&B.F.
PDS1Y-14	1	1.94	0.37	8.50	37.77	3001	54.4	272	2560	64000	424	7.80	S.F.&B.F.
PDS2Z-16 *	1	1.94	0.37	4.30	19.12	2769	52.2	255	2150	53750	704	13.50	C.S.&B.F.
PDS2Z-16 *	1	1.94	0.37	4.30	19.12	2769	52.2	255	2170	54250	710	13.58	C.S.&B.F.
PDS1Y-17 (R)	1	1.94	0.37	4.25	20.00	3001	54.4	272	2330	58250	728	13.40	C.S.&B.F.
PDS1Y-17 (R)	1	1.94	0.37	4.25	20.00	3001	54.4	272	2130	53250	666	12.24	C.S.&B.F.
PDS1Y-17 (R)	1	1.94	0.37	4.25	20.00	3001	54.4	272	2280	57000	713	13.10	C.S.&B.F.

* Using 6 vertically closed stirrups.

TABLE 12

ECCENTRIC PULL-OUT TEST RESULTS

Type of Bar: P-2, Plain Bar
Bar Diameter: 0.162 in.
Area of Cross Section: 0.0206 in.^{2}

Specimen No.	b in.	t in.	d' in.	L" in.	$\frac{L"}{D}$	f'_c psi	$f'^{0.5}_c$	$f'^{0.7}_c$	P lb.	$f=\frac{P}{A_s}$ psi	u psi	$X=\frac{u}{f'^{0.5}_c}$	Type of Failure
PDS1D-1	1	1.94	0.37	3.50	21.60	3358	57.9	292	722	34230	396	6.85	B.F.
PDS1D-1	1	1.94	0.37	3.44	21.22	3358	57.9	292	771	36600	431	7.45	B.F.
PDS1B-2	1	1.94	0.37	6.48	40.00	3132	53.0	280	1100	52200	327	6.17	B.F.
PDS1B-2	1	1.94	0.37	6.48	40.00	3132	53.0	280	1180	56000	350	6.60	B.F.
PDS1B-2	1	1.94	0.37	6.48	40.00	3132	53.0	280	890	42110	263	4.96	B.F.
PDS1B-3	1	1.94	0.37	8.30	51.20	3132	53.0	280	1240	58900	236	4.45	B.F.
PDS1B-3	1	1.94	0.37	8.24	50.78	3132	53.0	280	1220	57950	235	4.43	B.F.
PDS1B-4	1	1.94	0.37	9.25	57.08	3132	53.0	280	1240	58900	258	4.87	B.F.
PDS1B-4	1	1.94	0.37	9.25	57.08	3132	53.0	280	1230	58420	526	4.83	B.F.
PDS1B-5	1	1.94	0.37	10.00	61.70	3132	53.0	280	1218	57760	234	4.41	B.F.
PDS1B-5	1	1.94	0.37	10.00	61.70	3132	53.0	280	1200	57000	231	4.36	B.F.
PDS1B-6	1	1.94	0.37	10.47	64.60	3132	53.0	280	1240	58900	228	4.30	B.F.
PDS1B-6	1	1.94	0.37	10.47	64.60	3132	53.0	280	1250	59390	229	4.31	B.F.
PDS1B-6	1	1.94	0.37	10.47	64.60	3132	53.0	280	1240	58900	228	4.30	B.F.
PDS1B-7	1	1.94	0.37	10.81	66.80	3132	53.0	280	1235	58600	219	4.13	B.F.
PDS1B-7	1	1.94	0.37	10.88	67.20	3132	53.0	280	1220	57950	208	3.92	B.F.
PDS1B-7	1	1.94	0.37	10.91	67.48	3132	53.0	280	1240	58900	198	3.73	B.F.
PDS1B-8	1	1.94	0.37	13.00	80.25	3132	53.0	285	1185	56500	176	3.32	B.F.
PDS1Z-9	1	1.94	0.37	16.20	100.00	2769	52.2	255	1100	52200	131	2.51	B.F.
PDS1Z-9	1	1.94	0.37	16.20	100.00	2769	52.2	255	1265	60080	150	2.87	B.F.
PDS1Z-9	1	1.94	0.37	16.20	100.00	2769	52.2	255	1300	61700	154	2.95	B.F.
PDS1Z-10	1	1.94	0.37	18.06	115.00	2769	52.2	255	1290	61250	136	2.62	B.F.
PDS1Z-11	1	1.94	0.37	21.06	130.00	2769	52.2	255	1285	61000	115	2.22	B.F.
PDS1Z-12	1	1.94	0.37	24.30	150.00	2769	52.2	255	1260	59800	98	1.88	B.F.
PDS1Z-13	1	1.94	0.37	27.54	170.00	2769	52.2	255	1300	61700	91	1.79	B.F.
PDS1Z-14	1	1.94	0.37	29.16	180.00	2769	52.2	255	1292	61390	85.3	1.67	B.F.
PDS1Z-15	1	1.94	0.37	31.00	191.30	2769	52.2	255	1320	62700	83.2	1.59	B.F.
PDS1X-16 (R)	1	1.94	0.37	6.48	40.00	3184	56.4	282	1330	63120	395	7.01	B.F.
PDS1X-16 (R)	1	1.94	0.37	6.48	40.00	3184	56.4	282	1290	61250	383	6.80	B.F.
PDS1Y-16 (R)	1	1.94	0.37	6.48	40.00	3001	54.4	272	1310	62200	388	6.88	B.F.

APPENDIX (B)

BOND BEAM TEST RESULTS

TABLE 13(1)

UNIVERSITY OF TEXAS BEAM TEST RESULT

Specimen No.	Kind of wire	b (in.)	t (in.)	x (in.)	y (in.)	z (in.)	d' (in.)	f'_c (in.)	$f'^{0.5}_c$	No. of Stirrups in L region+	No. of Stirrups in R region+	No. of Stirrups in C region
BPF1A'-1	P-2	1	1.94	3.85	7.5	1.5	0.37	3339	57.9	6	2	-
BPF1A'-1	P-2	1	1.94	3.85	7.5	1.5	0.37	3339	57.9	6	2	-
BPF1A'-1	P-2	1	1.94	3.85	7.5	1.5	0.37	3339	57.9	6	2	-
BPF1F'-2(R)*	P-2	1	1.94	3.85	7.5	1.5	0.37	3184	56.4	6	2	-
BPF1F'-2(R)	P-2	1	1.94	3.85	7.5	1.5	0.37	3184	56.4	6	2	-
BPF1G'-2(R)	P-2	1	1.94	3.85	7.5	1.5	0.37	3001	54.4	6	2	-
BDF1D'-3	D-2	1	1.94	3.57	7.5	1.5	0.37	3418	58.2	5	2	-
BDF1D'-5	D-2.5	1	1.94	3.57	2.5	1.5	0.37	3418	58.2	5	2	-
BDF1D'-5	D-2.5	1	1.94	3.5	2.5	1.5	0.37	3418	58.2	5	2	-
BDF1F'-6(R)	D-2	1	1.94	3.57	2.5	1.5	0.37	3184	56.4	5	2	-
BDF1F'-6(R)	D-2	1	1.94	3.57	2.5	1.5	0.37	3184	56.4	5	2	-
BDF1E'-7(R)	D-3.5	1	1.94	3.77	3	1.5	0.37	2972	54.2	6	2	-
BDF1E'-7(R)	D-3.5	1	1.94	3.77	3	1.5	0.37	2972	54.2	6	2	-
BDF1F'-8(R)	D-2	1	1.94	4.60	4	1	0.37	3184	56.4	6	1	-
BDF1C'-9	D-2	1	1.94	5.93	5.5	1	0.37	2912	54.0	8	1	-

* (R) means rusted bar

+ See figure 30.

TABLE 13(2)

UNIVERSITY OF TEXAS BEAM TEST RESULTS

Specimen No.	P* (lbs.)	P** (lbs.)	n	Formula for P (lbs.)	P (lbs.)	M_{test} = Px (in.-lbs)	L" (in.)	D (in.)	L"/D	f_s (psi)	U*** (psi)	Type of Failure
BPF1A'-1	700	10.8	1	$\frac{1}{2}(P^*+P^{**})$	355.4	1368	6.48	0.162	40	33140	332	B.F.
BPF1A'-1	738	10.8	1	$\frac{1}{2}(P^*+P^{**})$	374.4	-	6.48	0.162	40			S.D.T.
BPF1F'-2(R)	760	10.8	1	$\frac{1}{2}(P^*+P^{**})$	385.4	1483	6.48	0.162	40			B.F.&D.T.
BPF1F'-2(R)	691	11	1	$\frac{1}{2}(P^*+P^{**})$	351.1	1350	6.48	0.162	40	52179	326	B.F.
BPF1F'-2(R)	731	11	1	$\frac{1}{2}(P^*+P^{**})$	371.1	1429	6.48	0.162	40			B.F.
BPF1G'-2(R)	701	11	1	$\frac{1}{2}(P^*+P^{**})$	356.1	1371	6.48	0.162	40			B.F.
BDF1D'-3	1443.2	1.85	3.5	$\frac{1}{4.5}P^*+\frac{1}{2}P^{**}$	321.6	1148	1.62	0.159	10.19	42718	1049	B.F.&C.S.
BDF1D'-5	1681.2	1.85	3.5	$\frac{1}{4.5}P^*+\frac{1}{2}P^{**}$	374.5	1337	1.75	0.178	9.84	41988	1067	B.F.&C.S.
BDF1D'-5	1581.2	1.85	3	$\frac{1}{4}P^*+\frac{1}{2}P^{**}$	398.0	1393	1.75	0.178	9.84			B.F.&D.T.
BDF1F'-6(R)	1451.1	4.6	3.5	$\frac{1}{4.5}P^*+\frac{1}{2}P^{**}$	324.8	1159	1.62	0.159	10.18	41417	1017	B.F.&C.S.
BDF1F'-6(R)	1321.1	4.6	3.5	$\frac{1}{4.5}P^*+\frac{1}{2}P^{**}$	295.9	1056	1.62	0.159	10.18			B.F.&C.S.
BDF1E'-7(R)	851.2	2.8	3	$\frac{1}{4}P^*+\frac{1}{2}P^{**}$	214.2	807.6	2.05	0.212	9.75	17645	452	B.F.&C.S.
BDF1E'-7(R)	883.2	2.8	3	$\frac{1}{4}P^*+\frac{1}{2}P^{**}$	220.8	837.7	2.05	0.212	9.75			B.F.&C.S.
BDF1F'-8(R)	901	11	2.5	$\frac{1}{3.5}P^*+\frac{1}{2}P^{**}$	-	-	3.24	0.159	20.2	-	-	S.D.T.
BDF1C'-9	1006.2	3.77	2	$\frac{1}{3}P^*+\frac{1}{2}P^{**}$	-	-	4.86	0.159	30.6	-	-	S.D.T.

+U*** Calculated using the equation (13) or (13A).

TABLE 14(1)

UNIVERSITY OF TEXAS BEAM TEST RESULTS

Specimen No.	Kind of wire	b (in.)	t (in.)	x (in.)	y (in.)	z (in.)	d' (in.)	f'_c (psi)	$f'^{0.5}_c$	No. of Stirrups in L region+	No. of Stirrups in R region+	No. of Stirrups in C region+
BDF1E'-10	D-4	1	1.94	4.12	3	1.5	0.37	2972	54.2	6	2	-
BDF1F'-11(R)*	D-4	1	1.94	3.2	2.5	1	0.37	3184	56.4	6	2	-
BDF1C'-12	D-4	1	1.94	4.09	5	1.2	0.37	2912	54	6	2	6
BDF1C'-12	D-4	1	1.94	4.09	5	1.2	0.37	2912	54	6	2	6
BDF1C'-13	D-4	1	1.94	4.09	5	1.2	0.37	2912	54	6	2	6
BDF1C'-13	D-4	1	1.94	4.09	5	1.2	0.37	2912	54	6	2	6
BDF1D'-14	D-2.5	1	1.94	6.04	6	1	0.37	3418	58.2	8	1	-
BDF1D'-15	D-3	1	1.94	3.57	2.5	1.5	0.37	3418	58.2	5	2	-
BDF1D'-16	D-3	1	1.94	5.13	4.2	1	0.37	3418	58.2	7	2	-
BDF1E'-17(H)**	D-3	1	1.94	3.95	2.5	1.5	0.37	2972	54.2	6	2	-
BDF1E'-17(H)	D-3	1	1.94	3.95	2.5	1.5	0.37	2972	54.2	6	2	-
BDF1E'-18	D-2.5	1	1.94	3.5	2.5	1.5	0.30	2972	54.2	5	2	-
BDF1E'-19	D-2.5	1	1.94	4.67	4	1	0.37	2972	54.2	6	5	-
BDF1E'-19	D-2.5	1	1.94	4.67	4	1	0.37	2972	54.2	6	5	-
BDF1E'-20	D-2.5	1	1.94	4.67	4	1	0.30	2972	54.2	6	5	-

*(R) means rusted bar, ** (H) indicates end anchorages at P.I., + See figure 30.

TABLE 14(2)

UNIVERSITY OF TEXAS BEAM TEST RESULTS

Specimen No.	P* (lbs.)	P** (lbs.)	n	Formula for P (lbs.)	P (lbs.)	M_{test}= Px (in.-lb.)	L" (in.)	D (in.)	L"/D	f_s (psi)	U*** (psi)	Type of Failure
BDF1E'-10	661.2	3.77	3	$\frac{1}{4}P^*+\frac{1}{2}P^{**}$	-	-	2.15	0.225	9.56	-	-	S.D.T.
BDF1F'-11(R)	771.1	4.62	2	$\frac{1}{3}P^*+\frac{1}{2}P^{**}$	-	-	2.15	0.225	9.56	-	-	S.D.T.
BDF1C'-12	1031.2	2.81	1.5	$\frac{1}{2.5}P^*+\frac{1}{2}P^{**}$	-	-	4.30	0.225	19.12	-	-	S.D.T.
BDF1C'-12	1041.2	2.81	1.5	$\frac{1}{2.5}P^*+\frac{1}{2}P^{**}$	-	-	4.30	0.225	19.12	-	-	S.D.T.
BDF1C'-13	926.2	2.81	1.5	$\frac{1}{2.5}P^*+\frac{1}{2}P^{**}$	-	-	4.30	0.225	19.12	-	-	S.D.T.
BDF1C'-13	991.2	2.81	1.5	$\frac{1}{2.5}P^*+\frac{1}{2}P^{**}$	-	-	4.30	0.225	19.12	-	-	S.D.T.
BDF1D'-14	1216.2	3.77	2	$\frac{1}{3}P^*+\frac{1}{2}P^{**}$	-	-	5.26	0.178	29.55	-	-	S.D.T.
BDF1D'-15	1471.2	1.85	3	$\frac{1}{4.5}P^*+\frac{1}{2}P^{**}$	-	-	1.87	0.195	9.59	-	-	S.D.T.
BDF1D'-16	1131.2	2.81	2	$\frac{1}{3}P^*+\frac{1}{2}P^{**}$	-	-	3.74	0.195	19.17	-	-	S.D.T.
BDF1E'-17(H)	1592.3	4.53	3	$\frac{1}{4}P^*+\frac{1}{2}P^{**}$	400.3	1581.4	1.87	0.195	9.59	37901.4	988	B.F.&C.S.
BDF1F'-17(H)	1262.3	4.53	3	$\frac{1}{4}P^*+\frac{1}{2}P^{**}$	317.9	1225.6	1.87	0.195	9.59			B.F.&C.S.
BDF1E'-18	957.3	4.53	3	$\frac{1}{4}P^*+\frac{1}{2}P^{**}$	241.6	845.6	1.75	0.178	9.84	23581.5	599	B.F.&C.S.
BDF1E'-19	802.3	10.8	2	$\frac{1}{3}P^*+\frac{1}{2}P^{**}$	-	-	3.5	0.178	19.71	-	-	S.D.T.
BDF1E'-19	902.3	10.8	2	$\frac{1}{3}P^*+\frac{1}{2}P^{**}$	-	-	3.5	0.178	19.71	-	-	S.D.T.
BDF1E'-19	727.3	10.8	2	$\frac{1}{3}P^*+\frac{1}{2}P^{**}$	-	-	3.5	0.178	19.71	-	-	S.D.T.

TABLE 15 (1)

SYMMETRICAL BOND BEAM TEST RESULTS

Specimen No.	Kind of wire	b (in.)	t (in.)	d (in.)	f'_c (psi)	$f'^{0.5}_c$	No. of Stirrups in L_1 region*	No.of Stirrups in L_2 region*	No.of Stirrups in R_2 region*	No.of Stirrups in R_1 region*
BPS1B'-1	P-2	1	1.94	0.37	3261	56.6	9	-	-	9
BPS1B'-1	P-2	1	1.94	0.37	3261	56.6	9	-	-	9
BPS1B'-1	P-2	1	1.94	0.37	3261	56.6	9	-	-	9
BDS1B'-2	D-2	1	1.94	0.37	3261	56.6	7	-	-	7
BDS1B'-2	D-2	1	1.94	0.37	3261	56.6	7	-	-	7
BDS1B'-2	D-2	1	1.94	0.37	3261	56.6	7	-	-	7
BDS1G'-3	D-2	1	1.94	0.37	3001	54.4	2	-	-	2
BDS1G'-3	D-2	1	1.94	0.37	3001	54.4	2	-	-	2
BDS1F'-4	D-2	1	1.94	0.37	3184	56.4	5	-	-	5
BDS1F'-4	D-2	1	1.94	0.37	3184	56.4	5	-	-	5

* See Figure 28.

TABLE 15(2)

SYMMETRICAL BOND BEAM TEST RESULTS

Specimen No.	P* + P** (lbs.)	$P=\frac{P^{*}+P^{**}}{4}$ (lbs.)	L" (in.)	M_{test}=PL" (in-lb.)	D (in.)	L"/D	f_s (psi)	U*** (psi)	Type of Failure
BPS1B'-1	741.8	185.4	6.48	1202	0.162	40	43176	270	B.F.
BPS1B'-1	723.8	180.9	6.48	1173	0.162	40			B.F.
BPS1B'-1	561.8	140.4	6.48	912	0.162	40	-	-	S.D.T.
BDS1B'-2	1290	322.5	4.77	1536	0.159	33.3	-	-	S.D.T.
BDS1B'-2	1250	312.5	4.77	1491	0.159	33.3	-	-	S.D.T.
BDS1B'-2	1320	330.0	4.77	1574	0.159	33.3	-	-	S.D.T.
BDS1G'-3	1879.3	469.6	1.62	760.8	0.159	10.19	28623	702	B.F. & C.S.
BDS1G'-3	2019.3	504.8	1.62	817.8	0.159	10.19			B.F. & C.S.
BDS1F'-4	1255.7	313.9	3.18	-	0.159	20	-	-	S.D.T.
BDS1F'-4	1205.7	301.4	3.18	-	0.159	20	-	-	S.D.T.

TABLE 16(1)

SYMMETRICAL BOND BEAM RESULTS

Specimen No.	Kind of Wire	b (in.)	t (in.)	d' (in.)	f'_c (psi)	$f'^{0.5}_c$	No. of Stirrups in L_1 region+	No. of Stirrups in L_2 region+	No. of Stirrups in R_2 region+	No. of Stirrups in R_1 region+
BDS1F'-5	D-4	1	1.94	0.37	3184	56.4	3	-	-	3
BDS1F'-5	D-4	1	1.94	0.37	3184	56.4	3	-	-	3
BDS1F'-6	D-4	1	1.94	0.37	3184	56.4	6	-	-	6
BDS1F'-6	D-4	1	1.94	0.37	3184	56.4	6	-	-	6
BDS1F'-7	D-4	1	1.94	0.37	3184	56.4	6	-	-	6
BDS1F'-8	D-4	1	1.94	0.37	3184	56.4	6	6	6	6
BDS1F'-8	D-4	1	1.94	0.37	3184	56.4	6	6	6	6
BDS1H'-9	D-2	1	1.94	0.37	2905	53.9	8	8*	8*	8

\+ See figure 28.

* Using open vertical stirrups.

TABLE 16(2)

SYMMETRICAL BOND BEAM TEST RESULTS

Specimen No.	P*+P** (lbs.)	$P=\frac{P^{*}+P^{**}}{4}$ (lbs.)	L" (in.)	M_{test} =PL" (in.-lb.)	D (in.)	L"/D	f_s (psi)	U** (psi)	Type of Failure
BDS1F'-5	1679.3	419.8	2.25	-	0.225	10	-	-	S.D.T.
BDS1F'-5	1744.3	436.1	2.25	-	0.225	10	-	-	S.D.T.
BDS1F'-6	1245.7	311.4	4.5	-	0.225	20	-	-	S.D.T.
BDS1F'-6	977.7	244.4	4.5	-	0.225	20	-	-	S.D.T.
BDS2F'-7	1075.5	268.9	4.5	-	0.225	20	-	-	S.D.T.
BDS1F'-8	1365.7	341.2	4.5	-	0.225	20	-	-	S.D.T.
BDS1F'-8	950.7	237.7	4.5	-	0.225	20	-	-	S.D.T.
BDS1H'-9	1822	455.5	3.38	1539.6	0.159	21.26	64317.5	756	B.F.&C.S.

APPENDIX (C)

MATERIALS

C.1 STEEL BARS

A. Tension Reinforcement:

(a) Deformed Bars:

The following indented mild steel wires (ASTM A496-64) were used in this investigation:

Type of Bars	Nominal Diameter	Area of Cross-section (in^2.)
D-2	0.159	0.02
D-2.5	0.178	0.025
D-3	0.195	0.03
D-3.5	0.212	0.035
D-4	0.225	0.04

(b) Plain Bars:

The plain bar used for reinforcing both the eccentric pull-out and the bond beam specimens consisted of 0.162 in. nominal diameter soft steel wire.

B. Compression Bars and Stirrups:

Plain soft steel wire (0.065 in. nominal diameter and 0.00332 in.^{2} nominal cross-sectional area) were used for both the compression reinforcement and the stirrups.

C. Tensile Testing Machine:

An Instron Universal Machine with a capacity of 20,000 lbs and a smallest division of 0.1 lbs. was used for tension tests on steel wires.

D. Stress-Strain Diagrams:

All deformed and 0.162 in. diameter plain reinforcing wires exhibited a well defined yield point while the yield point of 0.065 in. diameter plain steel wire was obtained by determining the steel stress corresponding to a 0.2% offset strain (accepted as the "Offset Yield Point"). A line was drawn from a strain value of 0.2% parallel to the initial elastic part of the strain-stress curve to intersect the curve in the "Offset Yield Point".

TABLE 17

Type of Bar: Plain Bar
Bar Diameter: 0.065 in.
Cross Sectional Area: 0.0332 in.^{2}
Material: Mild Steel

Spe. No.	Yield load lb.	Ultimate load lb.	Total Elongation in.
1	202	230	0.648
2	200	228	0.864
3	202	231	0.688
Average	201.3	229.7	0.733

Type of Bar: Plain Bar
Bar Diameter: 0.162 in.
Cross Sectional Area: 0.0206 in.^{2}
Material: Mild Steel

Spe. No.	Yield load lb.	Ultimate load lb.	Total Elongation in.
1	1368	1597	0.880
2	1365	1590	0.908
3	1362	1590	0.856
4	1334	1570	0.840
5	1352	1570	0.960
Average	1356.2	1585.4	0.8888

Type of Bar: D-2
Bar Diameter: 0.159 in.
Cross Sectional Area: 0.02 in.^{2}
Material: A.S.T.M. A496-64 Steel

Spe. No.	Yield load lb.	Ultimate load lb.	Total Elongation in.
1	1580	1652	0.454
2	1520	1601	0.512
3	1543	1630	0.516
4	1458	1543	0.584
Average	1525.3	1606.5	0.5165

TABLE 17
(CONTINUED)

Type of Bar: D-2.5
Bar Diameter: 0.178 in.
Cross Sectional Area: 0.025in.^{2}
Material: A.S.T.M. A496-64 Steel

Spe. No.	Yield load lb.	Ultimate load lb.	Total Elongation in.
1	1820	2010	0.472
2	1690	1970	0.688
3	1720	1975	0.480
4	1960	2200	0.704
5	1965	2200	0.480
Average	1831	2071	0.5648

Type of Bar: D-3
Bar Diameter: 0.195 in.
Cross Sectional Area: 0.03in.^{2}
Material: A.S.T.M. A496-64 Steel

Spe. No.	Yield load lb.	Ultimate load lb.	Total Elongation in.
1	1900	2090	0.488
2	1800	1965	0.304
3	1810	1950	0.688
4	1900	2150	0.520
5	1825	2070	0.480
Average	1847	2045	0.496

Type of Bar: D-3.5
Bar Diameter: 0.212in.
Cross Sectional Area: 0.035 in.^{2}
Material: A.S.T.M. A496-64 Steel

Spe. No.	Yield load lb.	Ultimate load lb.	Total Elongation in.
1	2775	2965	0.380
2	2700	2975	0.600
3	2640	2950	0.648
4	2630	2840	0.456
5	2650	2925	0.600
Average	2679	2931	0.5368

Type of Bar: D-4
Bar Diameter: 0.225 in.
Cross Sectional Area: 0.04 in.^{2}
Material: A.S.T.M. A496-64 Steel

Spe. No.	Yield load lb.	Ultimate load lb.	Total Elongation in.
1	2600	2790	0.50
2	2540	2740	0.62
3	2410	2640	0.46
4	2625	2800	0.612
5	2550	2750	0.700
Average	2545	2744	0.5784

FIG. A. D-2

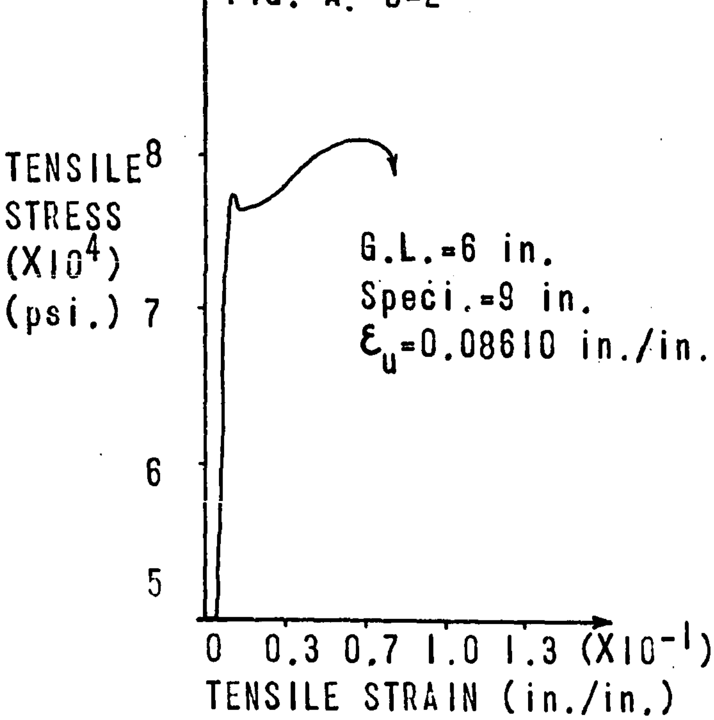
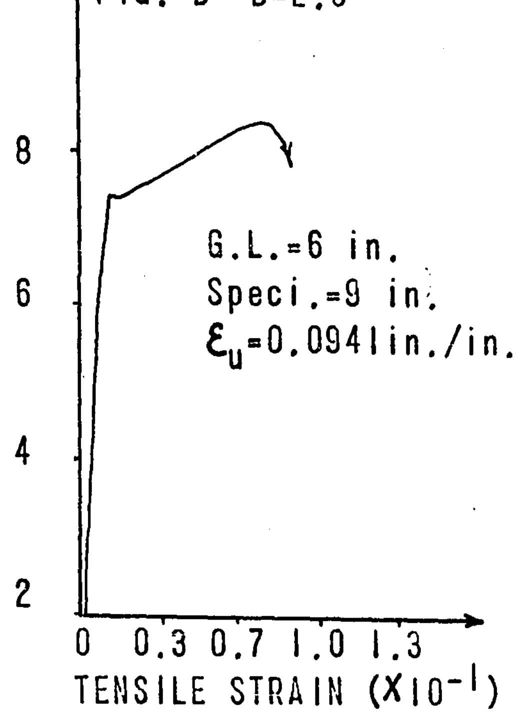


FIG. B D-2.5



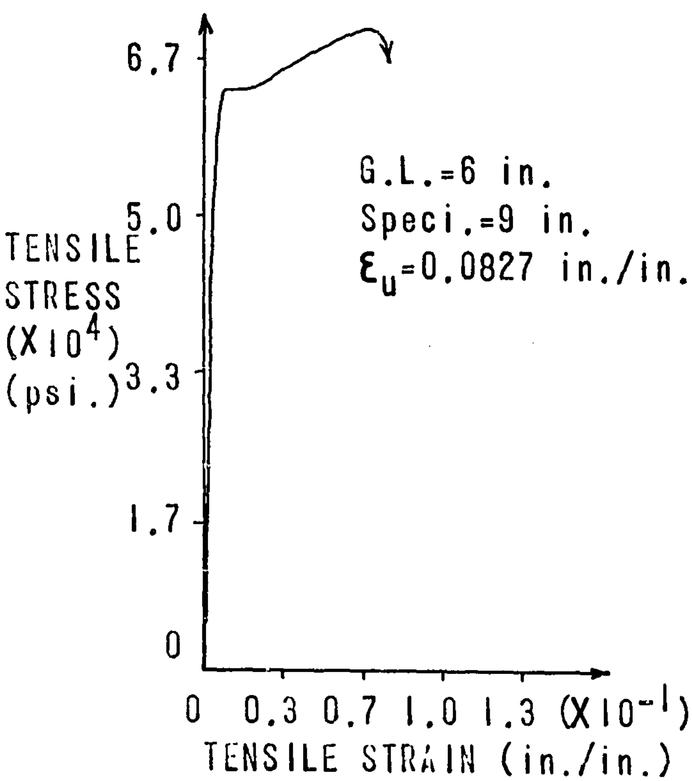


FIG. C D-3

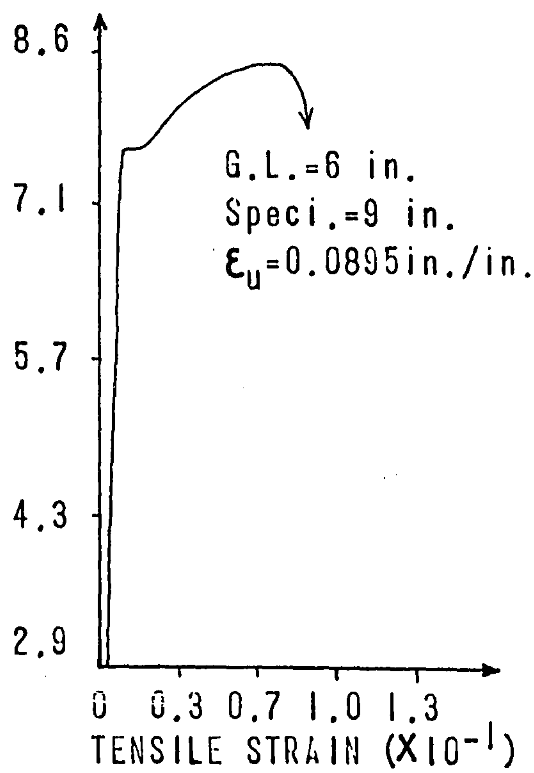
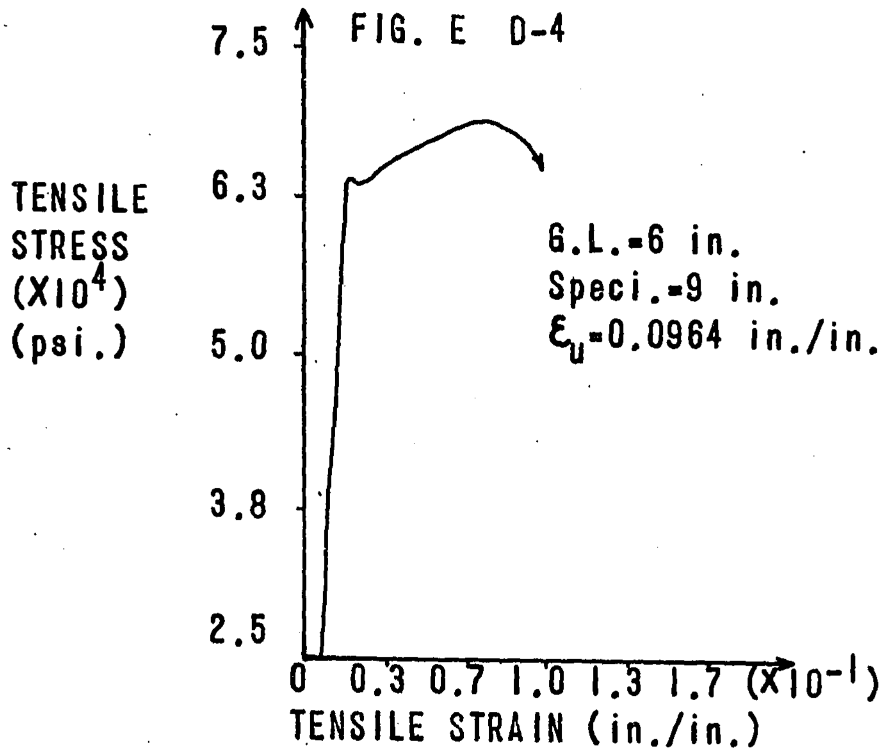


FIG. D D-3.5



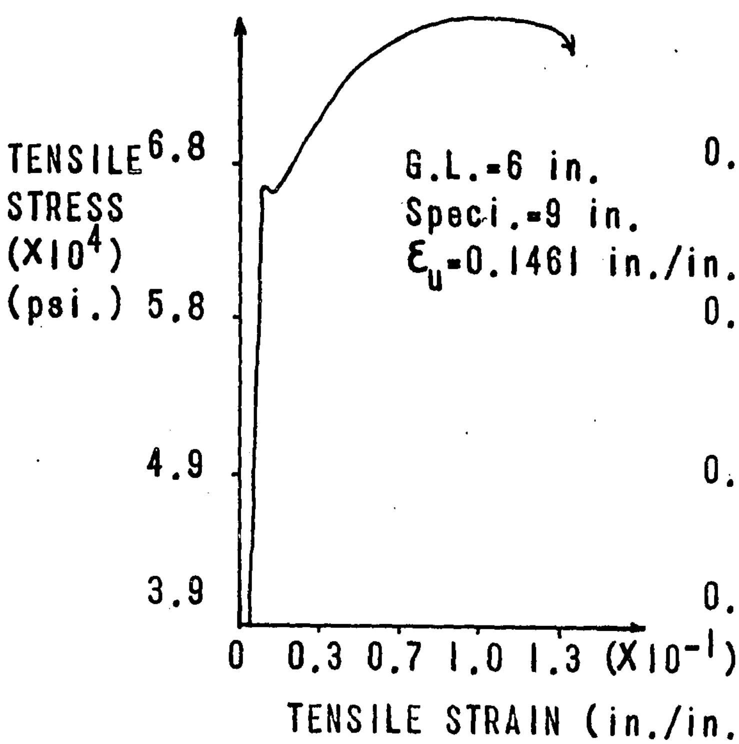


FIG. F PLAIN BAR
(D=0.162 in.)

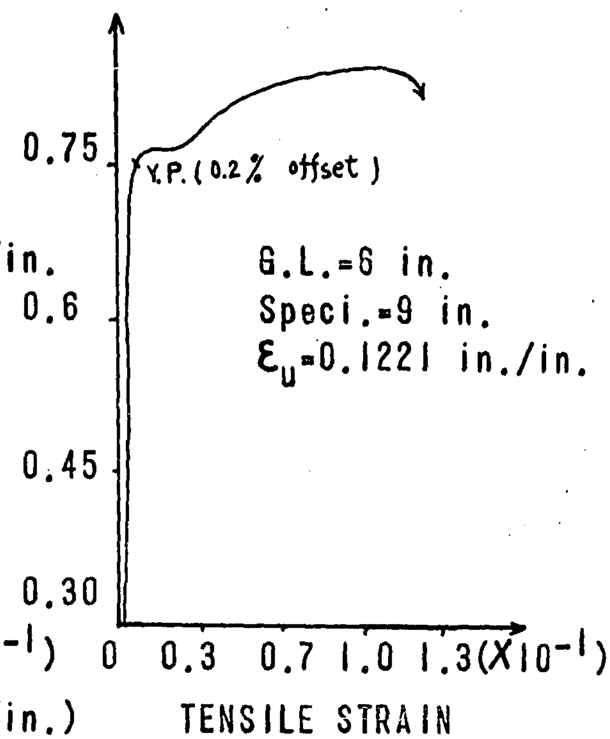


FIG. G PLAIN BAR
(D=0.065 in.)

C.2 CONCRETE (MICRO-CONCRETE)

A. Micro-concrete Mix and Properties:

The following mixes were used in this investigation. All proportions specified below are by weight. Figures within parenthesis indicate the weights of Cement and aggregates and the volume of water used for the mix per batch. Eight days was used as the standard curing period.

	High Early Strength Cement	Water	Sand	Nominal Concrete Strength f'_c (psi)
(1)	1 (4.62 lbs.)	0.795 (1667 c.c.)	3.25 (15 lbs.)	3000
(2)	1 (4.17 lbs.)	0.7 (1324.4 c.c.)	3.6 (15 lbs.)	4000
(3)	1 (6 lbs)	0.475 (1288 c.c.)	2.5 (15 lbs)	5000

A mixture of crushed quartz sand was used as aggregates in the following proportions for each batch of 15 lbs:

Sieve Size No. 10	3 lbs
Sieve Size No. 16	3 lbs
Sieve Size No. 24	3.75 lbs
Sieve Size No. 35 (act.No.40)	3.75 lbs
Sieve Size No. 70	1.50 lbs
Total	15.00 lbs

B. All test specimens and the accompanying control cylinders (3" x 6") were tested in a 60,000 lbs Riehle Universal Testing Machine (least count = 2 lbs).