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EFFECT OF CORROSION AND BAR SPACING ON BOND PROPERTIES OF

REINFORCING BARS IN CONCRETE

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To My Parents

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ABSTRACT

An integrated study of the effects of corrosion and bar spacing on bond strength of intermediate grade reinforcing bars in concrete was undertaken. Bars of various sizes conforming to ASTM A-305-56T were exposed to three different corrosive environmental conditions: (1) normal out-of-doors, (2) moist room (100 % relative humidity), and (3) simulated sea water spray, for exposure times varying up to 12 months. The effects of the presence of corrosion on the tensile strength and its associated influence on the splitting strength of the concrete (bar spacing) and bond strength (adhesion, friction, and lug action) were determined by 115 comparative eccentric bond pullout tests.

The results indicated some influence of corrosion on bond properties of the modern deformed bars, but no definite trend could be established due to the normal test scatter. This scatter was not totally unexpected because the ultimate strength of the specimens was primaryly controlled by the tensile strength of concrete. Also, bond strength was reduced by decreasing bar spacing.

An ultimate bond strength equation for deformed bars is suggested. Recommendations are made for future research.

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CHAPTER I

INTRODUCTION

1.1 General

The bond between concrete and a reinforcing bar can be thought of as that property which causes hardened concrete to grip an embedded steel bar and thus prevent the longitudinal sliding of the reinforcing bar through the concrete. The effective interaction between steel and concrete can exist only because of this property. Bond stress is a measure of "bond" and it is generally considered to be the unit longitudinal shear stress acting parallel to the bar at the contact surface between the bar and the concrete.

The bond between concrete and reinforcement is usually considered to consist of three components : (a) chemical adhesion, (b) friction, and (c) mechanical interaction between concrete and reinforcement.

It has been fairly well established that the bond strength of plain bars without lugs (surface deformation) depends primarily on adhesion, friction and to a very small extent on mechanical interlocking due to natural roughness of the bar surface. The present bar deformations (ASTM A-305) have been devised to reduce possible slip between the bar and concrete and to increase bond strength of such bars. Therefor tests have indicated that the bond strength of deformed bars is generated primarily by the mechanical interlocking of the lugs and concrete. Traditionally, past investigators of bond strength of deformed reinforcing bars have assumed a small additional bond resistance due to chemical adhesion and friction to exist between reinforcing bars and concrete. Since the experimental techniques necessary to substantiate or refute this assumption have not been developed, the significance of these forces in the case of deformed bars is debatable.

Although the new deformed bars have solved some of the bond failure problems of the plain bars the deformations have let to a new problem of significant proportions. As mentioned before, bond strength of deformed bars mostly depends upon the bearing of lugs upon concrete and upon shearing forces thus induced in the concrete between the lugs. These shearing forces set up radial compressive forces which produce tension splitting forces in the concrete thereby leading to a possible reduction in bond strength. Strictly speaking, a splitting failure is not the same phenomena as a bond failure. It would be desirable to set up a separate criterion against splitting failure. But to date, little experimental data are available on splitting forces.

The fact that the bond mechanism is complicated and bond strength is influenced by factors such as diagonal tension cracks, shear, and splitting forces, caused past investigators of the bond properties of reinforcing bars in concrete not to include all the parameters affecting bond strength, rather most have used binding devices to prevent certain failures of concrete not included in their investigations. In the case of pullout tests, spiral wire has been embedded in the concrete to prevent splitting of the concrete blocks have been cast of such size that the plain concrete could resist the splitting forces. In the case of beam tests, beam reactions have provided confining compressive forces across potential splitting planes. On the other hand, it is logical to assume that bond strength is not only a function of lug action (and possibly adhesion and friction) but also a function of confinement and spacing of reinforcing bars in concrete. Thus the effect of variations in bar spacing on the bond strength of reinforcing bars in concrete members is a weak spot in our knowledge of the mechanism of bond.

One of the old, unsolved, and still controversial problems relating to bond strength of both plain and deformed bars is the effect of corrosion of reinforcing bars (commonly referred to as rust) . Generally speaking, current practice tolerates a limited amount of corosion, but the element of personal judgement in classifying and deciding when the corrosion is excessive has been a source of controversy among builders and inspectors. Assuming that the classification and measurement of corrosion were possible, the fact remains that a state of confusion exists concerning the influence of any specific kind of degree of corrosion on bond resistance. This investigation is concerned with how each of the following types of corrosion on reinforcing bars influence the bond strength that they can develop as reinforcement: (a) early-stage corrosion; a thin, loose layer which rubs off easily, (b) intermediate-stage corrosion; a fairly thick, firm layer of corrosion, removable by rubbing, (c) late-stage corrosion; very thick, multiple layers of corrosion, the outer portion being loose, "flaky" and easily removable.

1.2 Survey of Previous Research

Early works of Withey (15), Abrams (16) and Shank (17) on the effects of corrosion on bond properties of reinforcing bars indicate that firm corrosion improves rather than weakens the bond because of its surface-roughening effect. Gilkey (18) concluded that "flaky" rust lowers the bond resistance, while Cox (19) and Kemp et al. (20) observed that ultimate pullout strength of the deformed bars was not greatly affected by their condition of corrosion and that bond properties of corroded bars did not appear to be improved or impaired by different degrees of corrosion. Abe (24) demonstrated the fact that corroded wires used in prestressed concrete showed less slip and higher bond strength than clean wires.

There is a difference of opinion among different investigators concerning the efficiency of bundled bars. Hadley (3,4), Boase (5) and an unidentified author (6) reported successful experience with bundling whereas Walker's tests (7) showed that beams reinforced with tied bars failed at slightly lower loads than those with spaced bars, that the ultimate failure was due to loss of bond, and that center deflection and end slip were less for beams with spaced bars than for beams with bundled bars. Hanson (8) confirmed the findings of Hadley (3,4). Ferguson et al. (10) concluded that the bond strength is lower for closely spaced bars in concrete, and in their tests the eccentric pull-out specimens with small bar spacings usually failed without any prior slip. Yee (12) compared the bond strength of bundled and spaced bars embedded in concrete. He concluded that the slip at the loaded-end was usually less for bundled bars than for spaced bars. He also found that the percentage increase in load for spaced bars as compared to three bundled bars based on the maximum load was 31.4 percent. This increase in load was higher than the theoretical 20 percent increase due to a corresponding increase of bond surface for the spaced bars as compared to the three bundled bars. Chamberlin (11) tested a number of beams reinforced with spaced and bundled bars. He concluded that the load-carrying capacity of the beams was higher for large bar spacing except in those cases where failure was in the steel. For all the bar spacings slippage was usually, but not significantly, reduced with the increase in bar spacing. Chamberlin suggested that the spacing of bars other than bundled bars did not appear to affect bond significantly.

A more detailed discussion of the previous research will be presented in the following chapter.

1.3 Bond Properties Investigated in the Present Study

1.3.1 Scope and Objectives

The objectives of this research have been the determination of: (a) the effect of corrosion, ill or favorable, on the bond strength of intermediate grade reinforcing bars and, (b) the effect of bar spacings on bond strength. The principal motivation for this study is the need for experimental data obtained from the tests of reinforced concrete specimens to establish the above mentioned effects and to provide engineers and field inspectors with a better understanding of the effects of corrosion and bar spacing on bond strength of reinforcing bars.

The results of the experimental program undertaken are limited to the study of eccentric pullout specimens which were carefully designed to closely simulate the flexural, diagonal, and bond stress conditions in flexural members. The exposure conditions studied in this investigation (sea water, outdoor, and indoor) are assumed to be representative of the existing corrosive environments.

1.3.2 Variables Studied

The primary variables for this research are:

(a) Different periods of exposure to corrosive environments,

(b) Different corrosive environments,

(c) Bar spacing, and

(d) Bar size.

1.4 Notation

The following notation is used in this dissertation:

a - Eccentric pullout specimen with 2 spaced bars.

b - " " " " 2 adjacent tied bars.

c,d,e " " " " 1 bar

f'_c - Adjusted concrete compressive strength.

f''_c - Concrete compressive strength of 4 x 8 inch cylinders at the time of testing.

f_s - Steel stress, ksi.

f_{sp} - Equivalent concrete tensile stress, split cylinder test.

h - Eccentric pullout specimen's height.

s - Loaded end slip.

w - Eccentric pullout specimen's width.

A - As-rolled

D - Nominal bar diameter.

D_1 - Bar diameter between the lugs, with mill scale.

D_2 - Bar diameter between the lugs, after one cycle of chemical pickling.

E_s - Modulus of elasticity of the reinforcing bars.

L - Length of reinforcing bar control coupons.

L" - Embedment length.

M - Exposure environment, indoor.

N - " " , outdoor.

S - " " , sea water.

T - Eccentric pullout specimen's failure load.

$T_{adj.}$ " " " " " , adjusted for concrete compressive strength.

$u_{adj.}$ Bond stress, adjusted for concrete compressive strength.

u_u - Ultimate bond stress.

V - Coefficient of variation.

W_b - Weight of control coupon, brushed.

W_i - Initial weight of control coupon, with mill scale.

W_f - Final weight of control coupon, mill scale removed.

W_1 - Weight of control coupon after one cycle of pickling.

W_2 - " " " " " two cycles of pickling.

W_4 - " " " " " four " " "

CHAPTER II

PREVIOUS RESEARCH AND CURRENT RECOMMENDED PRACTICE

2.1 Introduction

The failure modes of the reinforced concrete flexural members fall in three distinct categories : (a) flexural failure, for overreinforced members characterized by crushing of the concrete in the compression zones and for under-reinforced members by yielding of the reinforcement, (b) diagonal tension failure, which is characterized by the formation of an inclined crack between the tension and the compression faces of the member, and (c) bond failure, which in the case of present deformed bars is most generally a result of longitudinal splitting of the concrete. These mechanisms occur singly and in various combinations.

This research study was mainly concerned with the bond failure. It was also limited in the sense that the fundamental nature of bond was not the subject of this investigation, rather this study dealt with the effects of corrosion and bar spacing on the bond strength of reinforcing bars. In the following sections a review of the previous research dealing with the influence of corrosion and bar spacing on the bond characteristics of reinforcing bars is given along with current building code and recommended practices. The mechanisms of bond and corrosion of reinforcing bars will be discussed briefly.

2.2 Mechanism of Bond and Corrosion

2.2.1 Mechanism of Bond

In the field of reinforced concrete, bond strength may be defined as that property of hardened concrete which causes it to grip to an embedded reinforcing bar in such a manner as to resist forces tending to slide the reinforcing bar longitudinally through the concrete. Whenever the tensile or compressive forces in a bar change, to maintain the equilibrium, this change in bar force must be resisted at the contact surface between the bar and concrete by an equal and opposite force produced by bond between the reinforcing bar and concrete.

In the case of plain bars without surface deformations, bond strength is largely adhesive, but even after adhesion is broken by slipping of the bar, friction between the concrete and the reinforcing bar continues to provide a considerable bond resistance. Friction resistance is low for smooth bars and is higher for bars with rougher surface. Once adhesion and static friction are overcome at larger loads, small amounts of slip leads to interlocking due to the natural roughness of the bar with the concrete. However, this bond strength is low and the bar is pulled through the concrete.

The present bar deformations (ASTM A-305) have been devised to reduce possible slip between the bar and concrete and thus increase the bond strength. With such a deformation, the bond strength has been considered to depend primarily upon the bearing of the lugs upon the concrete and to a small degree, upon friction and adhesion.

2.2.2 Mechanism of Corrosion

Corrosion may be defined as the destructión or deterioration of metal by direct chemical or electrochemical reaction with the oxygen of its environment. This reaction occurs because in many environments most metals are not stable and have a tendency to revert to a more stable combination. Or, according to Evans (27), corrosion could be thought as the opposite of the chemical process in which a metal is refined from its ore. No matter how the reaction is defined, it is considered to be a function of the metal, the environment and the mechanical and physical conditions of the system under study.

Under most exposure conditions the corrosion products consist primarily of oxides, carbonates, and sulphates. Two states of oxidation are possible depending on the availability of oxygen. The first state is usually formed on the metal surface and it is considered to be ferrous hydroxide. The first layer is converted to hydrated ferric hydroxide at a short distance away from the surface of the bar, where it is in contact with more oxygen. In between these two layers there may exist combinations of the two compounds. Whenever the supply of oxygen is not adequate, however, the product may be black anhydrous magnetite or the green hydrated magnetite.

The composition of corrosion varies with corrosion environment. When a metal corrodes in the atmosphere the amount of ferrous corrosion produced is small, but when formed underwater the corrosion products contain a large proportion of ferrous iron. The subsequent corrosion process is affected by the stucture of the corrosion. If the corrosion layer is hard, dry, and adheres to the metal surface, it forms a protective film which retards the corrosion rate. While, if the layer of rust is flaky and easily removable it will continue reacting with oxygen and moisture from its environment.

2.3 Bond Strength Versus Corrosion, Time, and Bar Spacing

2.3.1 Previous Research

2.3.1.1 Bond Strength Versus Corrosion and Time. Although considerable research has been conducted dealing with the subject of bond, the experimental evidence on the influences of corrosion on bond properties of reinforcing bars has received little attention from the past investigators.

Withey (15) 1909

Withey conducted tests to determine the effects of corrosion on bond strength of smooth reinforcing bars using cocentric pullout specimens. He concluded that a "firm" hard coating of corrosion improved the bond strength of plain round reinforcing bars when compared with as-rolled bars. In similar tests Abrams (16) in 1913 substantiated Withey's conclusions.

Shank (17) 1934

Shank tested concrete beams which were reinforced with plain 1-inch-square, cold-rolled, steel bars. The bars had different surface conditions : (a) as-rolled, with mill scale, (b) rusted in the ground for 10 months, (c) weather-rusted for 10 months, (d) sand blasted and lubricated with paraffin oil. His results showed that rusting of the bar surface and subsequent sand blasting improved bond resistance. Lubricating the bar surface with paraffin oil reduced the bond strength.

Finally he concluded that firm corrosion improved the bond strength between reinforcing bars and concrete. He suggested that the improvement of bond strength was due to the roughened surface of the bars.

In the light of the fact that bond strength of smooth bars is mainly adhesive and frictional in nature, the conclusions reached by Withey, Abrams, and Shank seem reasonable, since the roughness of the bar surface due to corrosion improves both adhesive and frictional resistance of the bar to slip. With modern deformed bars, bond strength is believed to be dependent more on lug action than on adhesion and friction. Thus the conclusions of Withey, Abrams, and Shank for smooth bars are considered less significant for modern deformed bars.

Gilkey et al. (18) 1939

Plain round, 5/8-inch, rail-steel bars were exposed to the weather for 0,1,2,3,4,6,7, and 8 months respectively. Rust formation was observed and measured by removing the corrosion and weighing it. All bars were vertically cast into 4-x 4-x 10-inch, 28-day concentric pullout specimens. Amounts of slippage were measured at the loaded and unloaded ends of the bars. The authors reached the general conclusions that the light layer of loose, powdery red rust that first forms was of negligible importance and that "firm" rust tended to increase bond resistance. On the other hand, Gilkey et al. found that after the rust became deep, loose, and "flaky", bond strength was reduced slightly and wiping the corroded bars with burlap increased the bond strength. These tests showed that even for the longest exposures, which produced the deep, loose layer of rust, there was no significant reduction in the cross sectional area of the bars because of the corrosion.

In measuring the amount of corrosion the authors used two methods: (a) emery cloth was used to remove the rust and all debris (rust and/or mill scale) and they were weighed to the nearest gram, (b) the cross sections of two inch control specimens were photomicrographed (magnification ratio of 24), with half of the corroded cross section cleaned and the other half corroded. From the photomicrographs the approximate depth of the rust layer was measured.

Johnston and Cox (19) 1940

A significant series of tests made on the problem of corrosion of reinforcing bars was reported by Johnston and Cox in 1940. In this research, three different series of tests were made on various deformed bars of different sizes and different degrees of surface rust.

In the first series, 36 bars were selected from a fabricator's stock pile and tested in concentric pullout specimens. The results were inconclusive with respect to the net effect of varying degrees of corrosion upon the bond strength of corroded deformed bars. All the bars used in this series were 5/8-inch round, intermediate grade, with transverse lugs about two bar diameters apart. In the second series six 20-foot-long bars of each of the following sizes: 3/8-inch-round deformed, 1/2-inch-round deformed, 3/4-inch-round deformed, 1-inch-square deformed, and 1-1/4-inch-square deformed were cut into two-foot lengths. The bars had transverse lugs and were of intermediate grade steel. The bars were stored both in a moist-room and out-of-doors in an exposed position. The time of exposure was a variable. Concentric pullout specimens were made and tested after periods of exposure of 3,6,9,12, and 15 months, 12 months being the maximum time for the moist-room exposed bars and 15 months for the out-of-doors exposed bars. In all, some 330 tests were carried out but the results were somewhat scattered and did not exhibit the same pattern in the variation of bond strength with increasing degree of corrosion among different bar sizes. However, the investigators observed the following:

1. Rusted bars showed higher bond strength at low values of slip than unrusted bars.

2. The ultimate pullout strength of the deformed bars was not greatly affected by their conditions of corrosion.

3. The total amount of slip before reaching maximum load was usually greater for bars in the unrusted or slightly rusted condition than for those which were heavily rusted.

The third series of tests, which consisted of 45 deformed bars similar to the ones used in the first two series were exposed outdoors. This series of tests was intended as a check on the results of the second series. But the results were not consistent with the corresponding ones in the second series; the ultimate strength seemed to decrease rather than increase as in the second series with the increasing degree of corrosion.

Johnston and Cox's work is the first one reported on deformed bars up to 1940. Since 1940 the reinforcing bar deformations have changed and the conclusions reached by the authors are not directly applicable for modern deformed bars.

Up to this date (1940) all the investigators of corrosion of reinforcing bars used the concentric pullout tests on plain and semi-deformed bars, and as it was argued on Withey's work the conclusion that "firm" rust improved bond strength was understandable. However, most investigators no longer recognize the concentric pullout test as giving a realistic representation of critical bond stress stituations, since the concrete is subjected to compressive stresses in these tests.

Janney (25) 1954

The author reported a series of tests on a number of 2-x2-x 96-inch prestressed prisms with both rusted and clean smooth wires. Conrete compressive strength was 4500 psi. Corroded wires developed the full transfer of prestress at a more rapid rate and in less distance from the free end. In the testing of a number of 6-x 10-x 78-inch flexural specimens two modes of failure were observed. Beams with clean wire failed in bond, and all beams with rusted wire failed by fracture of the wires and carried a higher load. Janney concluded that the bond capacity of rusted wire is greatly superior to that of clean wire. He suggested that if pre-tension wires could be manufactured with a surface having the bonding qualities of rusted wire, it would be impossible that a flexural bond failure would occur in beams of practical dimensions. However, any set of rusting conditions which might result in a reduction in cross-sectional area of the wire should be avoided.

Abe (24) 1955

Abe tested a number of prestressed flexural members reinforced with corroded wires (rusted by nitric acid), bright and cleaned wires, and indented wires. He observed that the concrete specimens reinforced with corroded wires showed less slip and higher bond strength than the specimens reinforced with clean and indented wires.

Shermer (28) 1956

Shermer reported the failure of precast I-beams, which were exposed to warm moist air inside kilns where the only protection for the reinforcement was concrete cover. Due to corrosion, bars of 7/8-inch diameter were reduced to 1/2-inch in diameter in eight years. The failure occured under dead load. Concrete strengths ranged between 3,000 and 4,000 psi. Although corrosion was the primary cause, other factors such as high shear stress and slipping due to loss of bond were responsible for the failure.

Bureau of Reclamation (23) 1956

A series of concentric bond tests was conducted on reinforcing bars with deformations conforming to ASTM A-305-56T. Four bar surface conditions were examined : as-rolled, wire brushed, sand blasted, and burlap rubbed. The report concluded that :

(a) Corrosion is not harmful to the bond strength of reinforcing bars and that no benefit is gained by removing the corrosion from the bars. It was suggested that any reinforcing bar having what appears to be an excessive amount of corrosion be checked to see that the remaining effective cross-sectional area conforms to the specification for allowable deviation from the theoretical weight,

(b) Bond strength depends on the number and size of deformations, and,

(c) Corrosion increases the roughness of the surface of reinforcing bars. Consequently, the holding capacity of the bar is increased, although the effective area of the bar may be reduced.

Kemp, et al. (2) 1965

The latest and most up-to-date research on the effects of corrosion on bond properties of reinforcing bars was carried out by the authors. The reinforcing bars used in this investigation had deformations meeting ASTM A-305-56T specification. The principal parameter in the tests was the bar surface condition. A broad range of scale and rust conditions was studied. Because of the possibility of a splitting failure for the larger diameter bars, both No. 4 and No. 9 bars were used in the test series. A constant bond length was used for each bar size. Two companion series of specimens were cast with 3,300 and 5,600 psi concrete. In all 159 eccentric pullout specimens were tested.

The authors concluded that the bond characteristics of the deformed reinforcing bars with deformations meeting ASTM A-305-56T specification do not appear to be adversely affected by varying degrees or types of corrosion or ordinary mill scale as long as the unit weight of the bar meets the minumum ASTM weight and height of deformations requirements. It was also noted that the concrete strength appeared to control the bond behavior for a given bar size and deformation pattern to a much greater extent than the surface condition of the bar.

The authors' conclusions are questionable with respect to the method of loading they used. For instance, they attempted to eliminate the confinement of the reinforcing bar at the free end of the specimen by casting a sleeve around the bar. This may have eliminated the direct confinement of the bar, but the concrete around the bar near the free end was subjected to compressive stresses normal to reinforcement.

These compressive stresses are not normal in those cases where confinement is totally absent (see Sec. 3.1).

Summary

The past research on the effects of corrosion on bond strength of reinforcing bars could be divided into three groups. The first section (the work of the earliest investigators) involved most generally concentric pullout tests on plain reinforcing bars. These early investigators concluded that "firm" rust improved the bond strength of plain reinforcing bars as compared with as-rolled bars. They suggested that the improvement of bond strength was due to the roughened surface of the bars. They did not detect any significant reduction in the cross-sectional area of the bars because of the corrosion.

The second identifiable group of investigators conducted concentric pullout tests of the early deformed bars. These tests included a broader range of exposure durations and different corrosive environments. They concluded that the ultimate pullout strength of the deformed bars was not greatly affected by their conditions of rust.

The research conducted by Kemp et al. constitutes the third group. The major improvement of this more recent research has been the development of the eccentric pullout specimen in order to better simulate beam conditions. Another variation is represented by the changing bar deformation pattern to the current ASTM A-305. These later researchers have concluded that the bond strength of the modern deformed bars do not appear to be improved or adversely affected by varying degrees or types of corrosion, provided the weight of the bar meets the minimum ASTM requirements.

Finally, the variations between tests due to changing shapes and surface conditions of the bars and revisions of test procedures confuses efforts to correlate the results of the past research.

Several investigators have suggested that a qualitative determination of the firmness or looseness of corrosion be used as an acceptance criteria of corroded reinforcing bars. Others have attempted to measure the amount of corrosion as an aid in the formulation of a more rational decision. But the question remains : What is "firm", or "flaky" rust that some researchers have referred to in the past ? What dependable and reasonable definite basis is there for evaluating the amount or type of corrosion and its possible effects upon bond resistance ? How far can corrosion progress before it appreciably reduces the effective cross-section of the bar ?

2.3.1.2 Bond Strength Versus Bar Spacing.

Hadley (3,4) 1941

The author tested 4 beams, 6-x 12-inch and 10-foot long. Two of the beams were reinforced with a single, 1-inch-square bar extending the full length of the span. The other two beams were reinforced with the same cross-sectional area of steel obtained with four, 1/2-inch-square bars tied together (two upper bars directly above two lower bars) . The lower bars ran straight though the beams and the upper bars were bent up at 45 degrees and constituted the only web reinforcement of the beams. The concrete strength was 4,530 psi and the deformed reinforcing bars had a yield strength of 52 to 58 ksi. All four beams failed by tension in the steel under two symmetrically placed center loads

12 inches apart. Hadley concluded that bundling of reinforcing bars increased the load carrying capacity of the beams by seven percent over the beams reinforced with a single bar. Later from the tests of two hollow precast beams reinforced with bundled bars the author found no indication of weakness or trouble. The author concluded that the only thing "wrong with bundling" was that it was in violation of the spacing requirements of the building and other codes.

The word "bundled" used by Hadley is not in context with the present conotation of the work. The perimeter of four square bars used by Hadley is the same as the perimeter of one square bar used by him (the total area of the four bars being equal to the area of a single bar). While the perimeter of four bundled round deformed bars is not equal to the sum of the individual perimeters of the same bars. The author tested only four beams and the seven percent increase in load carrying capacity of beams with four tied bars over the beams reinforced with a single bar could be test scatter. The bars used in Hadley's research had pre-ASTM 305 deformations and, since the square shaped bars used in the tests are now obsolete, the quantitative extrapolation of these results is debatable.

Walker (7) 1951

Walker conducted tests of twelve beam specimens. Six of the beams were provided with 2-x 2-1/4x 8-inch blockouts for strain measurements. All of the beams had dimensions of 8-x 8-x 48-inch and were loaded at quarter points. Reinforcing bars with three different deformations types were used. One bar was a wartime product (slightly deformed), the other had more deformations, and the third conformed to

ASTM A-305-49. In addition to the beams a number of concentric pullout specimens eight inches in diameter and with reinforcing bar embedment length of eight inches were tested. Walker concluded that the beam specimens with bundled bars failed at a lower load and showed a tendency at high loading of higher end slip than those with spaced bars. The average steel stress was slightly higher in beams reinforced with bundled bars and the ultimate failure was due to a loss of bond. This checks with the tendency of the spaced bars to show less center deflection and less end slip thus indicating the efficiency of spacing of bars over bundling. However, the author concluded finally that there was no important loss of bond when deformed bars were tied together.

Walker studied a region where bond stress was distorted. The vertical compression due to the reaction at the supports of the beams would prevent the splitting of concrete in that region, thus increasing the bond strength of the reinforcing bar (see Sec. 3.1).

<u>Ferguson, et al. (10) 1954</u>

In an attempt to eliminate the shortcomings of concentric pullout test as a measure of bond strength, the authors devised a new eccentric pullout specimen. The specimen's cross section consisted of a half hexagon with a projection on the longest side where the reinforcing bars were embedded. The specimens were tested with the pull eccentric on the bar and concentric on the bearing block of the testing machine. In all, eleven double-bar eccentric specimens with No. 4 and No. 6 deformed bars were tested. Variables were bar clear spacing, and concrete cover. The clear bar spacing ranged from 0.5D to 2.33D, where D is the bar diameter. To simulate larger bar spacings, single bar specimens were used with bar spacings of up to 9D. To ensure bond failure, a light web reinforcement of No. 2 stirrups was used. The authors concluded that the observed free end slip prior to the ultimate load was 0.0001 inches or less in over 60 percent of the tests and 0.0005 inches or less in over 80 percent of the tests. Specimens with small bar spacings usually failed without any prior slip while 90 percent of the specimens with stirrups showed some slip prior to failure, and in most cases the failure was not sudden. Maximum load was reached after slips greater than 0.005 inches in 20 percent of the specimens. The authors suggested that, although bond strength is lower for close bar spacings, the real minimum spacing should be based on aggregate size alone. To check on the eccentric pullout tests, the investigators tested a number of beams and found out that bond stresses for the beams were generally less than 10 percent higher than the eccentric pullout tests.

The researchers finally observed that when splitting is not prevented by external forces, special reinforcement, or a large mass of concrete, such splitting appears to lower over-all bond resistance. A careful restudy of this element of design seems justified since in most bond tests attention has been largely centered on specimens reinforced or restrained against splitting. The authors concluded that the eccentric pullout test provides a reasonable measure of bond strength as it occurs in beams where splitting is possible. Simple span beam tests for bond strength become seriously involved with diagonal tension failures unless the beam is artificially strengthened against this type of failure. If stirrups are used, splitting is prevented and high bond strength is obtained.

The investigators suggested that, where splitting is possible, bond values seemed to be dependent upon bar spacing, mix proportions, stirrups and other factors.

The objective of the eccentric pullout specimen used by the authors was to eliminate and overcome the objections raised regarding the standard ASTM pullout tests. Although the pull was eccentric on the bar in these tests, the concrete was in compression at the loaded-end, a stress situation not found in flexural members. In simulating larger bar spacings the investigators used single bar specimens and varied the concrete width per bar. This technique is not entirely realistic in that it does not include the effects of splitting forces set up in the concreteby the adjacent bars.

Chamberlin, et al. (11) 1956

The authors tested a number of beam specimens 6-x 6-x 36-inch and 9-x 9-x 54-inch. The beams were loaded with a two-point symmetrical loading. The cross section of the central part of the beams was kept constant to simulate the beam conditions in the region of zero shear and constant moment. Slippages of bars were measured through the opening on the bottom of the beams. End portions of the beams, between the load point and reaction, had a narrow projecting rim of different width in different specimens on the tension side. This was an attempt to achieve variable bar spacings. The reinforcement consisted of No. 4 and NO. 6 bars, both plain and deformed. Two types of deformed bars were used; old-style deformed bar with transverse deformation, and modern deformed bars with deformation conforming to ASTM A-305-50T.

The authors concluded that ultimate loads increased with wider spacing until tensile failures developed. Bar slippages were greater for the narrowest spacing than for the others. All plain-bar beams failed by excessive slippage of the steel. Deformed-bar beams which did not fail in tension failed either by rupture of the concrete along a horizontal plane at the centerline of the steel or in combination with diagonal tension.

The irregularities in the cross section of the beams and the partial restraint of the bars at the reaction points make the conclusions reached by the authors debatable. Also in simulating bar spacing by varying the concrete width per bar, the effect of combinations of bars was neglected.

Hanson (8) 1958

Hanson reported the results of tests on 10 beams (3 with spaced bars and 7 with bundled bars) and 10 tied columns (2 with spaced bars and 8 with bundled bars). All the reinforcement was intermediate grade and had deformations which conformed to ASTM A-305-53T. Placement consisted of groups of four No. 6, four No. 8 or three No. 9 bars for the beams with bundled reinforcement, and groups of three No. 6 and three No. 8 bars for the tied columns. All the beam specimens were supported on a roller at one end and on a rocker at the other end with a 6-inch overhang at each end. The load was supplied to the beams through a reinforced column stub at the center of the beams. The 12-x 12-x 72-inch columns were tested under concentric loading with both ends fixed against rotation. The strains on the faces of the columns were measured. The results of Hanson's beams demonstrated that all beams, with both spaced and bundled bars, failed by yielding of the bars, excessive deflection, and final crushing of the compression zone at the column stub. There was no indication of bond failure in any of the beams, and there seemed to be no difference in the behavior of the beams with bundled reinforcement as compared with the spaced bars. In the case of the columns, which had steel percentages ranging up to 6.6 percent, comparisons of the ultimate strengths indicated that bundling is a safe detailing procedure whenever adequate ties are provided. The author assumed that these tests represented the extreme cases of bending only and compression only. Thus, he concluded that, since bundling was found to be safe in these cases, it was doubtful if it would be detrimental for the members subject to combined bending and axial load.

Yee (12) 1965

Yee compared the bond strength of three bundled and three spaced bars embedded in concentric pullout concrete blocks. The blocks were 10-x 10-inch in cross section with variable embedment lengths. The reinforcement consisted of four different sizes: No. 3, No. 4, No. 6, and No. 7 bars with deformations conforming to ASTIM A-305-56T. The author summarized his findings as follows:

1. The slip increased for both the free end and loaded end for a given bond stress as the bar size increased.

2. The bond stress for a given slip value at the loaded end was usually less for bundled bars than for spaced bars.

3. The percentage increase in load for spaced bars to bundled bars based on the maximum loads, was 31.4 percent. This is 11.4

percent over the expected 20 percent increase in load cpacity due to the difference in the total perimeter of bundled bars as compared to the spaced bars.

Yee's conclusions are limited by the recognized shortcomings of the use of the concentric pullout test as a test for bond strength.

Summary

The effect of bar spacing on the bond strength of reinforcing bars has received little attention in the past. Some investigators tested beam specimens with spaced and bundled bars. They concluded that specimens with tied bars failed at a lower load and had a tendency at high loading to show more end slip than those with spaced bars. This was disputed by some other researchers who reached the conclusion that there seemed to be no difference in the behavior and load carrying capacity of the beams with bundled reinforcement as compared with the spaced bars.

Another group of investigators simulated bar spacing by varying the concrete width per bar. They concluded that the ultimate load carrying capacity of the specimens was higher for large bar spacings.

2.3.2 Current Practice

It is not uncommon that reinforcing bars are left in the open at steel mills and construction sites for months and become rusted before they are used. Because of uncertainties of corrosion effects on bond strength of reinforcing bars, there has been considerable difference of opinion regarding the maximum amount of corrosion that could be tolerated safely. On numerous occasions contractors have been required to wire-brush bars at a considerable cost. For years the building codes in the United States have left the decision of acceptance or rejection of a certain corroded bar to the personal judgement of the field inspectors. The American Concrete Institute Building Code 318-63 in Section 504 states that "metal reinforcement, at the time the concrete is placed, shall be free from rust scale or other coatings that will destroy or reduce the bond". This rather vague statement is representative of the current state of the knowledge.

The ACI Code in Section 804-f states that " groups of parallel reinforcing bars bundled in contact to act as unit must be deformed bars with not over four in any one bundle and shall be used only when stirrups or ties enclose the bundle". The Code also sets a minimum clear spacing for bars of 1-1/3 times the maximum size of the coarse aggregate, or one inch or, in columns, 1-1/2 times the bar diameter, whichever is the largest.

The most recent statement concerning the effects of bar spacing on bond strength was related in the report of ACI Committee 408 as follows:

> " Splitting can devlop over 60 to 70 percent of the bar length without loss of average bond strength. Possibly because of a changing splitting pattern, width of beam influences the bond resistance. A single No. 11 bar in an 18 inch width, will develop higher bond stress than in a 16 inch width, and less than in a 24 inch width. The resistance of closely spaced bars creating a plane of weakness is substantially lower. This close spacing effect is one of the more serious factors still needing further investigation. Only in the case of lapped splices has it been reflected in the ACI Code.
>
> Obviously, clear cover over a reinforcing bar will be significant in connection with splitting resistance. Thin cover can be easily split; very thick cover can greatly delay splitting if bars are not too closely

> spaced laterally. While it is not economical to increase bond strength by varying cover, the designer should recognize that bond strength in a slab with 0.75 inch of cover is lower than in a beam with 1.5 to 2 inches of cover, unless bar spacing is so close as to lead to the horizontal splitting failure. A few tests have arleady indicated that a closely spaced layer of bars will split across the plain of the bars at stresses substantially below the ACI Code recognized values. In extreme cases of close bar spacing the shear stresses may become large and bond may not govern."

The report concludes that the effect of close spacing of bars (beam width per bar) is one of the weak spots in existing knowledge of bond theory, that the development of an adequate bond theory depends on the establishment of the real bond stress distribution, the real splitting forces developed, and what factors influence these two.

2.4 Summary

Although much progress has been made with regard to bond stress over the years, there is still a lack of knowledge of true mechanism of bond between concrete and a reinforcing bar.

Some differences of opinion exist concerning the effects of corrosion and bar spacing on the bond strength of reinforcing bars. Of course, most of these differences can be attributed to variations between tests due to changing shapes and surface conditions of the bars and revisions of test procedures.

With reference to the effects of corrosion, Withey, Shank, and Gilkey used concentric pullout tests on plain bars. In addition to the fact that plain bars are rarely used today, it is generally believed that the concentric pullout test is not entirely realistic as a measure of bond strength in a beam. In such a test the concrete is in compression while bond stresses in a beam are usually critical in the tension zone. Also, shearing stresses on the splitting plane in a beam complicate the failure, whereas the concentric pullout specimen carries no external shear. Therefore, the concentric pullout test develops local bond stresses always in excess of the average calculated from tests. Johnston and Cox's work in 1940 was the first of its kind conducted on deformed bars. They tested 330 concentric pullout specimens but their results were somewhat scattered and did not exhibit the same pattern in the variation of bond strength with increasing degree of rust among different bar sizes.

Ferguson et al., and Kemp et al. have used eccentric pullout tests on deformed bars in establishing bond strength. The latter investigation, the only one in which corrosion was studied, was the most extensive one to date, and the eccentric pullout specimen used by the investigators was an improvement over the concentric pullout test in better representing the normal conditions in a flexural member.

The allowable bar stresses recognized by the 1963 ACI Code were derived largely from tests using widely spaced bars or a single bar cast in concrete. Ferguson,et al. argue that there is some conservatism in the ACI Building Code on minimum bar spacing under some conditions. Under other circumstances, they point out, this minimum bar spacing results in inadequate protection against failure in bond. The authors further state that the Code permits bond stresses as high as 0.10 f'_c (not over 350 psi), but that these bond stresses are safe only whenever the splitting of the concrete is prevented by the use of spirals or whenever a large mass of concrete is used. In practice, bars are used in circumstances which differ from these conditions. Ferguson's investigation showed that failure occurs much below the 350 psi limit of the 1963 ACI Code for bar spacings of less than 2D (D is the bar diameter) and thus this part of the code needs further study. Concerning the one inch or one bar diameter minimum spacing rule set by the Code, Ferguson et al. have commented that this dimension does not assure adequate bond strength in all cases. They recommended that the minimum bar spacing be based on aggregate size alone.

CHAPTER III

PREPARATION OF THE TEST SPECIMENS

3.1 Introduction

The deficiencies of the standard concentric test are discussed by Ferguson, et al. (10,28), ACI Committee 408 (13), Kemp, et al. (20) and other investigators. From these discussions it is obvious that the concentric pullout test is not entirely realistic as a measure of bond strength in a flexural member. The horizontal shearing stresses which exist in considerable magnitute at the level of the bars in a beam are not represented. And the fact that the concrete is in near uniform compression is obviously contrary to the normal case, e.g., concrete subjected to a tensile stress. At the same time the current standard ASTM simple-span bond test beams are not ideal test specimens for investigating bond, either. Simple-span beam tests for bond strength become seriously involoved with diagonal tension failures unless the beam is artificially strengthened against this type of failure. If stirrups are used for this purpose, splitting is prevented and higher bond strengths are obtained, strengths that are not available unless the stirrups are present. In addition, the simple-span beam reactions tend to postpone the critical concrete splitting thus increasing the bond resistance. The following areas are examples of where splitting would appear to be a factor that could be important:

1. Any point on negative moment steel, without stirrups.

2. Positive moment steel near the points of inflection in continuous span beams.

3. Anywhere an anchorage length for a definite stress is required (except in mass concrete), such as : (a) stirrup lengths above or below mid-depth of beam, or (b) tension lap splices.

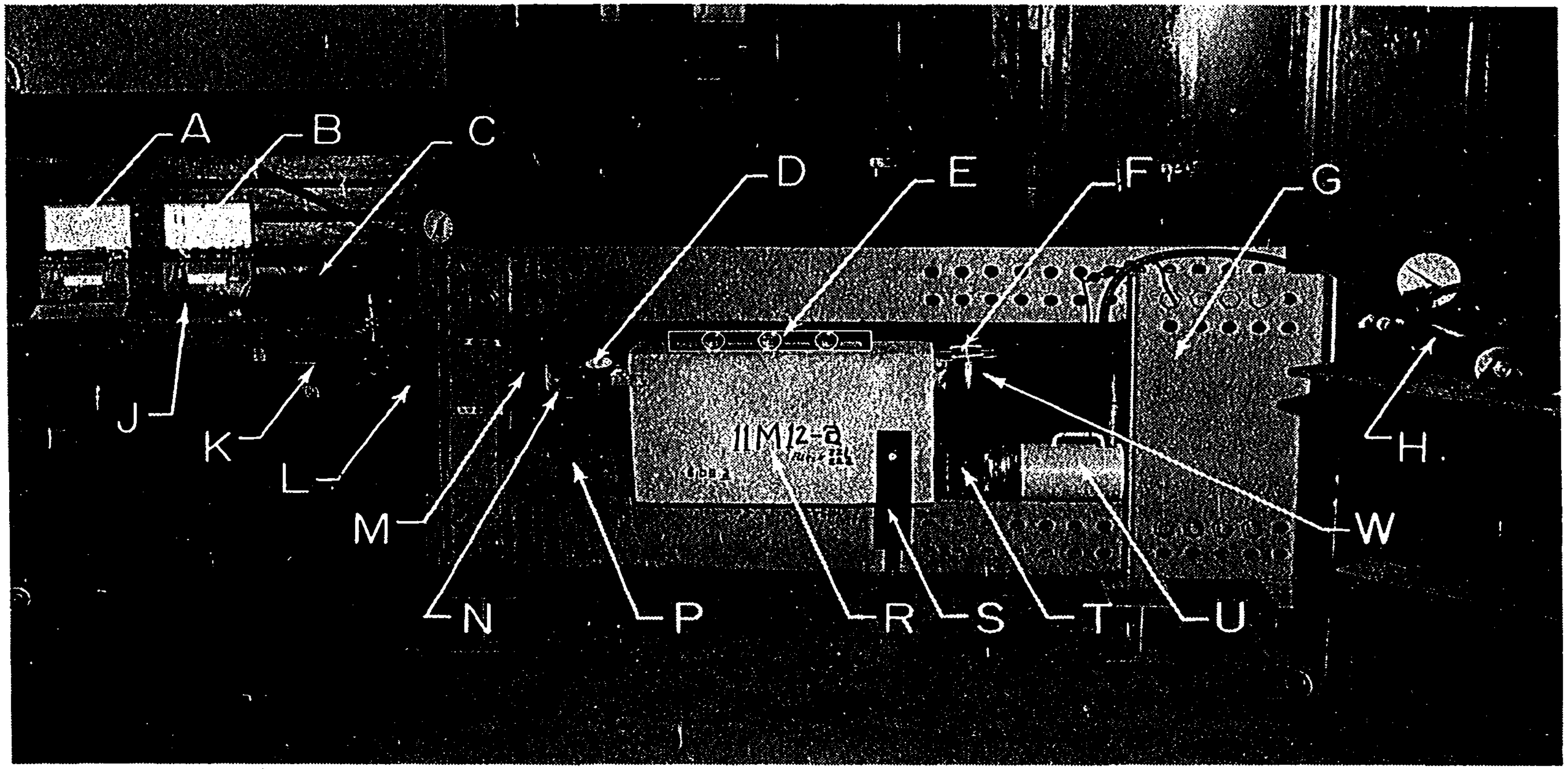
Therefore, in order to simulate ordinary beam conditions and to eliminate any external compressive forces on the splitting sections, an eccentric bond-pullout specimen, similar to that used by Kemp, et al. (20), was designed. The specimen and the testing frame are shown in Fig. 3.1. As opposed to the concentric pullout test, the concrete and steel in this eccentric pullout specimen undergo the same type of strains.

3.2 Analysis and Design of Eccentric Pullout Specimen

3.2.1 Dimensions of the Specimen

A number of pilot tests were performed. The objectives of the tests were to establish the development length of the bars and to check on the performance of the testing rig and the test specimens. The tests indicated that an embedment length of 19D (D is the bar diameter) exhibited a failure in bond resistance before the steel yielded. Yet, the average 38-ksi steel stress obtained in these tests was above the range of normal service conditions.

The ACI Committee 408 recently affirmed that the effect of close bar spacing (or beam width per bar) constitutes one of the weak spots



A - Strain Indicator	F - Dial Gage	L - Load Cell	S - Hold Down Yoke
B - Switch & Balance	G - Testing Frame	M - Howlett Grip	T - Comp. Block
C - Hydraulic Pump	H - Hydraulic Pump	N - Gripping Device	U - 60 Ton Ram
D - Dial Gage	J - Dummy Load Cell	P - Comp. Block	W - Reinf. Bar
E - Level	K - 60 Ton Ram	R - Eccen. Pullout Spec.	

Fig. 3.1 Testing Rig for Eccentric Pullout Specimens

in our knowledge of bond between concrete and reinforcing bars. A few tests have indicated that a closely spaced layer of bars will split across the plane of the bars at stresses substantially below the 1963 ACI Building Code recognized values. In extreme cases of close bar spacing the shear stresses, rather than bond stresses, may govern the failure of the reinforced concrete member. In general, splitting is not the same as bond failure. It would be desirable to establish a unique criterion against splitting. But at the present a crucial lack of experimental data on this subject exists. Therefore, it is evident that the close spacing effect is one of the more serious factors still needing further investigation.

Two clear bar spacings of 3D and adjacent tied were used, as shown in Fig. 3.2. With 1-inch side cover the width 'w' of the bond specimens was set at:

$$w = 5D + 2"$$

The concrete clear cover over the reinforcing bars was $1\frac{1}{2}$ inch. The height of the specimens were arbitrarily chosen approximately equal to 2w. The specimen dimensions for all five sizes of deformed bars used in this investigation were kept constant regardless of the number of bars in a specimen.

To resist the vertical shear forces, a light reinforcing cage was cast in each specimen. This was done to prevent diagonal tension failure. It should be pointed out that the stirrups were cut off two inches above the bars. A schematic of the test specimens and the reinforcing cage used is shown in Fig. 3.2.

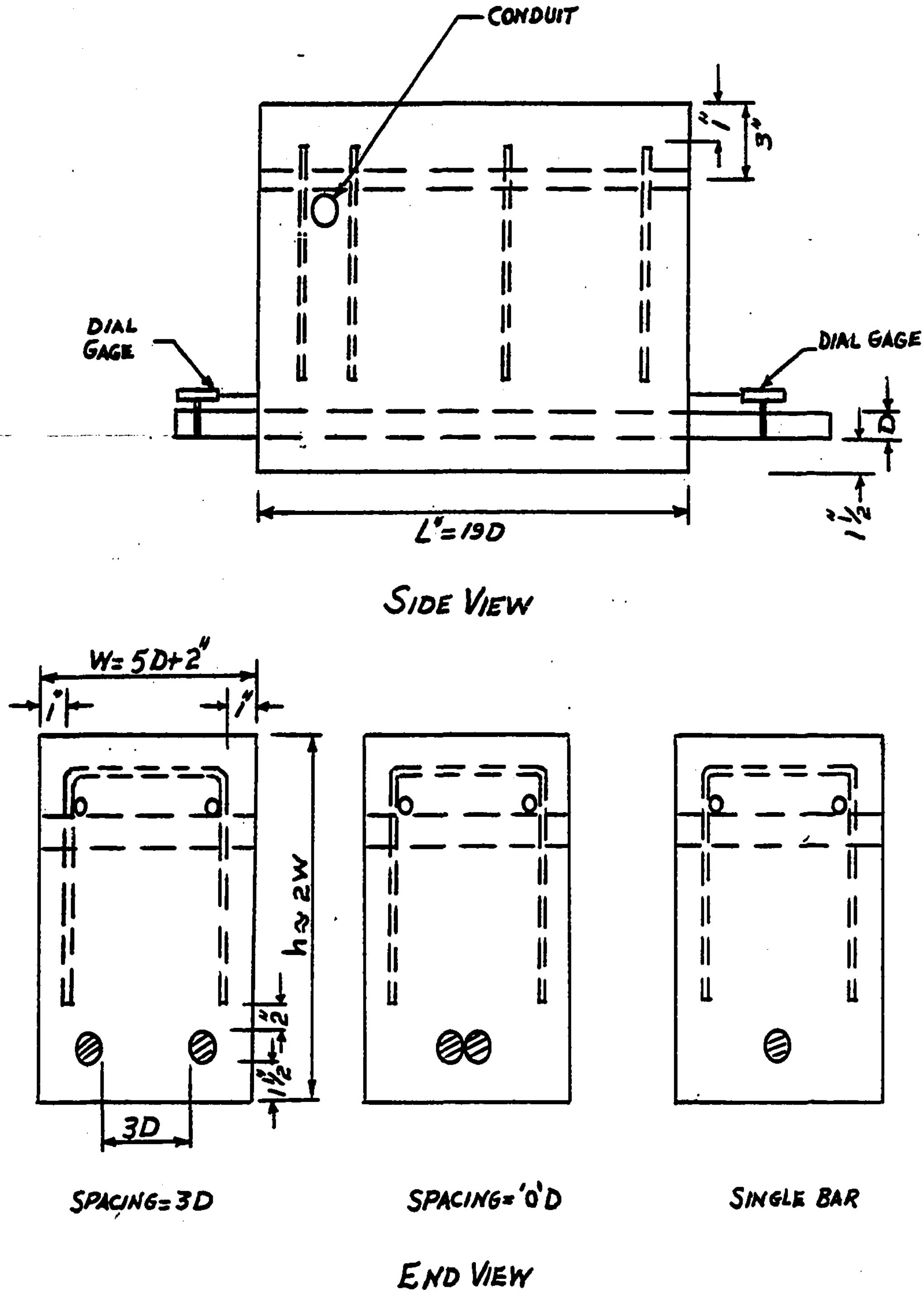


Fig. 3.2 Schematic of the test specimens.

3.2.2 Deformed Reinforcing Bars

The reinforcing bars used in this investigation were Nos. 4,6, 8,10, and 11 modern deformed bars conforming to ASTM Designation A-305-56T.

The important physical properties of the bars, the spacing and the height of deformations, as well as the minimum and maximum requirements stated in ASTM A-305-56T (34) are tabulated in Table 3.1. The bar areas were calculated by dividing the weight (in pounds) per linear inch of the bars by the theoretical weight of steel (0.2833 pci). The perimeters of the bars were determined from the areas. It should be mentioned that the weights and the bar lengths used in the determination of the bar areas were the averages of the weights and lengths of 86 control coupons; while the lug spacing and the deformation heights shown in Table 3.1 are the average values of measurements made on 47 control coupons (see Table A.1 in Appendix A).

3.2.3 Concrete

A nominal concrete compressive strength of 3,000 psi was used in this study. The concrete was made from Type III portland cement and locally available crushed limestone for coarse aggregate and Colorado river sand for fine aggregate. The concrete had the following characteristics: a water-cement ratio (W/C) of 0.60, an aggregate-cement ratio (A/C) of 3.28, and a sand-cement ratio (S/C) of 2.49. A three-cubic-foot capacity, electric-powered, concrete mixer was used to prepare the concrete. The mixing time, after the water was added, was about 5 minutes. The slump of each batch of concrete was measured. The concrete compressive strength was determined from compression tests

TABLE 3.1

PHYSICAL PROPERTIES, HEIGHT AND SPACING OF DEFORMATIONS,

AND AREA OF THE AS-ROLLED BARS ASTM A-305-56T

No. of Samples		Bar Size #4	#6	#8	#10	#11
10	Yield Strength (ksi)	50.0	48.2	52.0	47.4	48.1
	Coeff. of Variation %	1.8	2.2	1.7	2.5	1.6
	ASTM Requirement (ksi)	40.0				
10	Ult. Strength (ksi)	75.7	80.2	83.2	75.8	80.2
	Coeff. of Variation %	1.1	2.5	0.6	0.8	1.1
	ASTM Requirement (ksi)	70 to 90				
86	Area ($in.^2$)	0.20	0.44	0.79	1.23	1.54
	Nominal Area ($in.^2$)	0.20	0.44	0.79	1.27	1.56
	Perimeter (in.)	1.571	2.356	3.142	3.950	4.430
47	Lug Spacing (in.)	0.223	0.309	0.413	0.500	0.536
	Coeff. of Variation %	0.7	0.2	0.8	0.0	0.7
	ASTM Maximum Spacing	0.350	0.525	0.700	0.889	0.987
47	Lug Height (in.)	0.026	0.048	0.067	0.087	0.072
	Coeff. of Variation %	8.2	6.0	6.6	5.1	6.8
	ASTM Min. Height	0.020	0.038	0.050	0.064	0.071
86	6" coupon Surface Area ($in.^2$)	9.82	14.92	20.29	25.83	29.27

of 4-x 8-inch cylinders made in accordance with ASTM C-39-64. About 40 percent of the concrete control cylinders were tested in split cylinder test in accordance with ASTM C-496-64T.

3.2.4 Curing of Test Specimens

After concrete was poured into the specially constructed plywood forms, vibrated with an internal vibrator, and the top surface screeded to the level of the forms, the specimen and the companion control cylinders were covered with Griffolyn plastic sheeting material. Twenty four hours after placement of concrete, the specimens and the cylinders were stripped and were stored at room temperature until the time bhey were tested.

3.3 Acquisition and Storage of Reinforcing Bars

In order to minimize the variations in the surface condition and the mechanical properties of the reinforcing bars, and in order to be abole to accurately measure the amount of corrosion of the bars, an adequate number of modern deformed bar samples from the same heat were acquired immediately after their rolling at the onset of the test program. As soon as the bar samples were cool enough to be handled, they were placed in air-tight plastic bags and stored until the time of casting in concrete.(in the case of specimens with as-rolled bars) or exposure to outdoor, indoor, and sea water corrosive environments.

3.4 Environmental Exposure of Reinforcing Bars

It has been argued that loose and "flaky" corrosion is detrimental to bond strength while "firm" corrosion improves it. In order to study this conclusion, it was decided to produce different degrees of corrosion of reinforcing bars, from early, loose corrosion to old, firm corrosion. Therefore, the reinforcing bars were exposed to three corrosive environments for several periods of exposure starting in September 1966 and ending in September 1967. The periods of exposure for bars outdoor (air-rusted) were 3,6, and 12 months while those of indoor bars were 6 and 12 months. Of the bars aerated with sea-water only the 3-month rust was available.

The three types of corrosion environments were as follows:

a) Outdoor

Bars were stored individually on a rack especially designed to expose the entire bar surface. The rack was located in an exposed area. Three periods of exposure of 3,6, and 12 months were studied. Each bar test sample was accompanied by a short 6-inch control coupon. The length, weight and the physical properties of the coupons were measured before and after each period of exposure and are reported in Table A.1.

b) Indoor

Five different sizes of bars were placed vertically in a rack in a moist cabinet and aerated with fresh water for periods of 6 and 12 months. Each bar had a 6-inch long control coupon. The measured physical properties of these bars, measured before and after each period of exposure, are reported in Table A.1.

c) Sea-water

In order to produce a very corrosive environment similar to that which might be found at a sea side, "sea-water" was simulated by using a reconstituted sea-water made by mixing Pacific Ocean salt with local water. The chemical composition of the ocean salt used is shown in Table 3.2.

The bars and the control coupons were placed in a cylindrical barrel and the sea-water was circulated and sprayed over the bars. The pH of the water was checked periodically, but no change was observed. The water was changed every week. The test apparatus is shown in Fig. 3.3.

3.5 Summary

An eccentric pullout specimen was designed to better duplicate the stress conditions which exist in a beam. The pilot tests indicated that an embedment length of 19D developed a steel stress above the range of normal service conditions (38 ksi).

With $1\frac{1}{2}$-inch concrete clear cover over the reinforcing bars, the specimens' dimensions were kept constant while the bar spacing was varied. Three types of specimens were tested : single-bar, double-spaced-bar, and double-adjacent-bar. The concrete compressive strength was 3,000 psi.

The deformed reinforcing bars (ASTM A-305-56T) were stored in air-tight bags until the time of casting or exposure to the different corrosion environments.

In order to produce a wide range of corrosion (early, loose to old and firm corrosion), the reinforcing bars were exposed to the following three corrosion environments: (a) outdoor (air-rusted), (b) indoor (moist-cabinet), and (c) sea-water, for periods of 3, 6, 12; 6, 12 ; and 3 months, respectively.

TABLE 3.2

THE CHEMICAL COMPOSITION OF THE OCEAN

SALT USED

Compound's Name	Percentage
Sodium Chloride	98.980 %
Magnesium Carbonate	0.480
Calcium Silicate Oxide	0.480
Sodium Sulphate	0.005
Calcium Sulphate	0.001
Other Compounds	0.054
	100 %

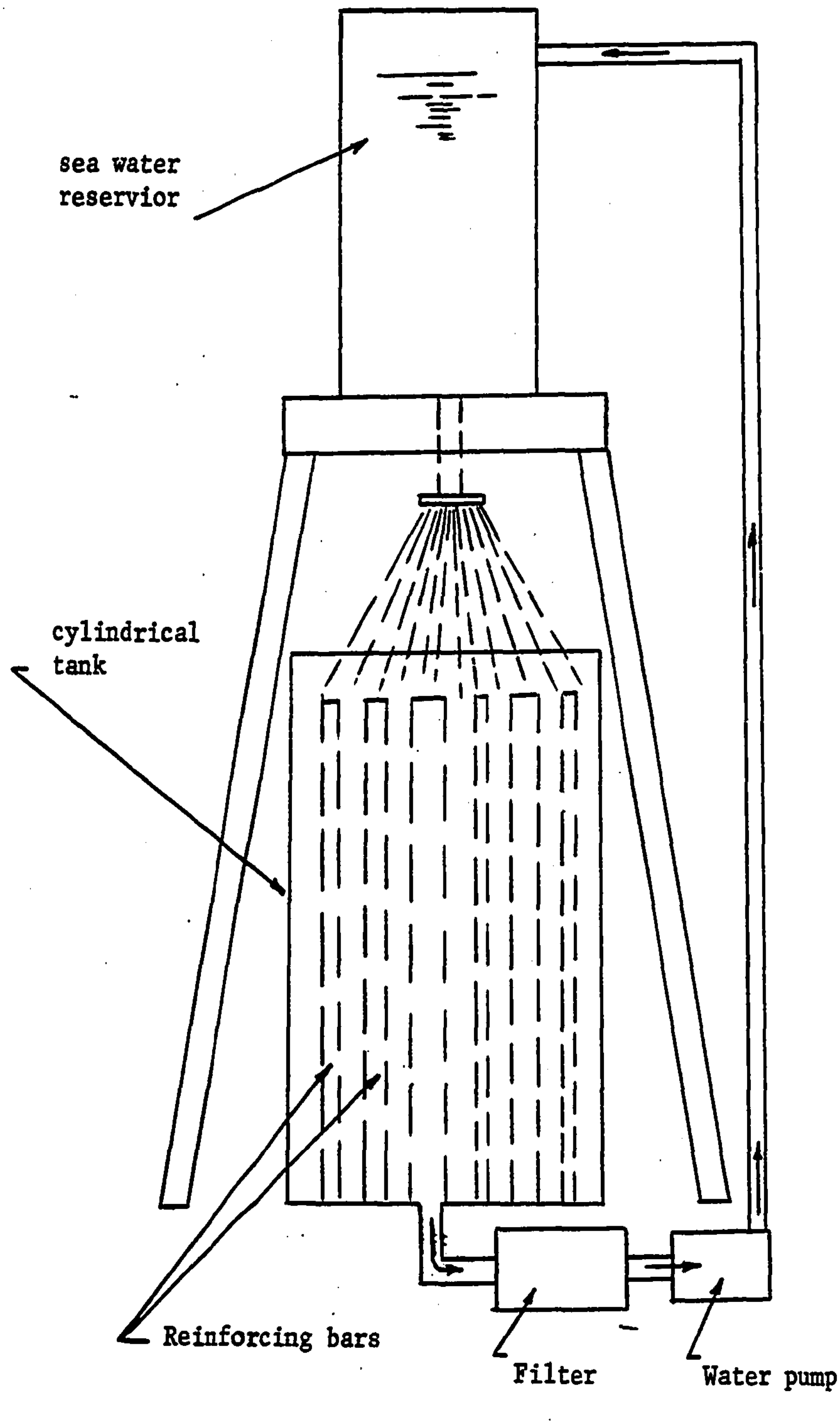


Fig. 3.3 Schematic of sea water flow apparatus.

CHAPTER IV

EXPERIMENTAL PROGRAM

4.1 Introduction

The type of loading used in this investigation shown in Fig. 3.1 was devised to obtain a bond condition undisturbed by load points, reactions or the stirrups. To produce such bond condition an eccentric bond pullout specimen was designed, Fig. 3.2.

The objectives of this study was the evaluation of the effect and significance of different degrees of corrosion and bar spacing on bond properties of deformed reinforcing bars as compared with the as-rolled bars. To this end, a total of 115 eccentric pullout specimens were tested in three series. The first series of 15 specimens was cast with as-rolled reinforcing bars. Series II contained 50 specimens with 3-months-outdoor, 3-months-sea-water, and 6-months-indoor corroded bars. The third series of 50 specimens was cast with 6,12-months-outdoor and 12-months-indoor rusted bars. Table 4.1 shows the number and the type of specimens in each of the series.

4.2 Materials

4.2.1 Properties of Coarse and Fine Aggregates

The aggregates used in this research were obtained locally. The coarse aggregate was crushed limestone of 3/4-inch maximum size. The

TABLE 4.1

NUMBER AND THE TYPE OF SPECIMENS CAST

FOR EACH PERIOD OF EXPOSURE

	Number of Specimens						
Corrosive environment	As-rolled	Indoor		Outdoor			sea water
Months exposed	0	6	12	3	6	12	3
Single bar specimen	1	2	2	3	3	3	2
Double-bar specimen 3D bar spacing	1	1	1	0	0	0	1
Double-bar specimen adjacent tied	1	1	1	0	0	0	0
Sub Total	3	4	4	3	3	3	3

Total = 23 specimens per bar size

TABLE 4.2

PROPERTIES OF THE COARSE AND FINE AGGREGATES

	Coarse Aggregate	Fine Aggregate
Type:	Crushed Limestone	Colorado River Sand
Unit Weight:	100 lb./in.^3	-
Apparent Specific Gravity:	2.77	2.41
Absorption Rate (S.S.D.):	1.50 %	0.65 %
Fineness Modulus:	-	2.58

properties of fine and coarse aggregates were found following the procedure outlined in ASTM Desginations : C29-60, C90-47, C127 & 128-59, C136-63, and are reported in Table 4.2.

4.2.2 Design of Concrete Mix

A nominal concrete compressive strength of 3,000 psi was chosen for this investigation. Portland cement (Type III) was used which produced the desired compressive strength in a few days. The concrete mix, based on surface dry conditon of the aggregates, had the following characteristics: a water-cement ratio (W/C) of 0.60, an aggregate-cement ratio (A/C) of 3.28, and the sand-cement ratio (S/C) was 2.49. Slump of the concrete was maintained between 3½ and 4½ inches. Each batch of concrete produced from two to five pullout specimens depending on the size of the specimens. From each batch at least five or more control cylinders (4-x 8 inches) were made. Each group of cylinders tested was divided as follows: about 40 percent, tensile strength (split cylinder) and the remainder, compressive strength. The control cylinders were tested at the same age as the pullout specimens. The average values of the cylinder compressive strengths were used in adjustment of the bond strength of the specimens. The adjustment was necessary in order to eliminate the effect of variations in concrete strength between the batches.

4.2.3 Reinforcing Bars

The Nos. 4,6,8,10, and 11 deformed reinforcing bars conformed to ASTM A-305-56T. In order to minimize the variations in mechanical properties of the bars, as well as variations in surface conditions, the 4-foot-long bar samples were cut from 80-foot-long bars of the same heat of steel. Adequate care was taken to minimize disturbance of the mill scale on the surface of the bars.

In measuring the amount of corrosion of the bars, it was necessary to weigh the 6-inch control coupons cut from the bar samples before exposure to corrosive environments and after the corrosion was removed. The difference in the weights would be the amount of corrosion on the bars.

4.2.3.1 Methods of Removing and Measuring Mill Scale. Mill scale is a form of ferric oxide produced on the surface of reinforcing bars in the rolling operation.

The 6-inch coupons were clamped in a vice and were wire-brushed with a medium soft wire brush until all the f· ky mill scale was removed. Flaky scale was defined as that scale which could be removed by this brushing. The coupons were weighed again and the weights were recorded as W_b in Table A.2. It was observed that brushing alone did not remove all the scale. Therefore, the coupons were soaked in a plastic tub containing 10 percent by volume surfuric acid for 30 minutes and were neutralized for three minutes in another plastic tub containing 10 percent by volume sodium hydroxide. The coupons were dried, wire-brushed and the weights were recorded as W_1. After removing the mill scale, the diameter of the bars, lug height and the length of the coupons as well as the spacing between the deformations were measured (Table A.2) with beam calipers accurate to 0.001 inch. The height of the lugs (deformations) was calculated by averaging the difference between the average diameter between the lugs and the average diameter over the lugs. All the values reported in Table A.2 are the averages of three measurements.

It was observed that one pickling cycle did not remove all the mill scale. Thus the process was repeated. After brushing, the coupons appeared shiny and clean with virtually all of the scale removed. The coupons were subjected to a third and a fourth cycle of the chemical pickling in order to remove the small remainder of the scale and to assess the effects of the acid on the metal. Since the weight differences (W_2 - W_4) in Table A.2 are small, it may be concluded that the acid did not remove substantial amounts of non-corroded metal.

Mill scale was removed from 20 control coupons of each bar size. The coupons were weighed to the nearest 0.01 gram by means of a "Mettler" analytical balance.

<u>4.2.3.2 Tensile Test.</u> The objective of this investigation was to determine how far corrosion could progress before it appreciably reduced the effective cross-sectional area of the bar, thereby reducing its ultimate strength. To fullfil this objective 10 clean, as-rolled bars of each bar size were tested in tension using a 200,000 lbs. capacity universal testing machine.

The bar elongation of a 8-inch long gage length was measured with the aid of two dial gages (0.0001-inch least count) mounted on opposite sides of the bars. After the bars yielded, the dial gages were removed and the elongation of the bar in the gage-length was measured with a ruler at 1/16-inch intervals. In the same manner a number of corroded bars of each size were tested. A least-square polynomial fit of a straight line through the data points in the elastic range was used to determine the modulus of elasticity. The tensile test-rig is shown in Fig. A.1D of Appendix A.

4.3 Fabrication and Curing of the Bond Test Specimens

Specially-constructed adjustable plywood forms were used. The forms were coated with a mixture of epoxy resin for durability.

The stirrups with the auxiliary compression reinforcement were assembled into a cage. The specimen design required the encasement of a conduit normal to the main reinforcement near the unloaded end of the specimen. This was needed for clamping the end of the specimen to the testing frame to provide the force necessary to produce the counter-balance moment essential for equilibrium of the specimen, Fig. 3.2. To insure 1-inch concrete cover over the cage and to maintain the ends of the stirrups about 2 inches above the main reinforcement, the cage was suspended from the top of the end-forms by means of 1/16-inch-diameter wires.

The concrete was placed in the forms in two layers and each layer was vibrated by an internal vibrator for a short period of time. The top surface of the specimen was screeded to the level of the forms and was finished with a steel trowel. The specimens and the compression control cylinders (4 x 8 inches) were covered with plastic sheet immediately after finishing. The cylinders and the bond-specimens were stripped after 24 hours and they were stored at room temperature.

4.4 Concrete Compressive and Tensile Split Cylinder Tests

The control cylinders were capped for the compression tests with a sulphur capping compound. The loading rate was approximately 1,000 psi per minute. The uncapped cylinders were tested in accordance with ASTM C496-64T for split cylinder test. Each value of the compressive or tensile strength given in Table 4.3 represents the average of three or more cylinder tests. The cylinders were 4-x 8 inches. Should the reader wish to convert these strengths to the strength of concrete specimens of different shapes and sizes, he may use the following equation reported by Neville (31):

$$\frac{P}{P_6} \times \frac{d}{d_6} = 0.8878 \, (A/A_6)^{0.4525}$$

where:

P = concrete compressive strength of the desired shape and size, psi,

P_6 = concrete compressive strength of 6-inch cube, psi,

d = lateral dimension of the desired cross section, inch,

d_6 = lateral dimension of 6-inch cube,

A = cross-sectional area of the desired shape and size, in.^{2},

A_6 = area of 6-inch cube.

The above equation is based on the test results on cylinders by the 12 investigators listed by Neville (31). Fig. 4.1 is a plot of the above equation.

4.5 Loading System

Instead of the usual pullout tests, an eccentric pullout specimen was designed in order to obtain bond strengths undisturbed by reactions or other point loads. The loading system used in this

TABLE 4.3

RESULTS OF CONCRETE COMPRESSIVE AND TENSILE SPLIT CYLINDER TESTS

Specimen No.	No.,Cyls.	Days cured	f''_c psi	V %	No.,Split cyl.	f_{sp} ,psi	V %
11A0-a	4	8	3324	3.2	1	316	-
11A0-b	3	26	3235	1.7	1	318	-
11A0-c	3	24	2787	0.6	0	-	-
10A0-a & b	2	21	3423	3.7	0	-	-
10A0-c	4	20	3503	5.3	0	-	-
8A0-a & b	4	8	3280	6.8	2	316	4.4
8A0-c	2	18	3646	3.0	-	-	-
6A0-a & b	4	10	3280	6.8	2	316	4.4
4A0-a,b,c	2	9	3423	1.4	2	316	4.4
11N3-c 11N6-e	4	19	3260	5.1	1	294	-
11N3-d & e	4	19	3427	9.8	1	269	-
10N3-c 10N6-e	5	23	3920	8.2	2	398	7.2
10N3-d & e	4	20	3662	6.3	1	354	-
8N3-c & d	5	16	3503	7.2	2	251	3.2
8N3-e, 8N6-e	8	21	3344	19.0	3	320	9.2
6N3-c 6M6-a,b,c,d	4	16	3025	219	0	-	-
6N3-e,d N6-c,d,e	6	10	2070	7.0	0	-	-
4N3-c,d 4M6-c	4	10	3561	5.1	1	430	-
11N6-c & d	2	18	3623	0.2	1	362	-
11M6-a & b	5	23	3514	3.4	2	340	4.1
11M6-c & d	5	21	3065	4.1	2	314	10.1
10N6-c & d	3	21	3396	8.2	1	259	-
10M6-a & b	3	24	2635	6.7	1	334	-
10M6-c & d	5	24	3619	15.7	2	412	10.1
8 M6-a & b	4	24	2154	5.9	2	274	8.2
8 M6-c,d 4M6-a	3	16	2580	4.1	1	352	-
8 N6-c,d 4M6-a,b	5	15	2388	1.3	0	-	-
11M12-a & b	5	17	3196	4.9	2	322	7.4
11M12-c & d	5	14	3411	5.8	2	277	20
11N12-c & d	5	14	3767	10.9	2	364	1.6
11N12-e	4	14	3348	6.7	2	308	3.2
10M12-a & b	6	15	3076	6.7	3	370	1.7
10M12-c & d	4	14	2826	5.4	4	450	9.1
10N12-c & d	4	16	2986	9.1	2	342	4.6
10N12-e 8N12-e	5	16	3446	4.6	2	322	18.5
8 N12-c & d	5	16	2882	7.7	3	407	8.5
8 M12-c & d	5	15	3196	5.3	3	349	4.6
6 M12-a,b N12-d,e	8	13	3049	1.3	4	337	12.7
6 & 4 M12-c, d	5	11	4700	1.1	3	374	5.7
11S3-a	5	14	3519	4.6	2	425	10.0
11S3-c & d	4	12	3439	12.8	1	434	-
10S3-c & d	6	12	3177	5.0	4	301	1.1
10 & 8 S3-a	5	13	2842	11.6	1	410	-
8S3-c & d	8	10	2838	2.8	4	320	12.1
			Average	5.8%		342	7.4%

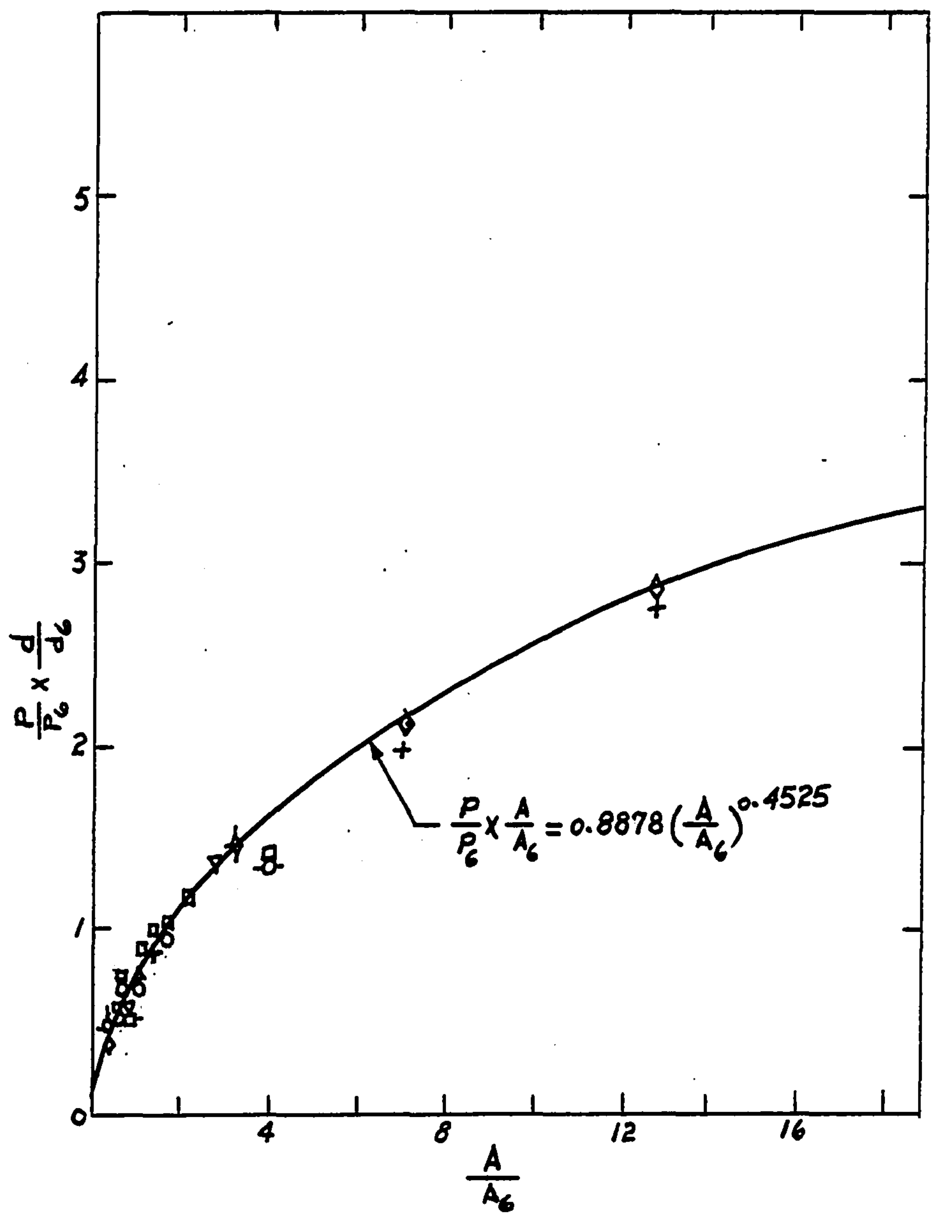


Fig. 4.1 Relation between $\left(\frac{P}{P_6}\right) \times \left(\frac{d}{d_6}\right)$ and $\frac{A}{A_6}$

investigation is shown in Fig. 3.1.

The testing rig consisted of a frame made of four channels and two pieces of steel plates, supported by two cross I-beams. The specimen was held in equilibrium by a vertical and a horizontal couple. The vertical couple was developed by the friction forces on the ends of the specimen. These friction forces were developed by the action of the 60-ton hydraulic ram U and the reaction from the frame. The pilot tests indicated a small up-lifting of the unloaded end of the specimen, which led to a premature failure of the specimen in the compression zone. This was due to the inadequate magnitude of shear forces on the ends of the specimen. To supplement these forces, the specimens were provided with a transverse encased conduit through which a bolt was passed. The bolt was fastened to the hold-down yoke S, thereby, clamping the specimen to the testing frame, Fig. 3.1.

The horizontal couple was produced by the action of the 60-ton hydraulic ram K on the reinforcing bar and the horizontal reaction of the compression block P. The axial load of the ram was transmitted to the reinforcing bar through a high strength rod, the Howlett Grip M and the gripping device N. Another Howlett Grip was used to transform the outward push of the ram K into an axial pull on the rod.

For the specimens with two bars, tied or spaced, three special gripping devices shown in Fig. 4.2 were desgined. The gripping box G_1 was used for specimens with tied reinforcement. The bars were inserted into the box and a steel bearing plate with two holes drilled in it was placed and welded onto the ends of the reinforcing bars. The gripping beam G_2 was made with two rectangular slots cut and spaced to accomodate

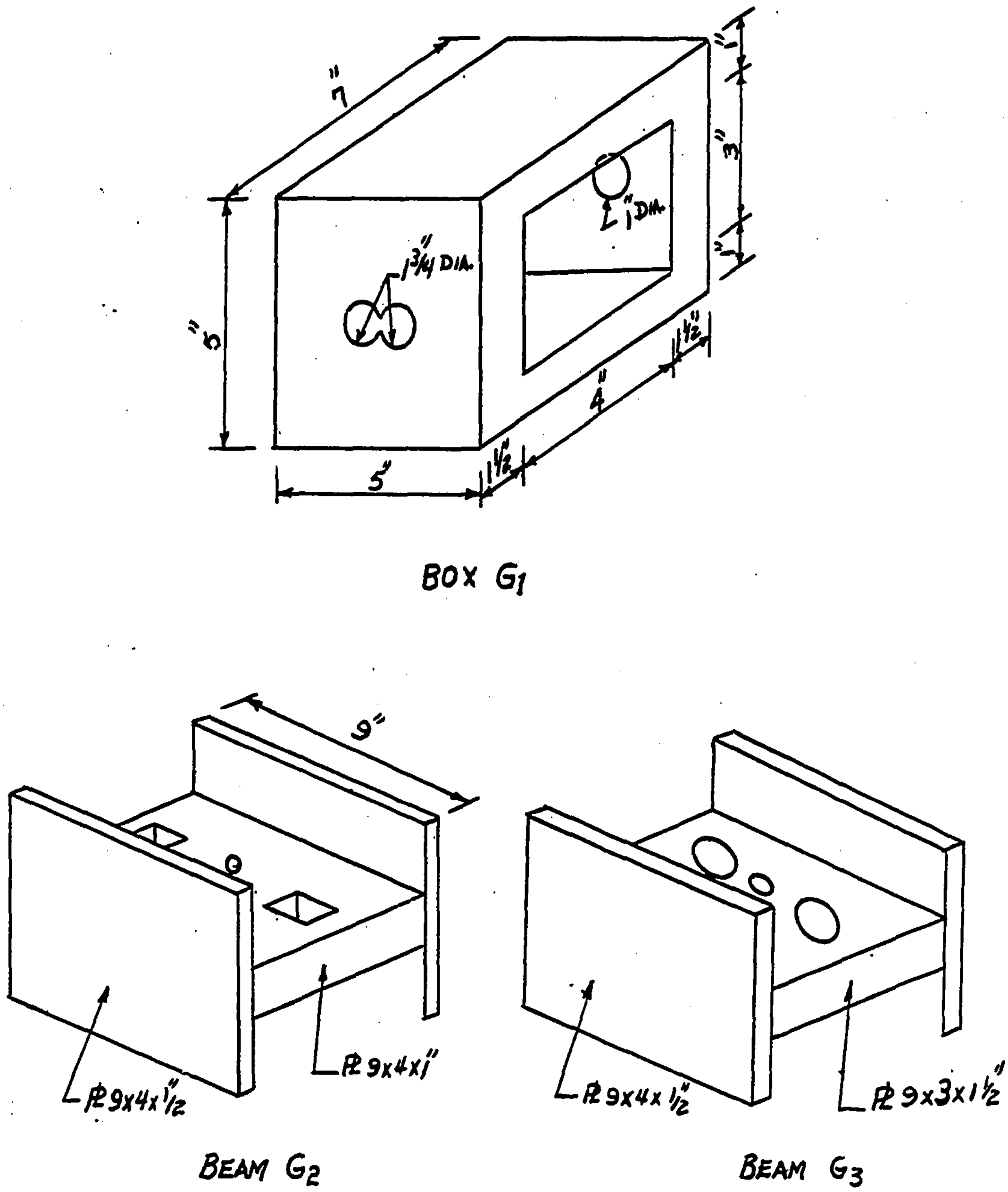


Fig. 4.2 Gripping devices.

Nos. 8,10, and 11 bars with 3D bar spacing. Beam G_3 was designed for No. 4 and No. 6 specimens with 3D bar spacing. Two Howlett grips were used to grip the ends of the reinforcing bars.

Dial gages were mounted on the reinforcing bars at the loaded end and free end of the specimens in order to measure the relative movement of the bars and concrete.

4.6 Instrumentation

The hydraulic ram K produced the tensile load applied to the reinforcing bar. In order to measure this axial force, a 100,000-lb.-capacity load cell was placed between the ram K and the channels of the testing frame (Fig. 3.1). The load cell was attached to the frame by means of rwo metal straps. This arrangement allowed the adjustment of the position of the load cell for different height specimens. A similar load cell J, connected in series with load cell L, was used as dummy for a check on the possible drift of the load cell L. The load cells were connected to a 10-channel switch and balance unit, which in turn was connected to a portable digital strain indicator A. The load cell readings, in micro-inches per inch, were converted to loads by means of a predetermined calibration curve.

To measure the relative movement of the reinforcing bar and the concrete at the loaded end, two dial gages (0.0001-inch least-count) were mounted on a ring. The ring was fastened to the bar by three set screws, with the tips of the dials riding on the end of the specimen. To eliminate any false reading of the dials due to irregularity of the surface of the concrete, the tips of the dial gages were placed on small aluminum shims which were glued to the end of the specimen. As a check on the loaded end dial gages, a target was marked on the loaded end of the bar, near where the dial mounting ring was in contact with the reinforcement, and a piece of steel ruler graduated to 1/64-inch was glued to the side of the block along its longitudinal axis. The target and the ruler were sighted through a Dietzgen T-3 theodolite set 10 feet from the testing frame. The relative movement of the target was read from the ruler.

The slip at the free end of the specimen was measured in a similar manner. During the first half of the experiments two 0.001-inch least-count dials were used at the free end. In the second half, however, additional 0.0001-inch least-count dials were used at this end. All the dial readings were estimated to the nearest 0.00001-inch.

A 18-inch-long steel level was used to level the specimen in both the longitudinal and transverse directions. Two hydraulic pumps (C and H in Fig. 3.1) with calibrated pressure dial gages were used in connection with rams K and U thus providing a static check on the load cell L.

4.7 Test Procedure

The length, width and the height of the specimen were measured and the specimen was seated on the testing frame in an inverted position. This position was used for the ease of instalation and removal of the specimens. It also facilitated the inspection and marking of the cracks on the tensile face of the specimens. The gripping box G_1 or beams G_2 and G_3, whichever necessary, and the Howlett grips were attached to the reinforcing bars. The concrete block was loaded in compression by ram U (Fig. 3.1) until the specimen was centered and leveled in the testing frame. Then the clamp at the free end was fastened to the frame and tightened. The cross bar assembly was connected to the gripping device and was passed through the load cell L and ram K. The strain dial gages were mounted on the rings previously slipped over the reinforcing bars. The distance between the end of the block and the point of contact of the dial gages was measured. The specimen was loaded in compression through two 4in.x 12 in.x 3/4 in. plates on the ends of the specimen. The bottom of the plates were 1½ inches above the bottom of the concrete blocks. Depending on the size of the specimen, the compression stress was maintained between 2,000 and 3,000 psi throughout the test.

The load cells were connected to the strain indicator. The load cells were zeroed and initial dial gage and theodolite readings were taken. Depending on the size of the specimen, a loading rate of 1 to 2.5 kips per load stage was used. The dial gages were estimated to the nearest 0.00001-inch and the load cell readings in micro-inches per inch and the pump pressure dials readings were recorded. Between each loading interval the specimen was checked for those cracks which were macroscopically visible. The cracks were marked and designated by hydraulic pressure reading of ram K. The crack pattern was also sketched on the data sheet.

From the progress of the cracks and the readings of the dial gages it was possible to estimate the percentage of the ultimate failure load attained. At this point, the dial gages were dismounted and the specimen was loaded to failure. The failure load was recorded, and the final crack pattern of the concrete specimen was marked. A discussion of the behavior of the specimens is presented in Sec. 5.3.

4.8 Corrosion Measurement

The 6-inch-long control coupons cut from the reinforcing bar samples were weighed to the nearest 0.01 gram by means of a "Mettler" analytical balance prior to exposure to corrosive environment. After the corrosion was removed the coupons were weighed again. The difference between the original weight and the final weight was recorded as the weight of the corrosion (Table 4.4).

The color and the nature of the corrosion covering the surface of the reinforcing bars was recorded. A black and white picture of the corroded bars, and a number of color slides, were taken for optical observation of the different degrees of corrosion.

In order to remove the corrosion, a pickling process similar to that described in Sec. 4.2.3.1 was employed. The coupons were immersed in a solution of 10 percent by volume sulfuric acid contained in a plastic tub. After one hour the coupons were removed from the acid and were neutralized in a solution of 10 percent by volume sodium hydroxide. The coupons were dried, brushed and weighed. Brushing made the surface of the coupons shiny and clean. A note was made of the surface condition of the bars, observing especially any sign of pitting. Photographs of the chemically cleaned and wire brushed bars were taken.

TABLE 4.4

THE AMOUNT OF CORROSION ON CORRODED BARS

Type & Months Exposed	Initial wt. W_i(gr.)	Final wt. W_f (gr.)	Corrosion $W_i - W_f$ (gr.)	Average Corrosion (gr.)	Coef. of Var. V %
		No. 4 bars			
S3	161.30	151.44	9.86		
"	159.90	151.72	8.18	9.32	7.8
"	158.11	148.09	10.02		
"	159.36	150.13	9.23		
		No. 6 bars			
S3	337.78	325.78	12.0		
"	337.65	324.04	13.61	13.81	8.3
"	332.97	318.22	14.77		
"	335.40	320.54	14.86		
		No. 8 bars			
S3	586.37	564.93	21.44		
"	594.51	577.80	16.71	20.81	13.0
"	605.56	581.30	24.26		
"	601.43	580.61	20.82		
		No. 10 bars			
S3	902.41	870.00	32.41		
"	942.55	913.08	29.47	30.58	16.5
"	935.10	905.91	29.14		
"	937.90	906.60	31.30		
		No. 11 bars			
S3	1165.80	1138.56	27.24		
"	1171.89	1131.63	40.26	34.17	14.6
"	1187.99	1156.10	31.89		
"	1184.50	1147.19	37.31		
		No. 4 bars			
N3	157.11	156.40	0.71		
"	150.03	149.00	1.03	0.89	15.2
"	156.35	155.41	0.94		
M6	154.70	153.46	1.24		
"	152.24	150.93	1.31		
"	160.62	159.10	1.52	1.37	9.3
"	156.63	155.30	1.33		
"	154.18	152.61	1.57		
"	156.60	155.34	1.26		
N6	155.43	154.61	0.82		
"	158.43	157.60	0.83	0.95	18.6
"	154.20	153.00	1.20		

M = indoor, N = outdoor, S = sea water

TABLE 4.4 (con't)

Type and Months Exposed	Initial wt. W_i(gr.)	Final wt. W_f (gr.)	Corrosion $W_i - W_f$ (gr.)	Average Corrosion (gr.)	Coef. of Var. V %
M12	151.22	149.46	1.76		
"	147.09	145.30	1.79		
"	154.21	152.69	1.52	1.66	6.4
"	155.21	153.52	1.69		
"	155.60	154.08	1.52		
"	155.50	153.82	1.68		
N12	150.70	149.55	1.15		
"	152.07	151.00	1.07	1.11	3.0
"	156.30	155.20	1.10		
		No. 6 bars			
N3	335.29	334.16	1.13		
"	341.95	339.95	2.00	1.41	29.2
"	341.43	340.32	1.11		
M6	329.44	327.73	1.71		
"	339.70	338.11	1.59		
"	341.55	339.30	2.25	1.88	19.0
"	338.12	336.50	1.62		
"	335.46	332.95	2.51		
"	333.00	331.36	1.64		
N6	332.21	330.55	1.66		
"	339.50	338.14	1.36	1.58	10.0
"	338.53	336.80	1.73		
M12	341.20	338.55	2.65		
"	332.55	329.82	2.73		
"	338.45	335.95	2.50	2.53	10.6
"	334.28	331.64	2.64		
"	332.61	329.91	2.70		
"	334.65	332.70	1.95		
N12	328.18	326.18	2.00		
"	342.83	340.90	1.93	1.97	1.6
"	343.00	341.01	1.99		
		No. 8 bars			
N3	590.93	588.09	2.84		
"	600.98	598.20	2.78	2.75	3.3
"	600.62	598.00	2.62		

M = indoor, N = outdoor, S = sea water

TABLE 4.4 (con't)

Type and Months exposed	Initial wt. W_i (gr.)	Final wt. W_f (gr.)	Corrosion $W_i - W_f$ (gr.)	Average Corrosion (gr.)	Coef. of Var. V %
M6	593.78	589.93	3.85		
"	599.41	595.89	3.52		
"	598.29	594.35	3.94	4.01	7.8
"	590.45	586.05	4.40		
"	595.17	590.76	4.41		
"	588.29	584.35	3.94		
N6	598.70	595.91	2.79		
"	590.40	587.82	2.58	2.76	6.1
"	599.52	596.61	2.91		
M12	590.60	585.90	4.70		
"	580.95	576.08	4.87		
"	584.50	579.69	4.81		
"	594.65	590.05	4.60	4.84	11.7
"	616.50	610.53	5.97		
"	603.55	599.47	4.08		
N12	598.15	595.00	3.15		
"	593.73	590.80	2.93	3.07	3.3
"	600.88	597:74	3.14		
		No. 10 bars			
N3	917.80	914.32	3.48		
"	938.46	934.22	4.22	3.90	8.1
"	933.96	929.96	4.00		
M6	932.49	927.10	5.39		
"	926.70	920.89	5.81		
"	920.68	915.47	5.21	5.53	6.1
"	943.84	938.28	5.56		
"	945.76	940.41	5.35		
"	950.67	944.82	5.85		
N6	947.83	943.95	3.88		
"	948.48	945.20	3.28	3.48	8.0
"	910.39	907.10	3.29		
M12	943.10	936.51	6.59		
"	943.10	936.87	6.23		
"	982.35	975.39	6.96	6.67	4.2
"	942.40	935.58	6.82		
"	966.00	959.00	7.00		
"	935.73	929.28	6.45		

M = indoor , N = outdoor, S = sea water

TABLE 4.4 (con't)

Type and Months exposed	Initial wt. W_i (gr.)	Final wt. W_f (gr.)	Corrosion $W_i - W_f$ (gr.)	Average Corrosion (gr.)	Coeff. of Var. V %
		No. 10(con't)			
N12	943.83	939.11	4.72		
"	934.05	929.68	4.37	4.62	4.0
"	946.58	941.79	4.79		
		No. 11 bars			
N3	1168.29	1163.85	4.44		
"	1192.25	1188.51	3.74	3.94	9.0
"	1168.40	1164.75	3.65		
M6	1197.33	1191.68	5.65		
"	1192.62	1186.00	6.62		
"	1193.76	1187.36	6.40	6.16	5.0
"	1162.83	1156.61	6.22		
"	1187.42	1181.29	6.13		
"	1181.60	1175.65	5.95		
N6	1168.40	1164.36	4.04		
"	1174.01	1170.30	3.71	4.05	10.7
"	1167.72	1163.31	4.41		
M12	1156.08	1148.58	7.50		
"	1184.15	1176.80	7.35		
"	1185.69	1179.09	6.60	7.18	6.9
"	1212.75	1206.19	6.56		
"	1181.46	1173.49	7.97		
"	1178.48	1171.36	7.12		
N12	1189.14	1185.03	4.11		
"	1154.47	1149.85	4.62	4.66	10.0
"	1173.75	1168.50	5.25		

M = indoor, N = outdoor, S = sea water

4.9 Summary

A total of 115 eccentric bond-pullout specimens were tested. Type III Portland cement and 3/4-inch coarse aggregate were used with sand for fine aggregate. The concrete mix design had the following characteristics: water-cement ratio (W/C) of 0.6, aggregate-cement ratio (A/C) of 3.28, and sand-cement ratio (S/C) 2.49. The slump of the concrete was between 3½ inches and 4½ inches. The reinforcing bars were Nos. 4,6,8,10, and 11 deformed bars conforming to ASTM A-305-56T. The tensile strength of as-rolled and corroded bars were determined. The specimens were cured at room-temperature. The loading system consisted of a frame, two hydraulic rams and an eight-SR-4 strain gage, self-compensating, load cell. Loaded- and free-end slip measurements were taken using four dial gages. Loaded-end slip was checked by optical means. A chemical pickling process with surfuric acid was used to remove the corrosion of the bars as well as the mill scale.

CHAPTER V

EXPERIMENTAL RESULTS

5.1 Introduction

In this investigation, a total of 115 eccentric-bond pullout specimens cast with intermediate grade reinforcing bars of sizes Nos. 4,6,8,10, and 11 (ASTM A-305-56T) were subjected to monotonic load to collapse. The pullout specimens, desgined to closley approximate a non-restrained tension zone in flexural members, were cast with single bar, double bars spaced at three bar diameters, and double bars adjacent tied (Sec. 3.2.1). All of the reinforcing bars were collected at the rolling mill site and were protected from corrosion as soon as they were cool enough to be handled (Sec. 3.3).

The surface conditions of reinforcing bars studied were the result of exposure to the following environmental conditions: (a) as-rolled, (b) normal exposure (outdoor), and (c) special accelerated corrosive environments created in the laboratory. The laboratory corrosive environments consisted of an alternating wetting and drying of the bar samples which had been sprayed with (1) fresh tap water, and (2) simulated sea-water (tap water mixed with salt from the Pacific Ocean), (Sec. 3.4).

The specimens No. 1 through 15 tested with as-rolled reinforcing bars served as the basis for the comparison of the bond properties of reinforcing bars with different surface corrosion. For each bar size three specimens were tested: one with single bar, one with double bars spaced three bar diameters, and one with double tied adjacent bars.

The specimens No. 16 through 115 contained reinforcing bars with different degrees of surface corrosion. The number and the type of the specimens tested are reported in Table 4.1. Three of the single bar specimens with Nos. 8,10, and 11 bars were cast with the web reinforcement binding the main reinforcing bars. This was done to evaluate the effect of binding on bond strength and on the splitting behavior of the concrete specimens.

Also reported in this chapter are the results of tensile tests on 50 as-rolled and 80 corroded Nos. 4,6,8,10, and 11 reinforcing bars.

5.2 Bond Stress Versus Loaded-and Free-End Slips

The bond data taken during the testing of a bond eccentric pullout specimen consisted of strain gage readings from a calibrated load cell and the dial gage readings at the loaded and free ends of the test specimen. The load cell readings were converted into load values and were verified by pressure dial readings, thus maintaining a static check on the load cell. A uniform bond stress distribution was assumed to exist over the embedment length of the reinforcing bars in concrete. Bond stresses were calculated from the equation:

$$u = \frac{f_s A_s}{\sum o \; L''}$$

where u is bond stress, f_s is stress in the reinforcement, A_s is the area of the bar, and $\sum o$ is the perimeter of the bar and L" is the embedment length of the bar in concrete. The nominal design concrete compressive strength was 3,000 psi, however, the strength of concrete varied as a result of the normal test scatter (Table 4.3). Thus, the bond stresses reported (Figs. 5.1 - 5.20) were corrected by applying the multiplier:

$$\sqrt{\frac{3,000}{f''_c}}$$

to the calculated bond stresses (10,12,20,35). (f''_c is the compressive strength of concrete at the time of testing.).

The gross loaded-end slips were determined by averaging the readings of the two dial gages mounted at the loaded end. The net slip at the loaded end was found by subtracting from the gross slip the elastic elongation of the reinforcing bar occuring between the face of the concrete specimen near the reinforcing bar and the point of attachment of dial gages to the bar. Since the steel was not stressed at the point of measurement at the free end, the recorded readings for free-end slip represented the actual movement of the free end of the bar. The magnitude of the free-end slip was very small and the ultimate values for each specimen are tabulated in Table A.3, Appendix A. A check was maintained on the loaded-end dial gages by sighting a target on the reinforcing bars through a Dietzgen theodolite (Sec. 4.6).

When the dial gage readings or the cracking pattern of the eccentric pullout specimen seemed to indicate that a failure was imminent, the dial gages were dismounted. Therefore, slips for the failure bond stresses were estimated by extrapolation of the curves of Figs.5.1 through 5.20.

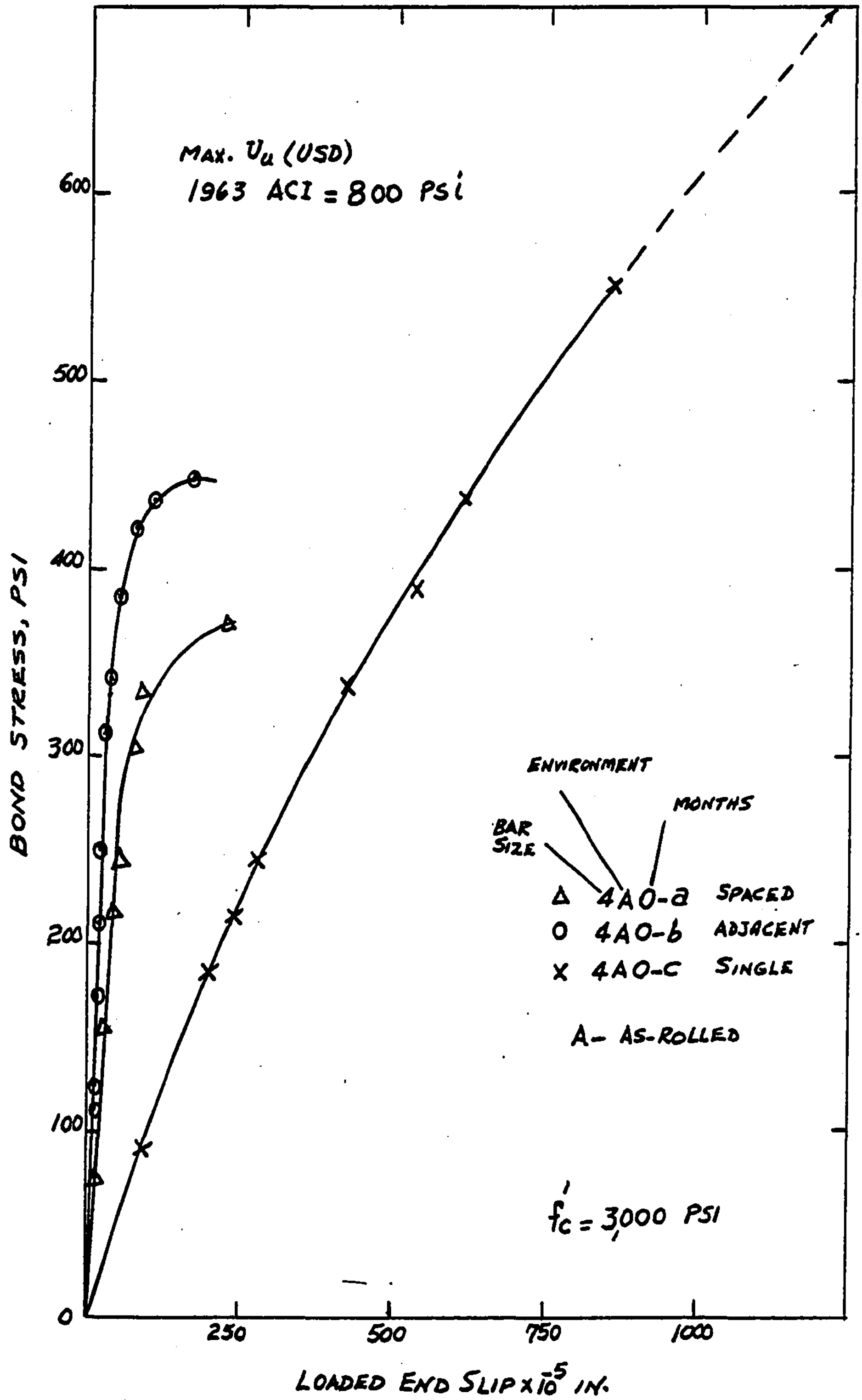


Fig. 5.1 Bond stress-loaded end slip curves for No. 4 as-rolled bars.

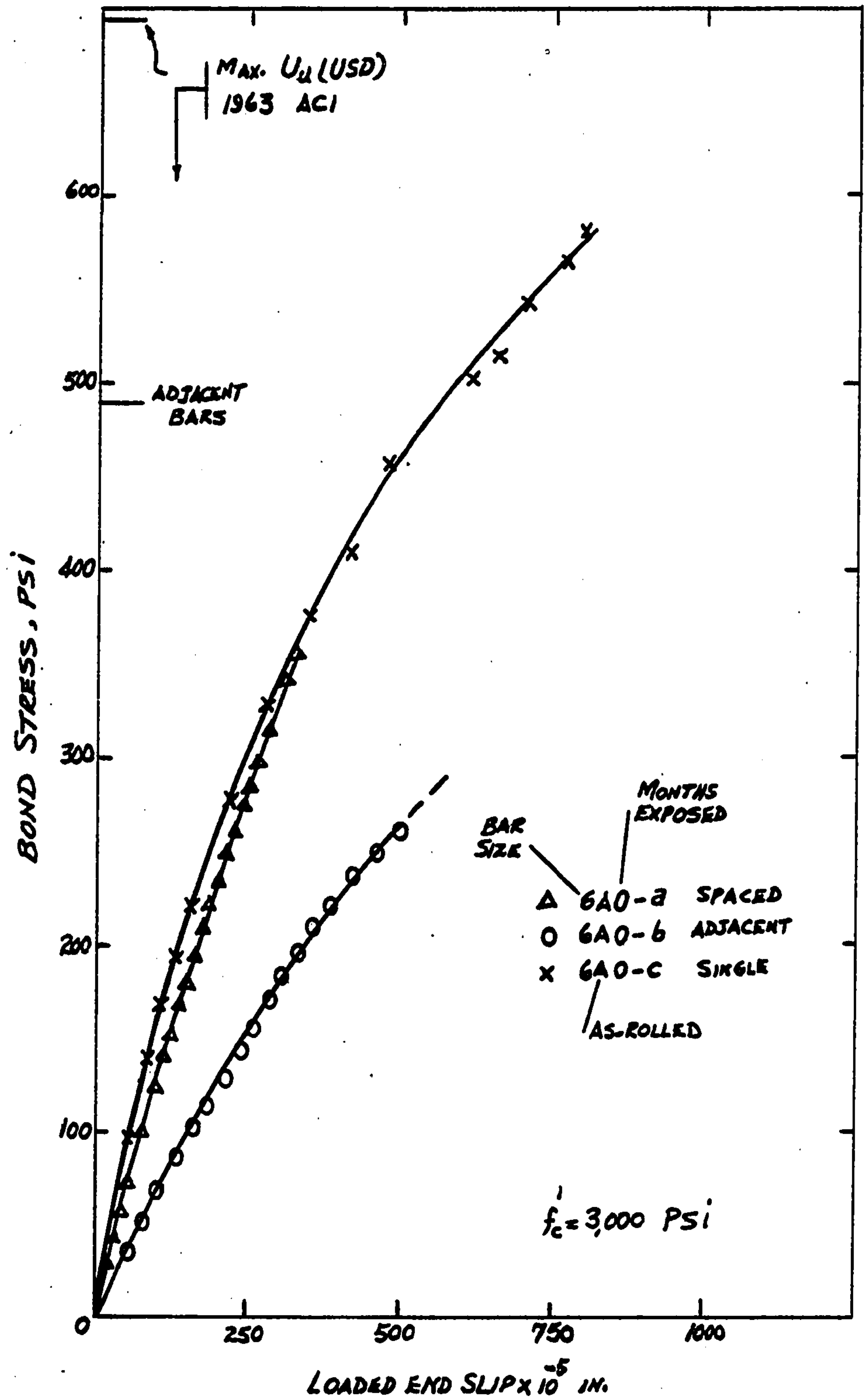


Fig. 5.2 Bond stress-loaded end slip curves for No. 6 as-rolled bars.

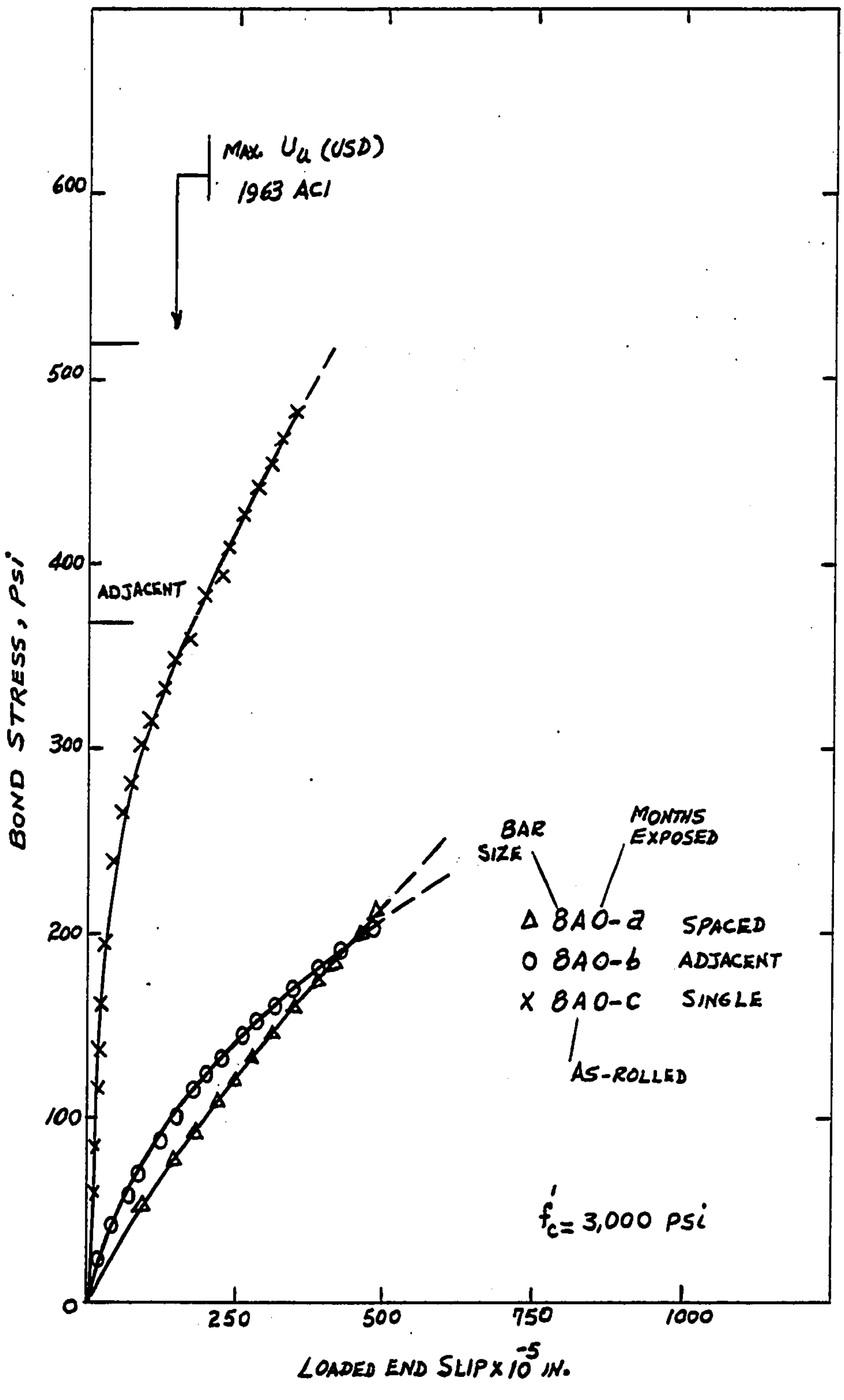


Fig. 5.3 Bond stress-loaded end slip curves for No. 8 as-rolled bars.

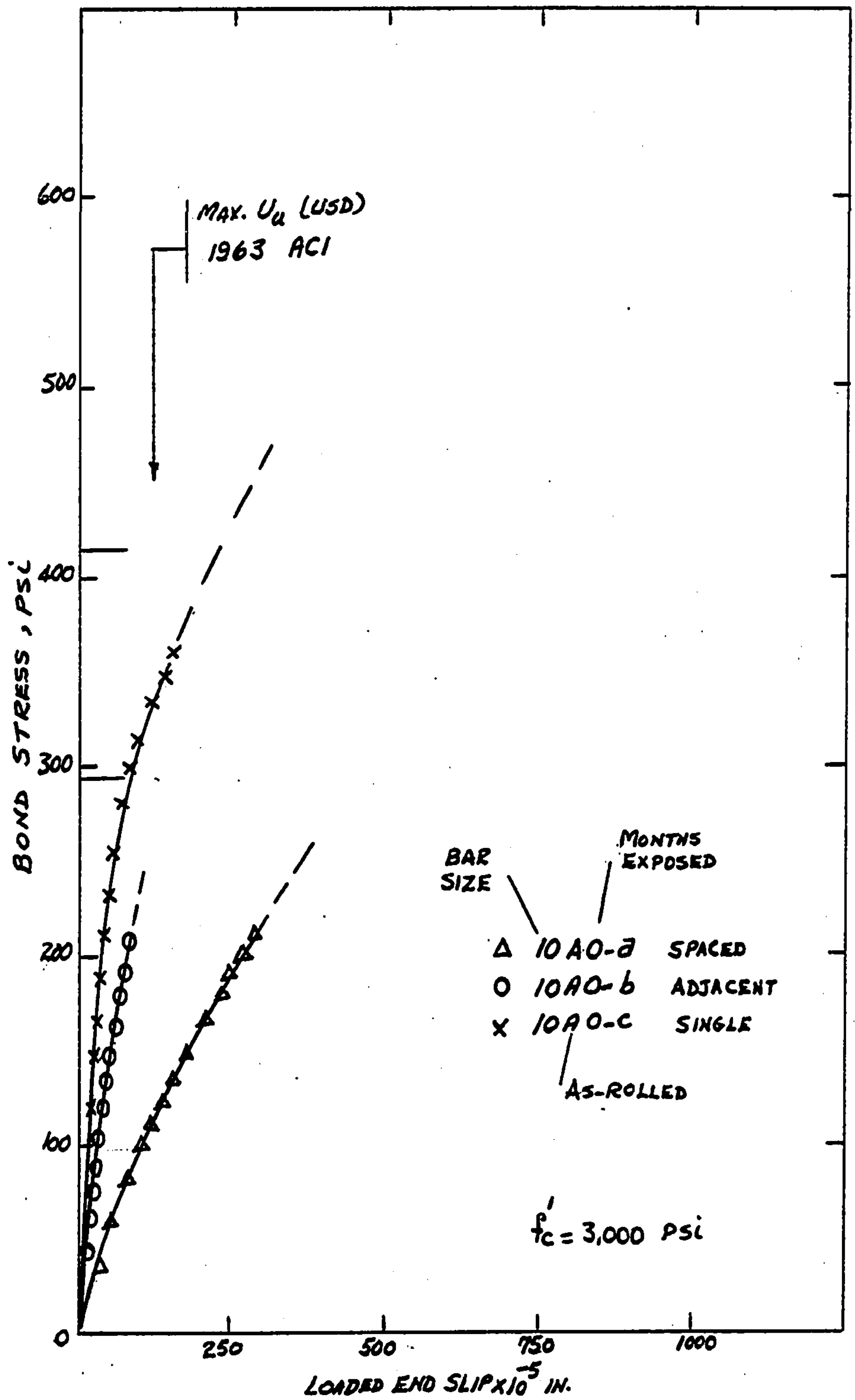


Fig. 5.4 Bond stress-loaded end slip curves for No. 10 as-rolled bars.

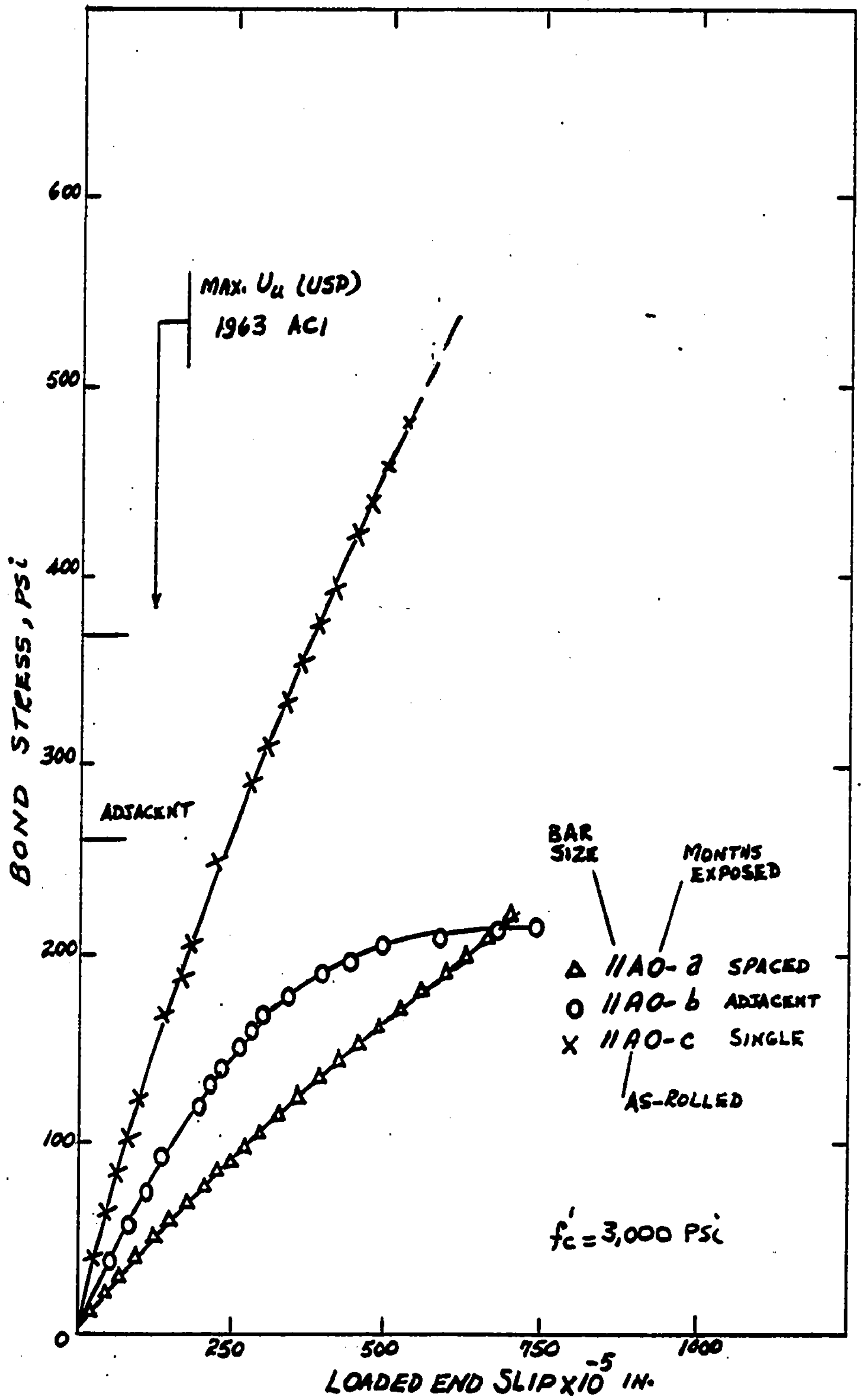


Fig. 5.5 Bond stress-loaded end slip curves for No. 11 as-rolled bars.

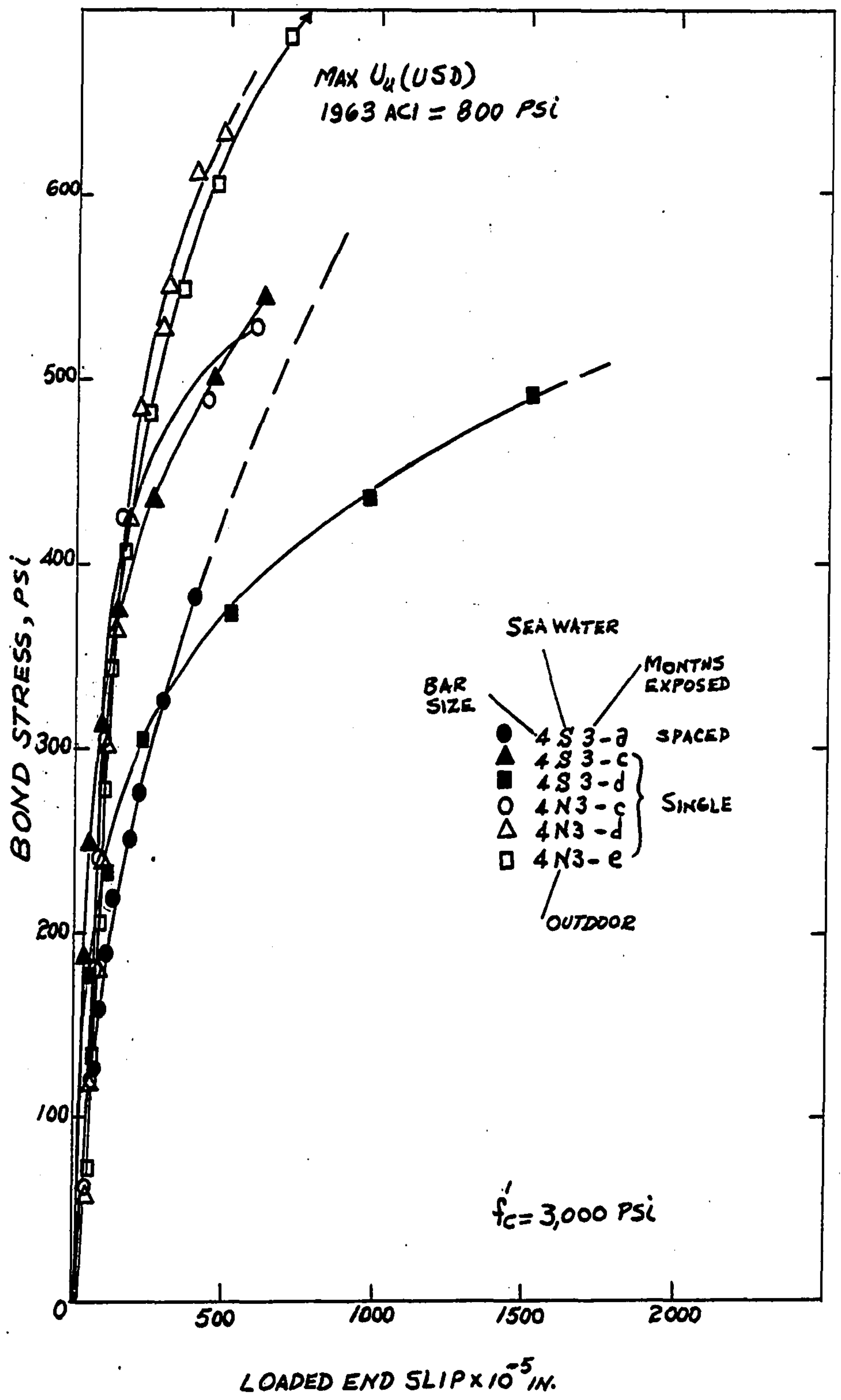


Fig. 5.6 Bond stress-loaded end slip curves for No. 4 bars (3 months corrosion, outdoor & sea water).

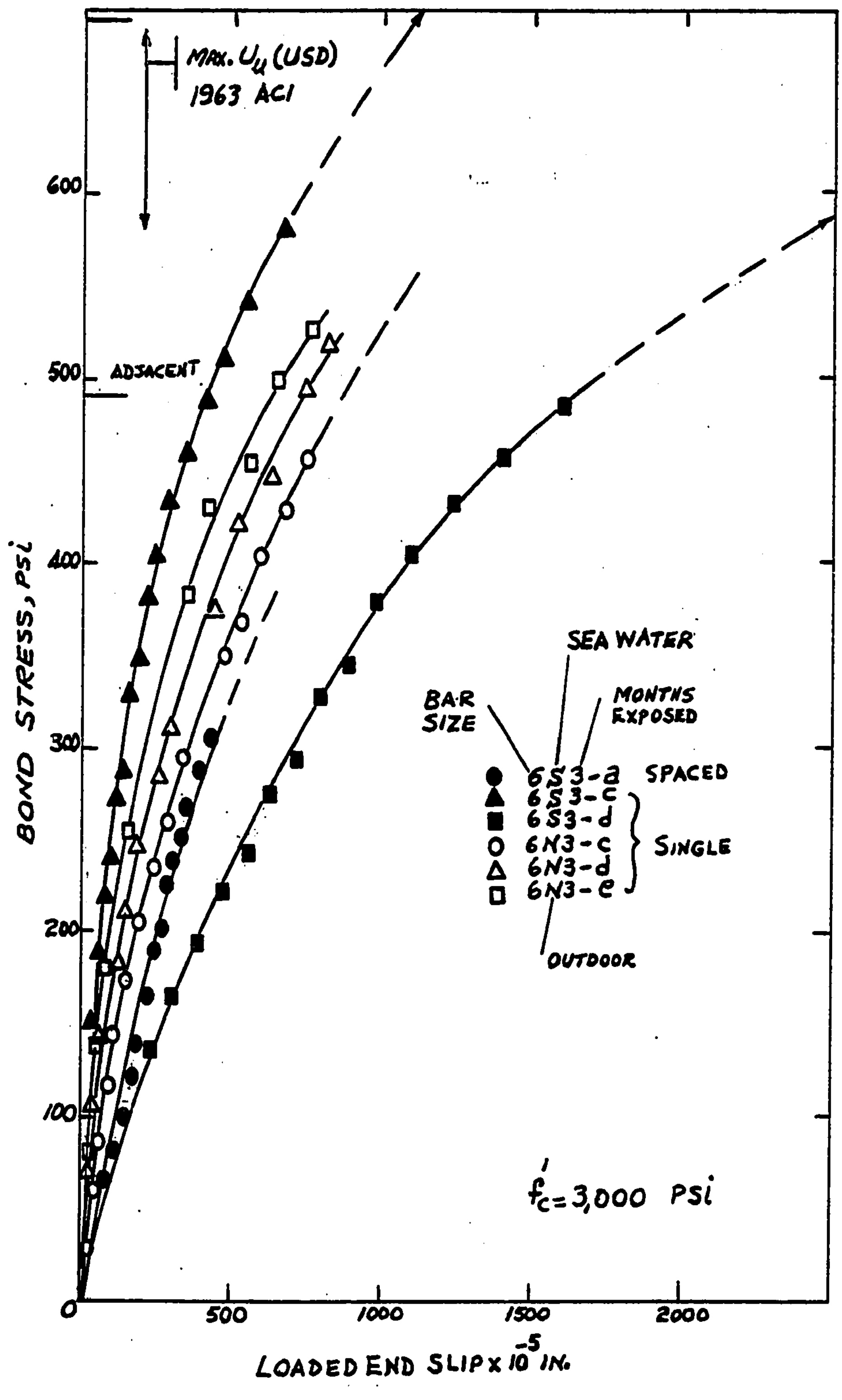


Fig. 5.7 Bond stress-loaded end slip curves for No. 6 bars (3 months corrosion, outdoor & sea water).

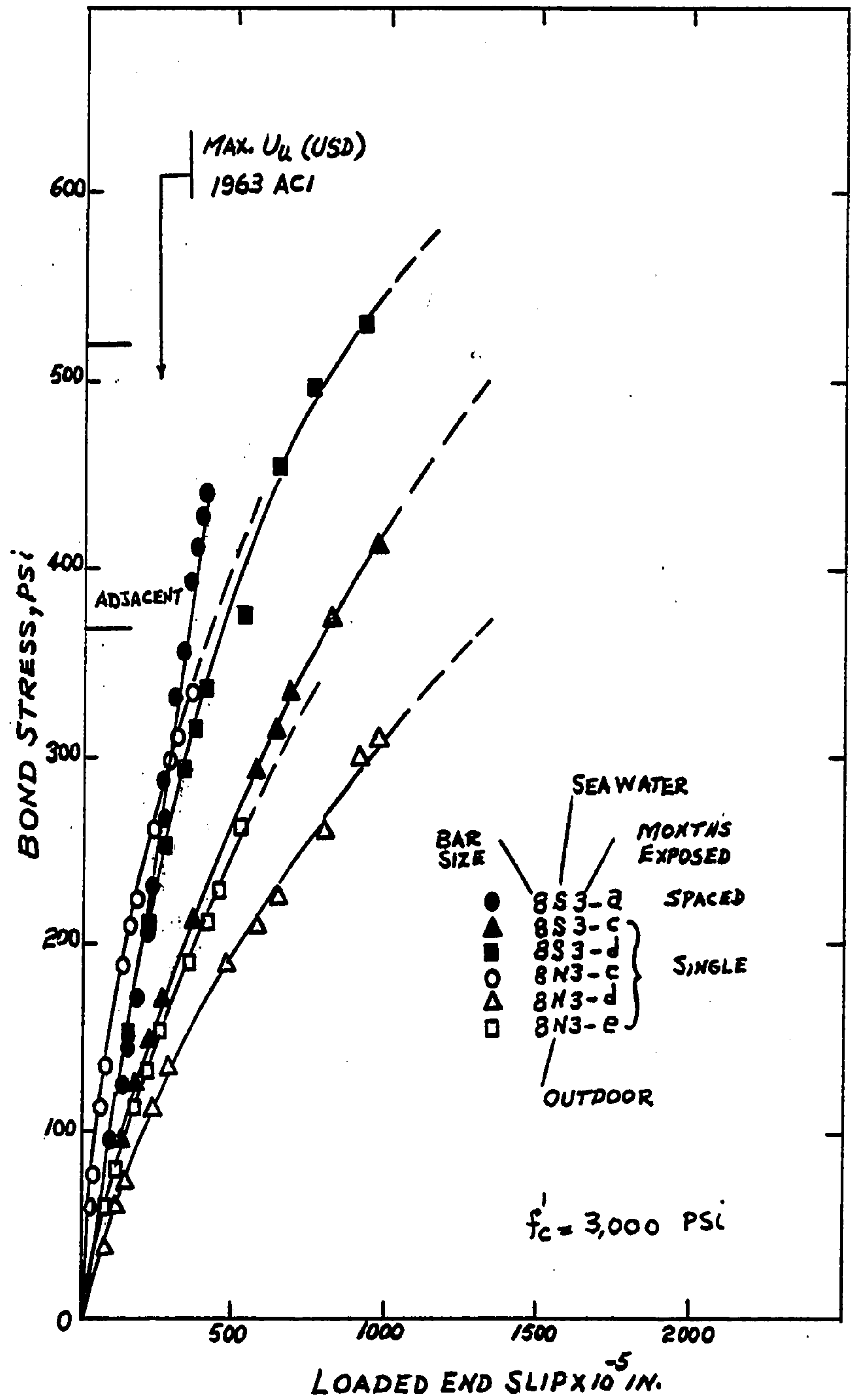


Fig. 5.8 Bond stress-loaded end slip curves for No. 8 bars
(3 months corrosion, outdoor & sea water).

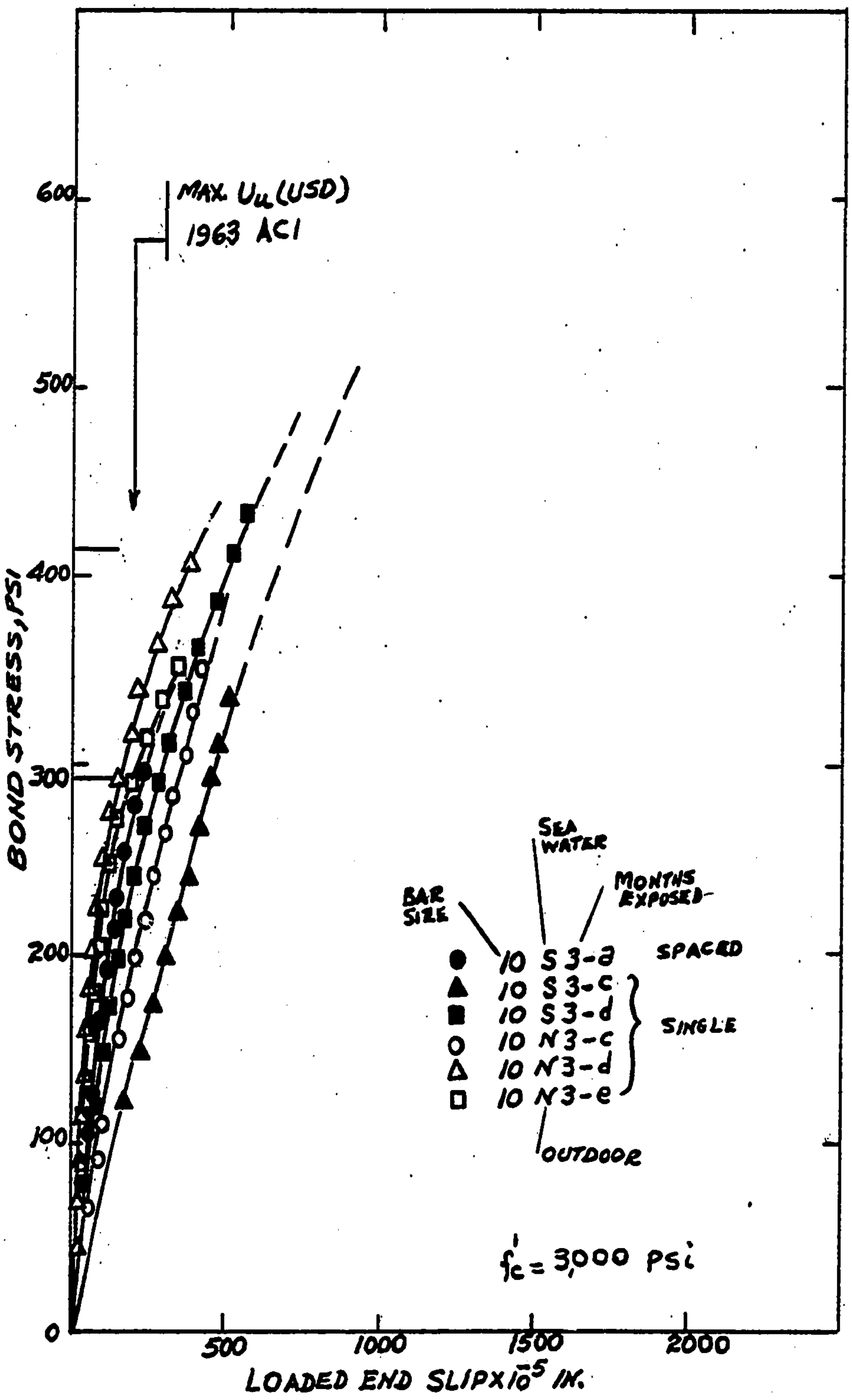


Fig. 5.9 Bond stress-loaded end slip curves for No. 10 bars (3 months corrosion, outdoor & sea water).

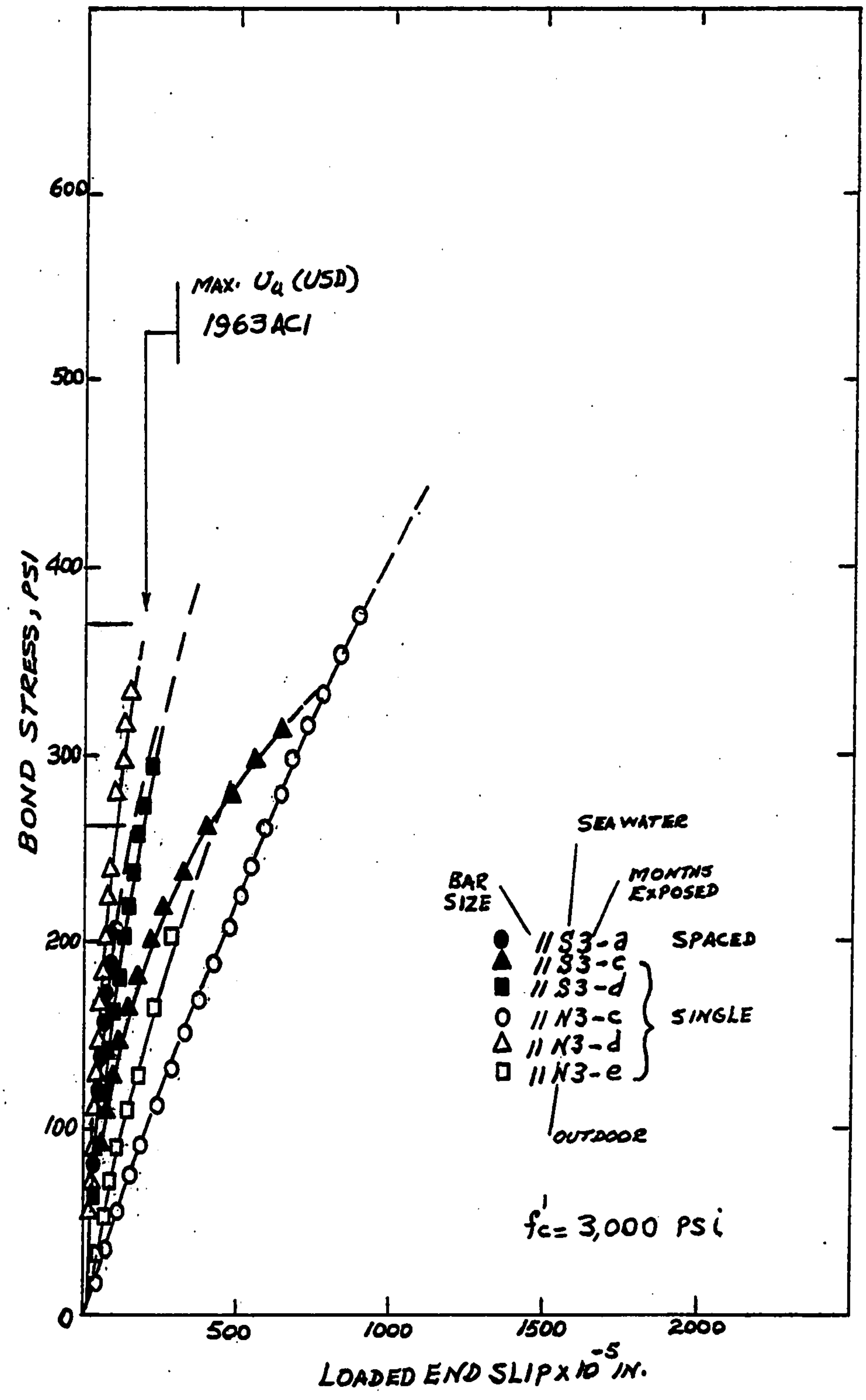


Fig. 5.10 Bond stress-loaded end slip curves for No. 11 bars (3 months corrosion, outdoor & sea water).

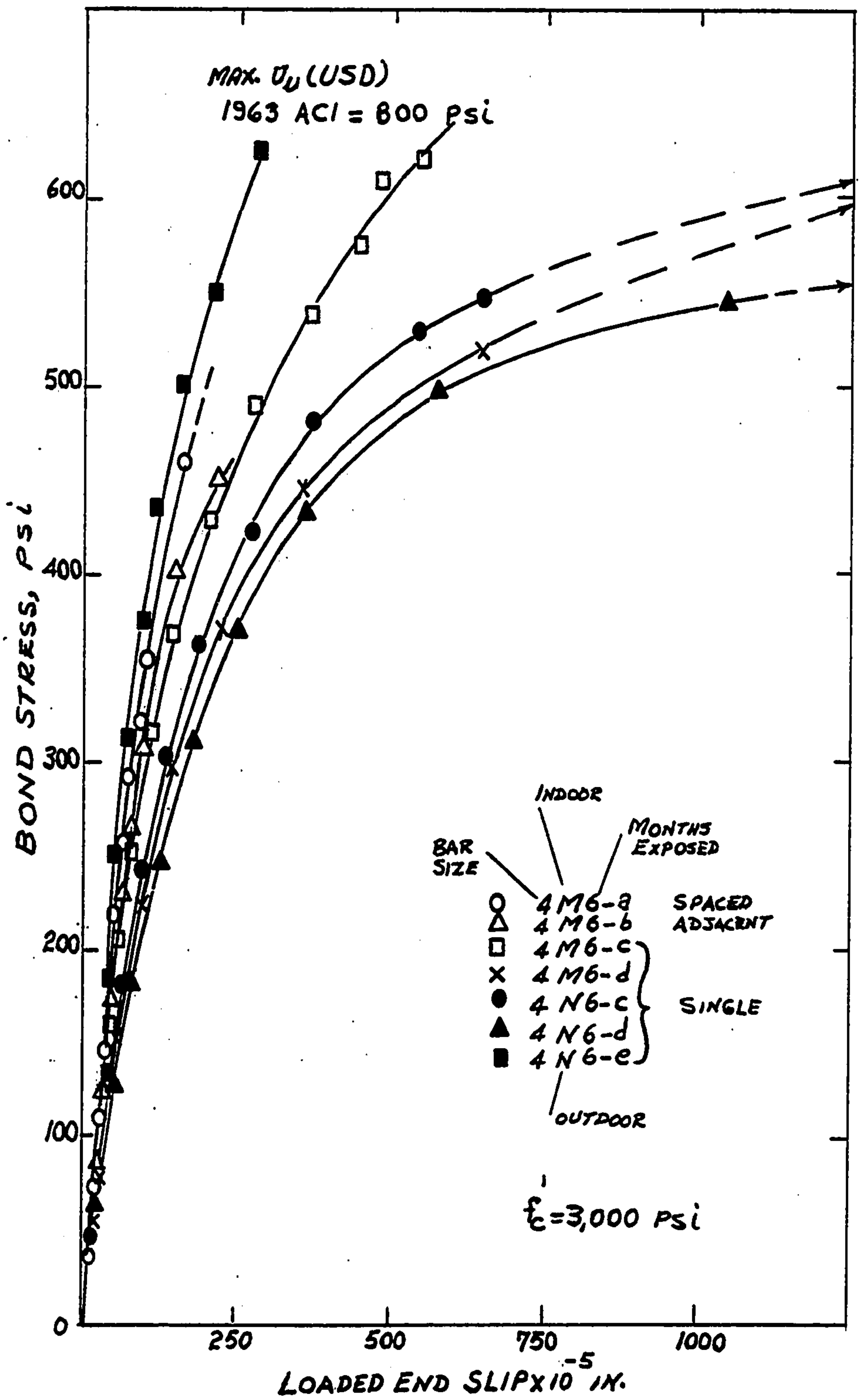


Fig. 5.11 Bond stress-loaded end slip curves for No. 4 bars (6 months corrosion, indoor & outdoor).

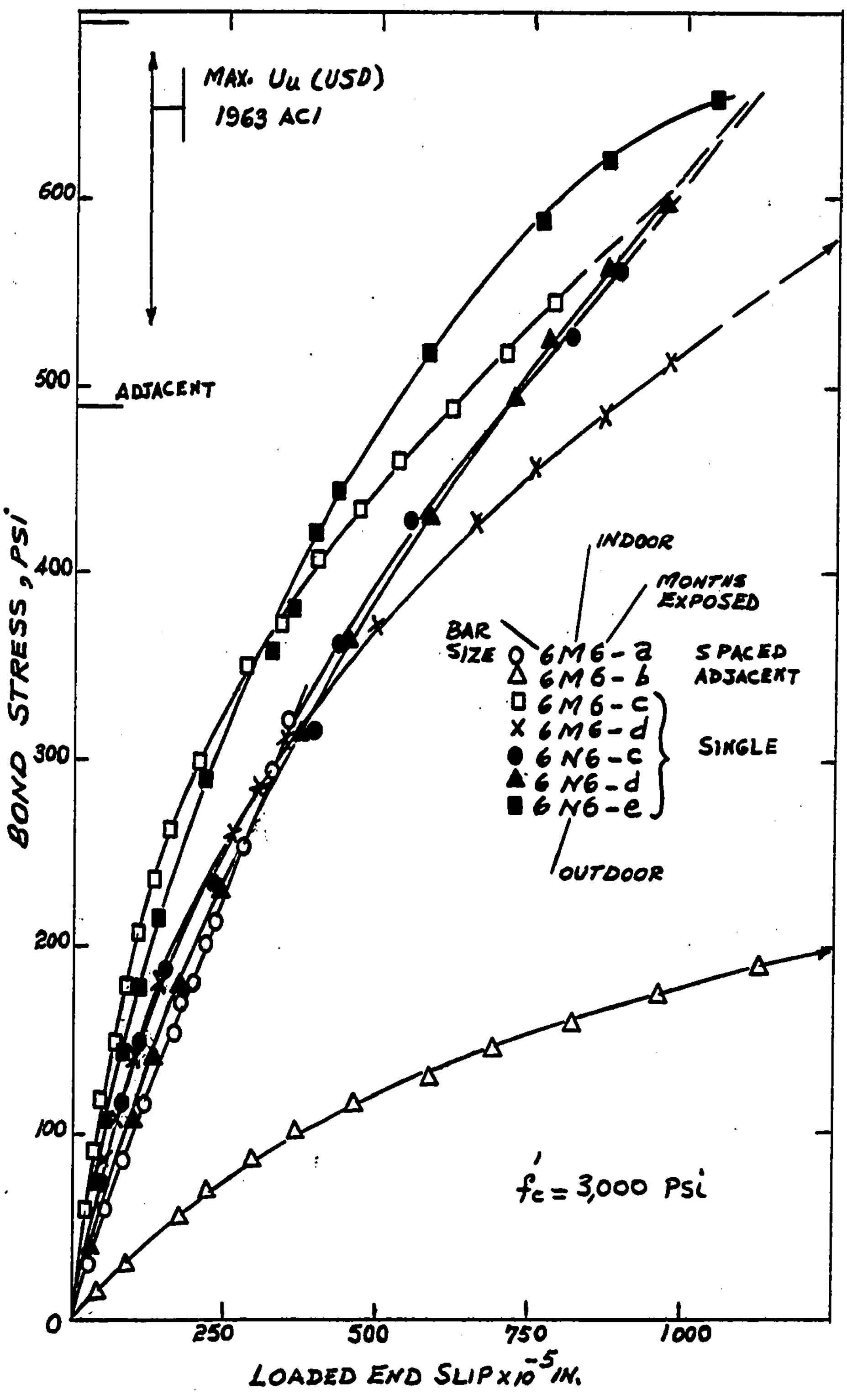


Fig. 5.12 Bond stress-loaded end slip curves for No. 6 bars (6 months corrosion, indoor & outdoor).

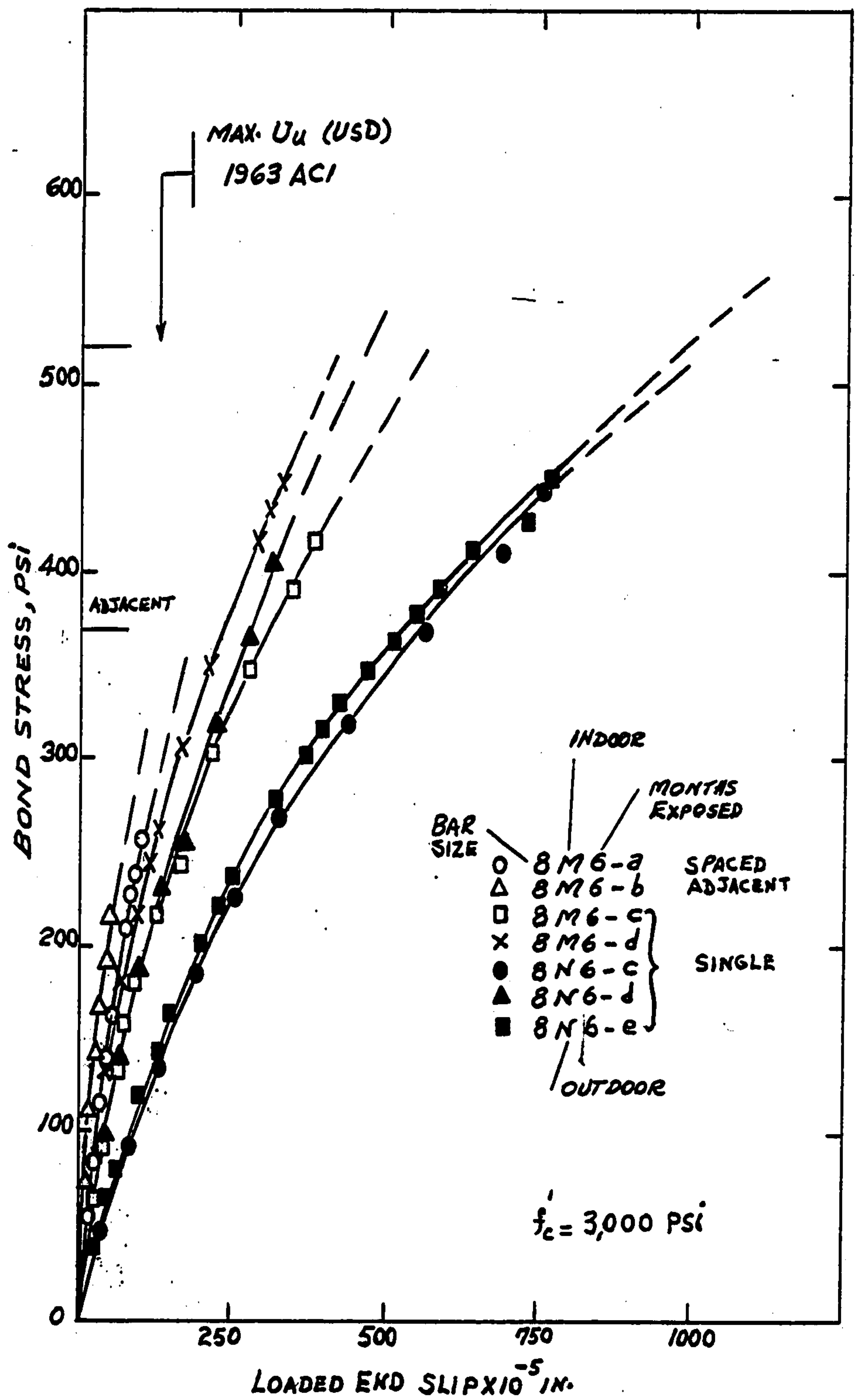


Fig. 5.13 Bond stress-loaded end slip curves for No. 8 bars (6 months corrosion, indoor & outdoor).

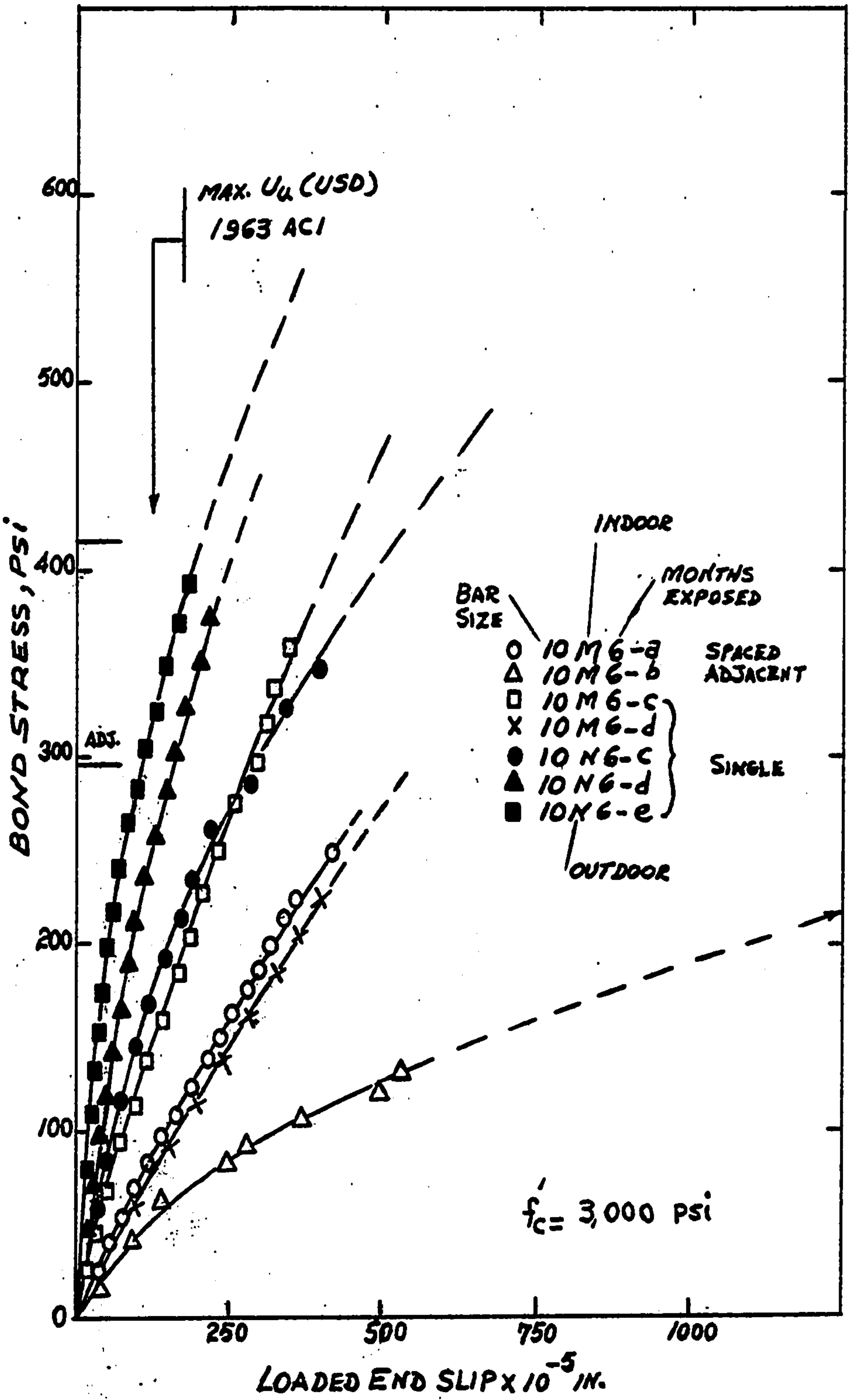


Fig. 5.14 Bond stress-loaded end slip curves for No. 10 bars (6 months corrosion, indoor & outdoor).

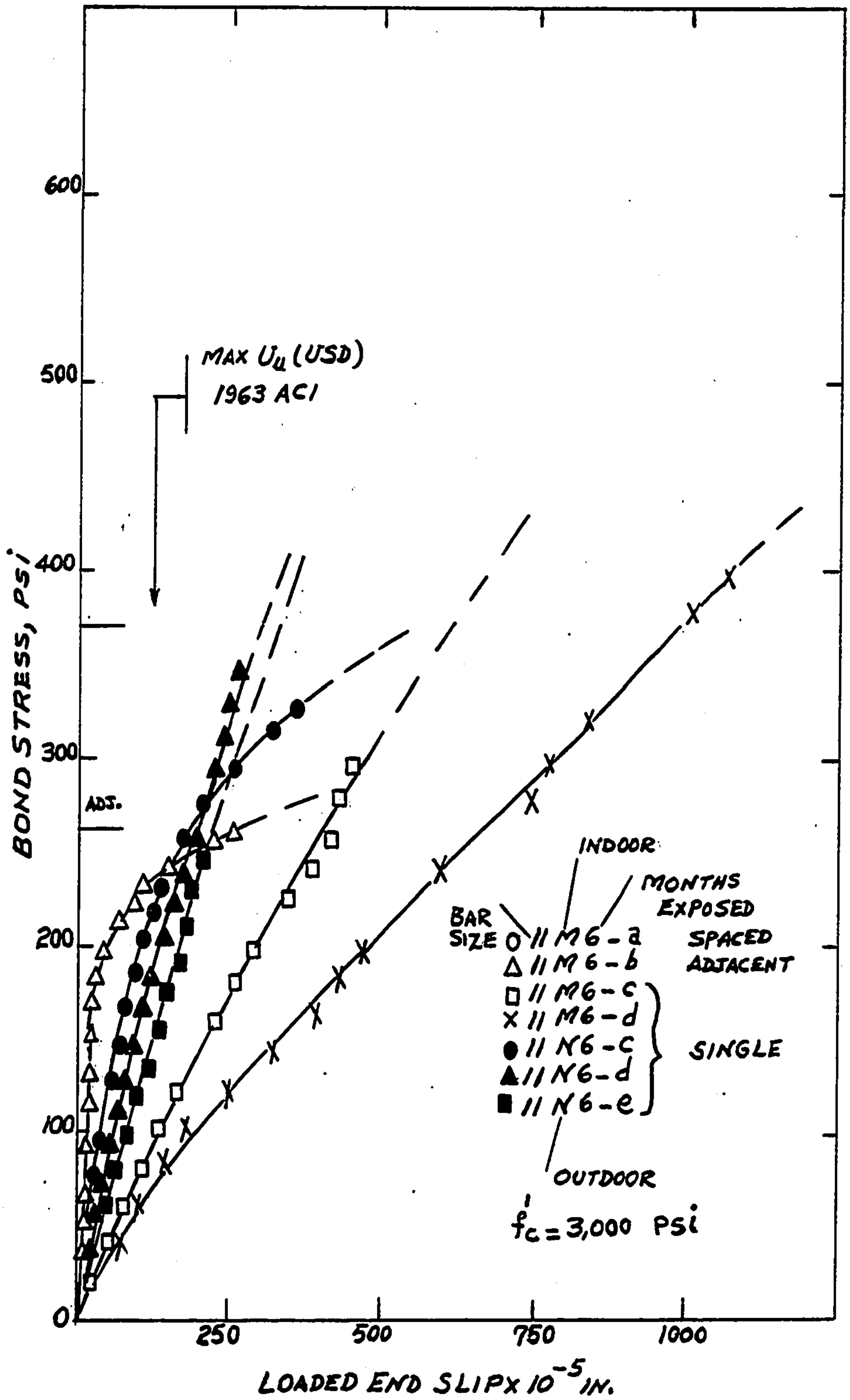


Fig. 5.15 Bond stress-loaded end slip curves for No. 11 bars. (6 months corrosion, indoor & outdoor)

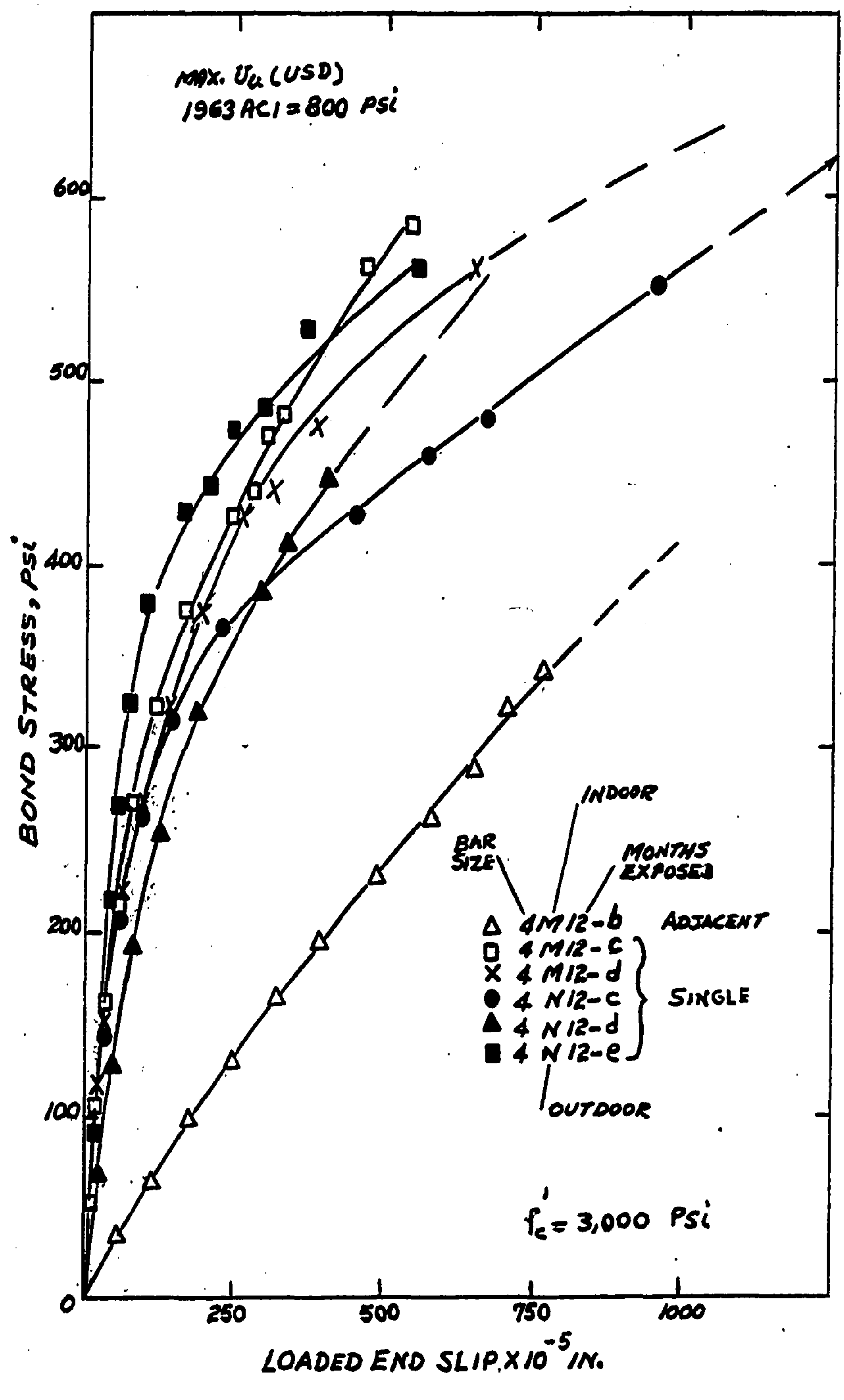


Fig. 5.16 Bond stress-loaded end slip curves for No. 4 bars (12 months corrosion, indoor & outdoor)

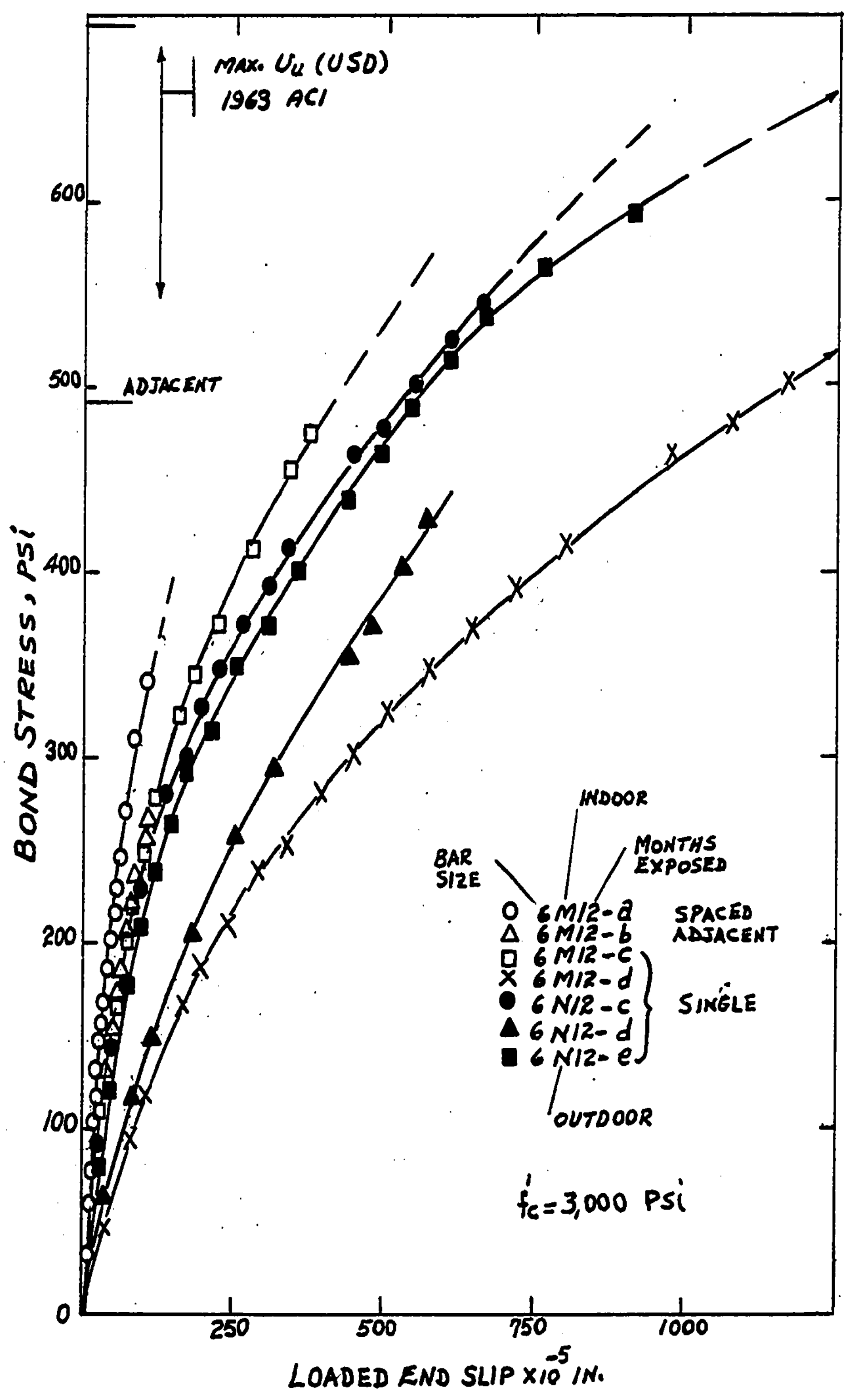


Fig. 5.17 Bond stress-loaded end slip curves for No. 6 bars. (12 months corrosion, indoor & outdoor)

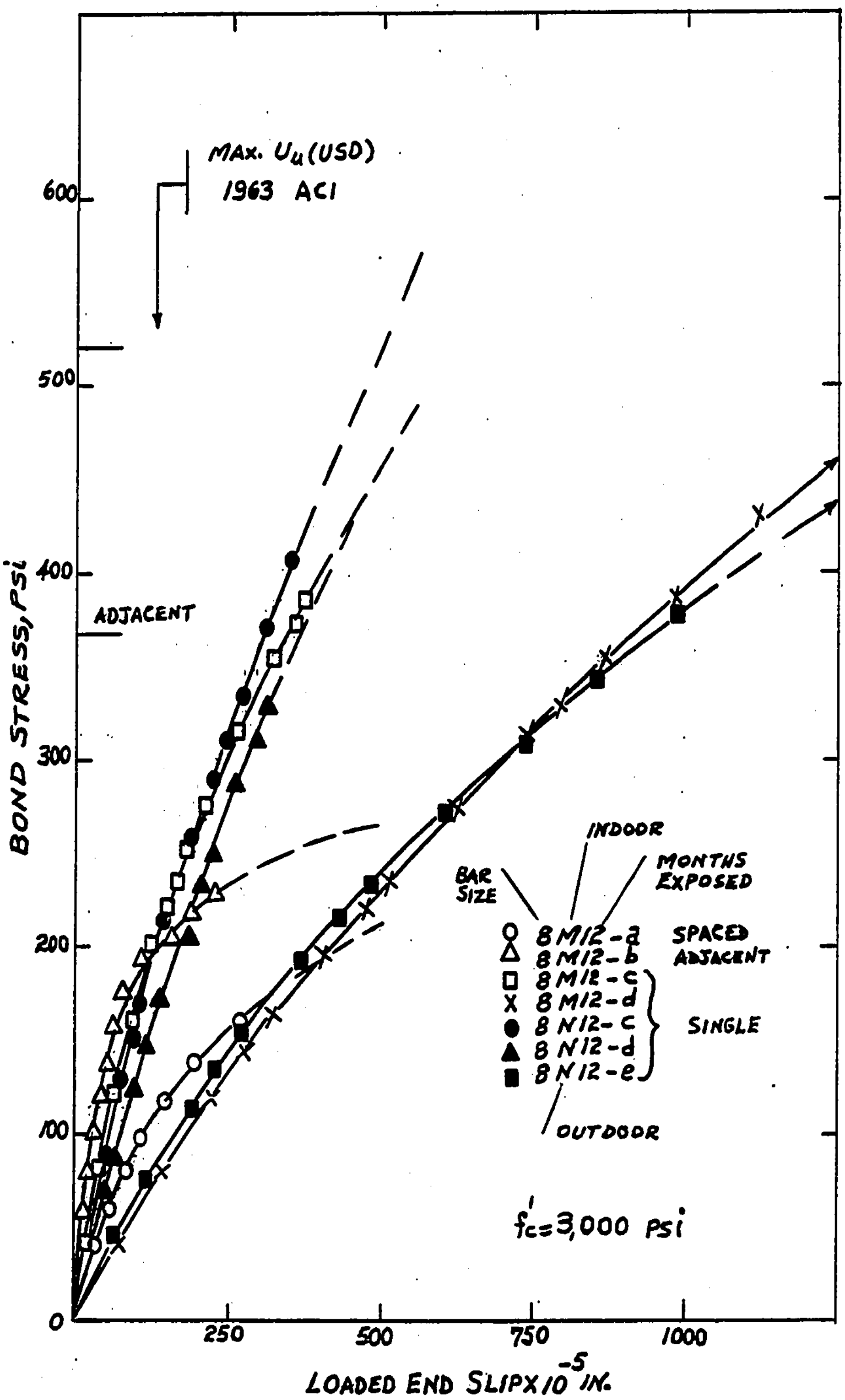


Fig. 5.18 Bond stress-loaded end slip curves for No. 8 bars. (12 months corrosion, indoor & outdoor)

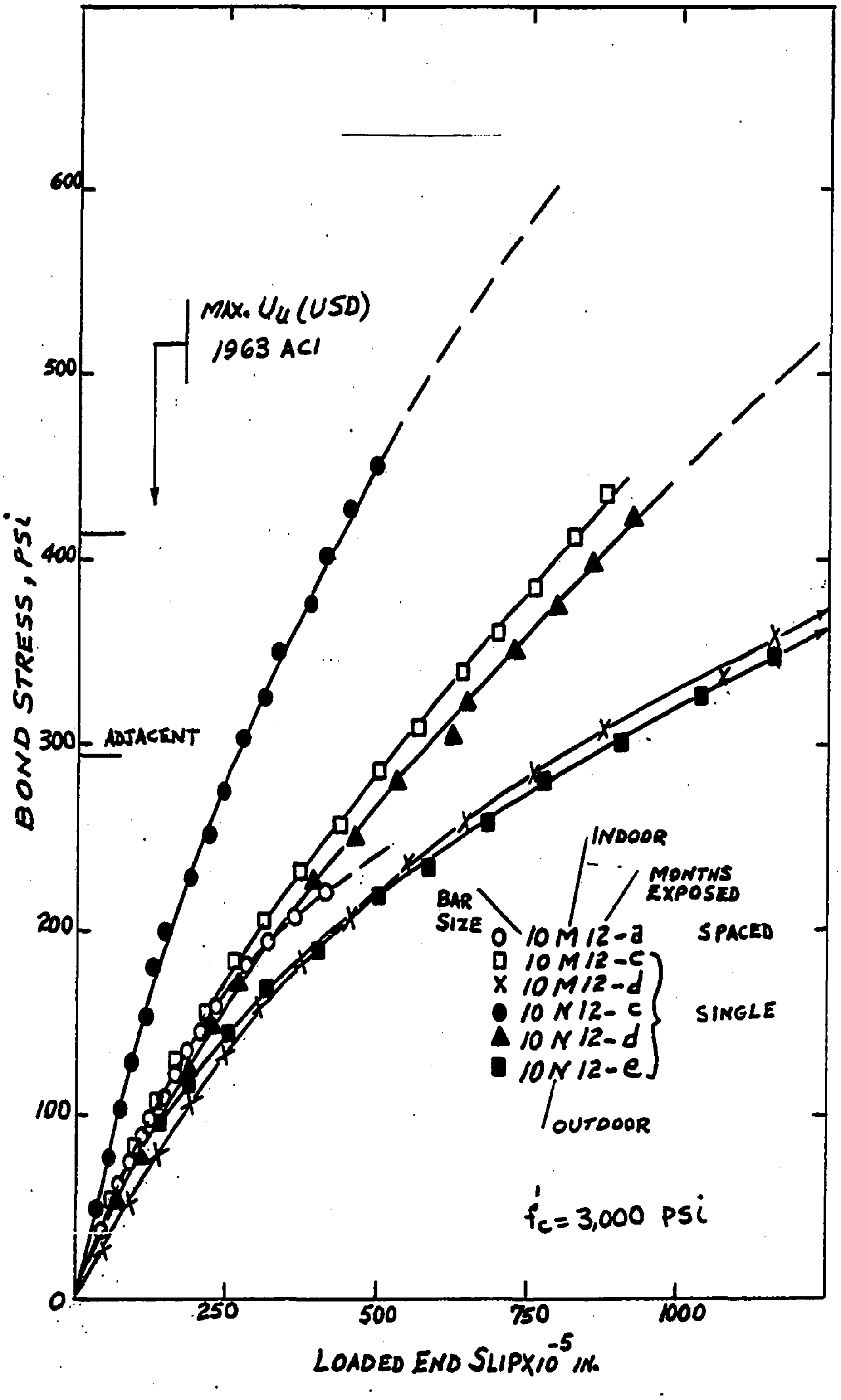


Fig. 5.19 Bond stress-loaded end slip curves for No. 10 bars.
(12 months corrosion, indoor & outdoor)

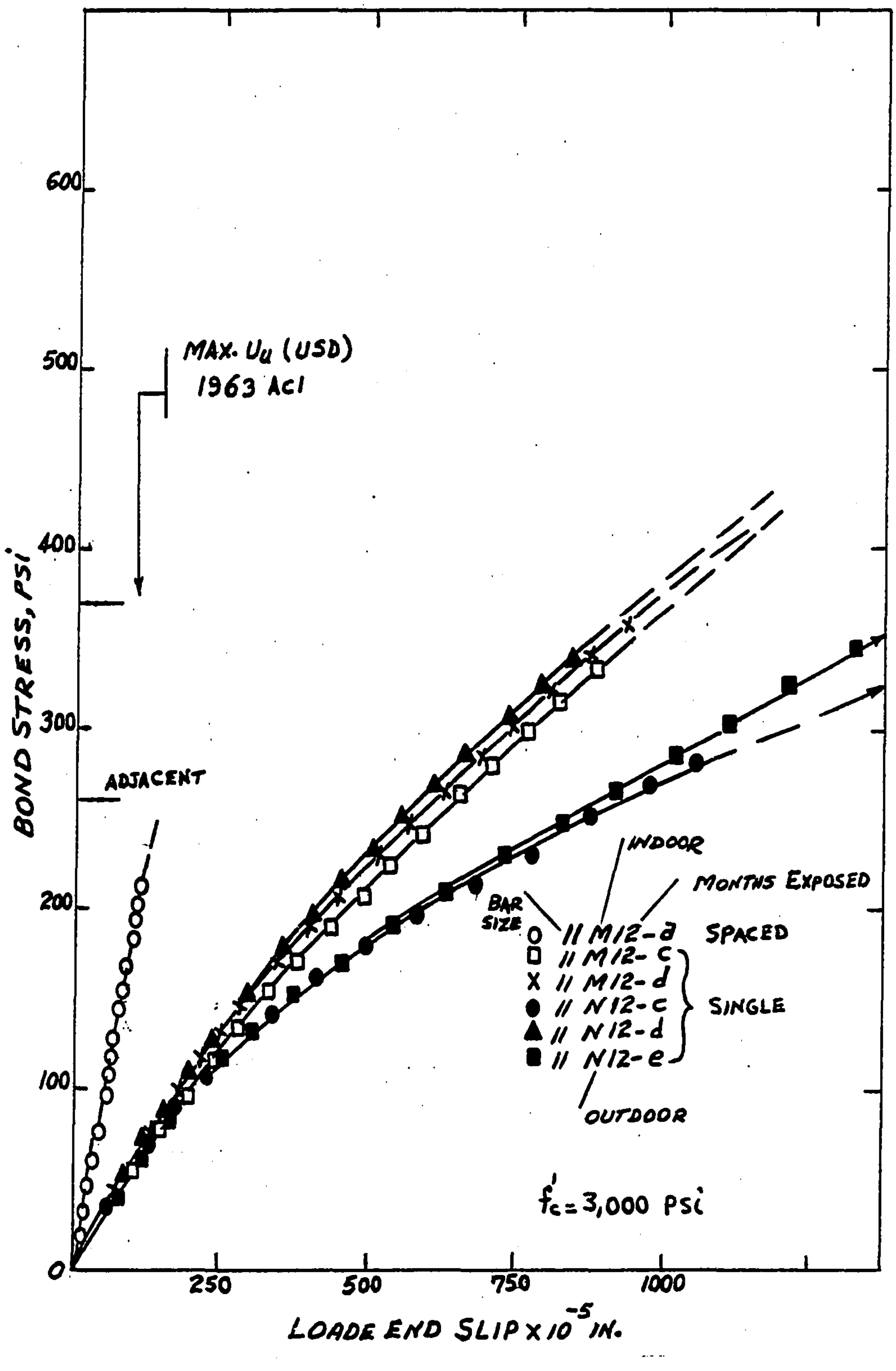


Fig. 5.20 Bond stress-loaded end slip curves for No. 11 bars .
(12 months corrosion, indoor & outdoor)

In order to evaluate the effect of binding on bond strength and on the splitting behavior of the concrete specimens, three single-bar specimens were cast with Nos. 8,10, and 11 reinforcing bars with web reinforcement. In comparison with the specimens with no web reinforcement, the specimens with web reinforcement developed a slightly higher bond stresses, but the failure mechanism was the same.

The maximum permissable bond stresses recommended by the 1963 ACI Code are shown on Figs. 5.1 - 5.20. These stresses were calculated from the equation: $u_u = \frac{9.5}{D}\sqrt{f'_c}$ given in Sec. 1801-c of the Code, where f'_c is the concrete compressive strength and D is the bar diameter. In the case of the specimens with double bars tied adjacent, the two bars were replaced with a single bar and the equivalent bar diameter was used in calculating the allowable bond stress.

5.3 Specimen Behavior and Cracking Pattern

The behavior of the specimens were further exemplified by observing the crack pattern as they developed during the tests. This was accomplished by marking the macroscopically visible cracks on the surfaces of the specimens at different load levels, Figs. 5.21- 5.35. No shrinkage cracks were observed. The arrows on the above figures indicate the sequence of appearence of cracks at different load levels. The number of specimens with the same crack pattern are also indicated.

A study of these crack patterns was made in an attempt to determine the mechanism of failure and to establish the influence of different degrees of surface corrosion and bar spacings on the mechanism. It was observed that, in general, cracking started at a load of about

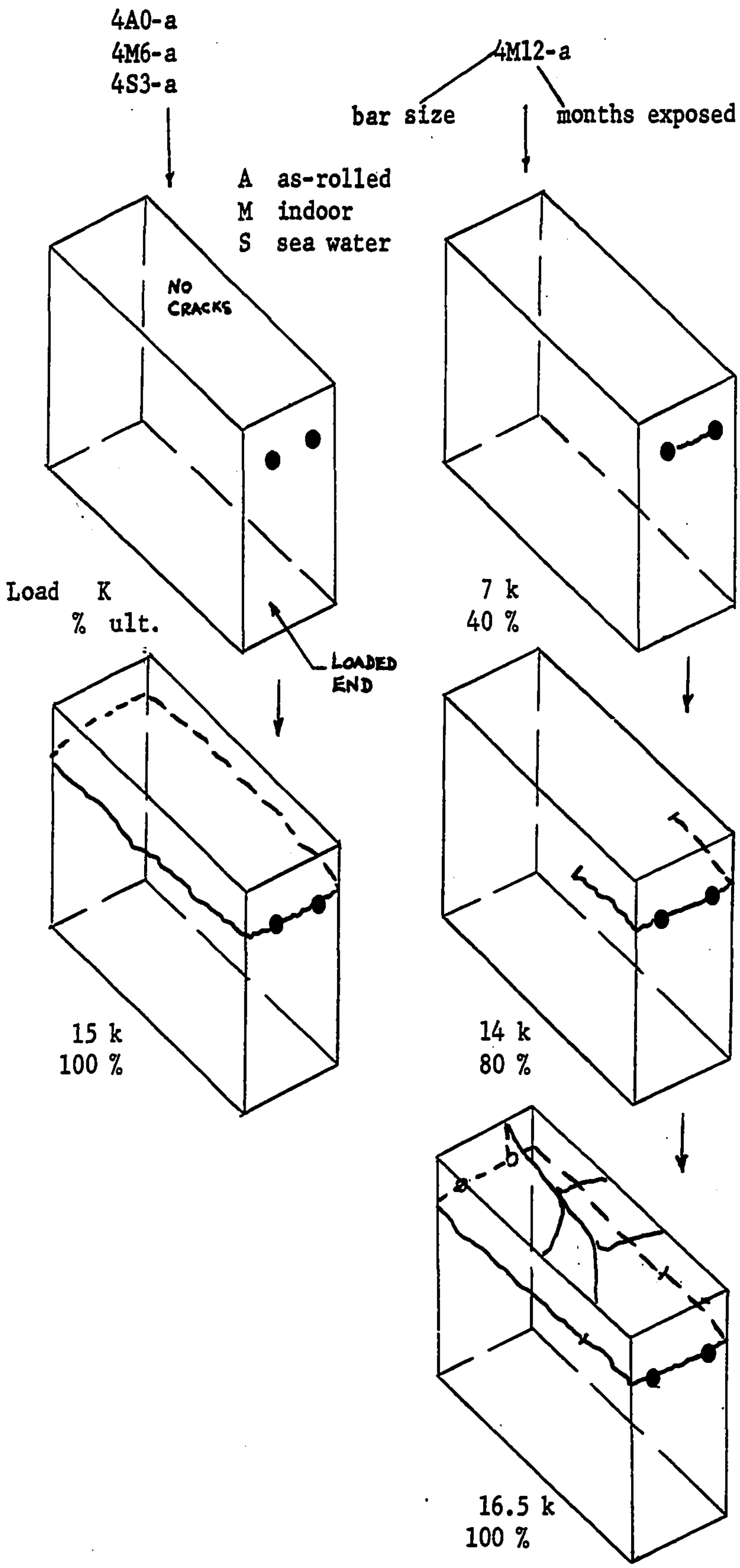


Fig. 5.21 Crack pattern for specimens with No. 4 double spaced bars.

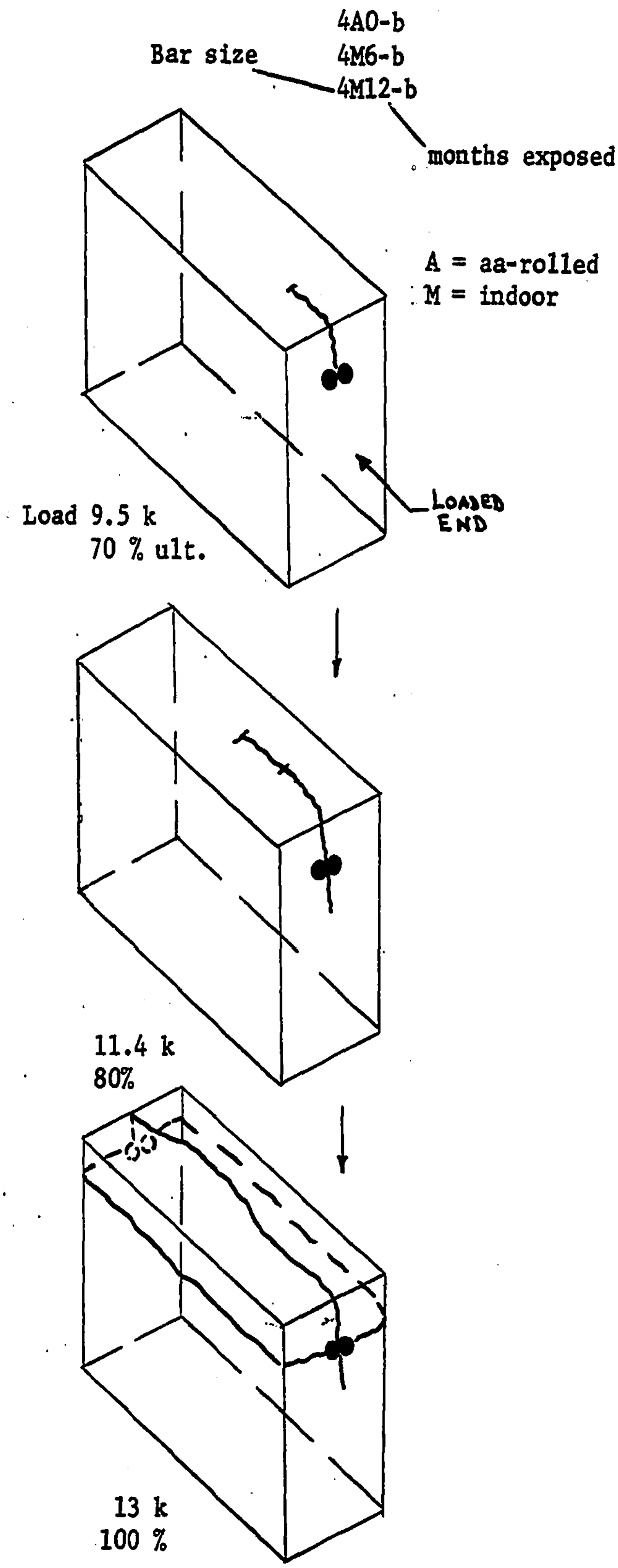


Fig. 5.22 Crack pattern for specimens with No. 4 double adjacent bars

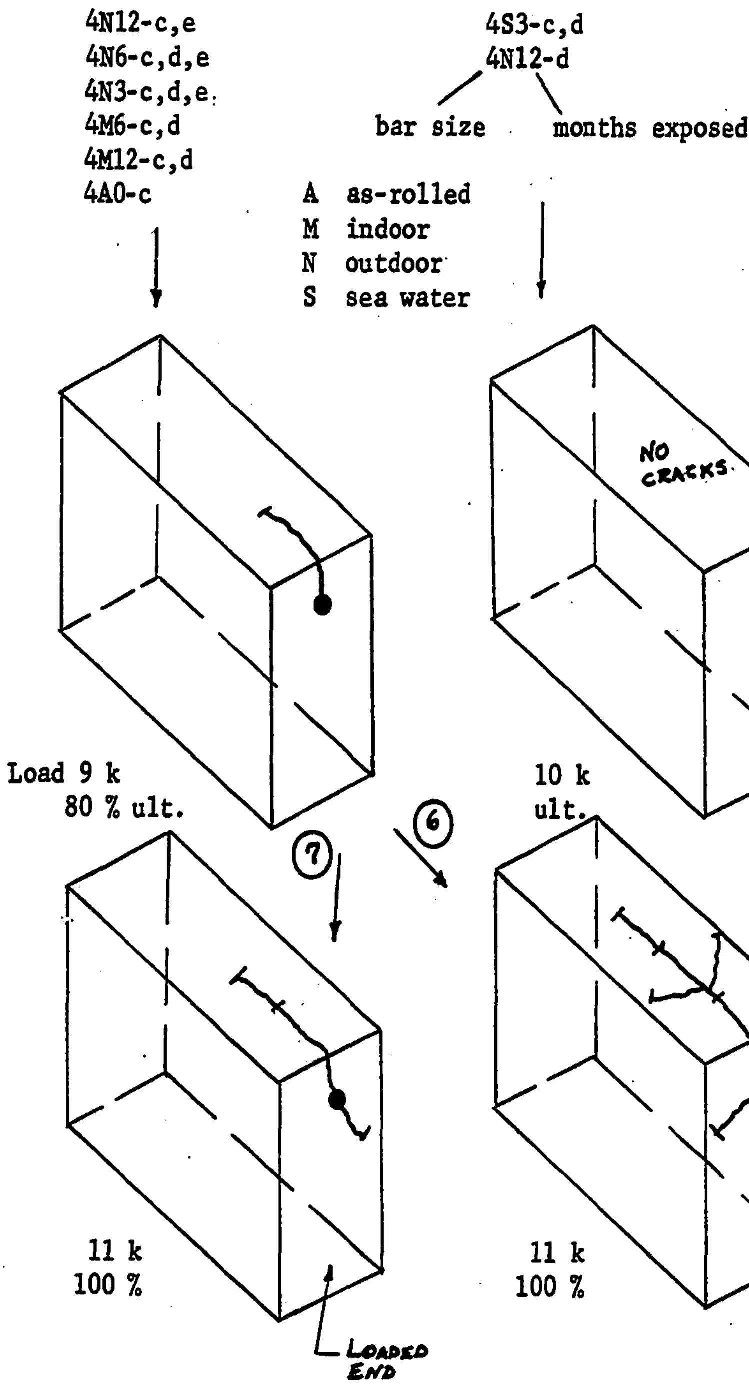


Fig. 5.23 Crack pattern for specimens with No 4 single bar.

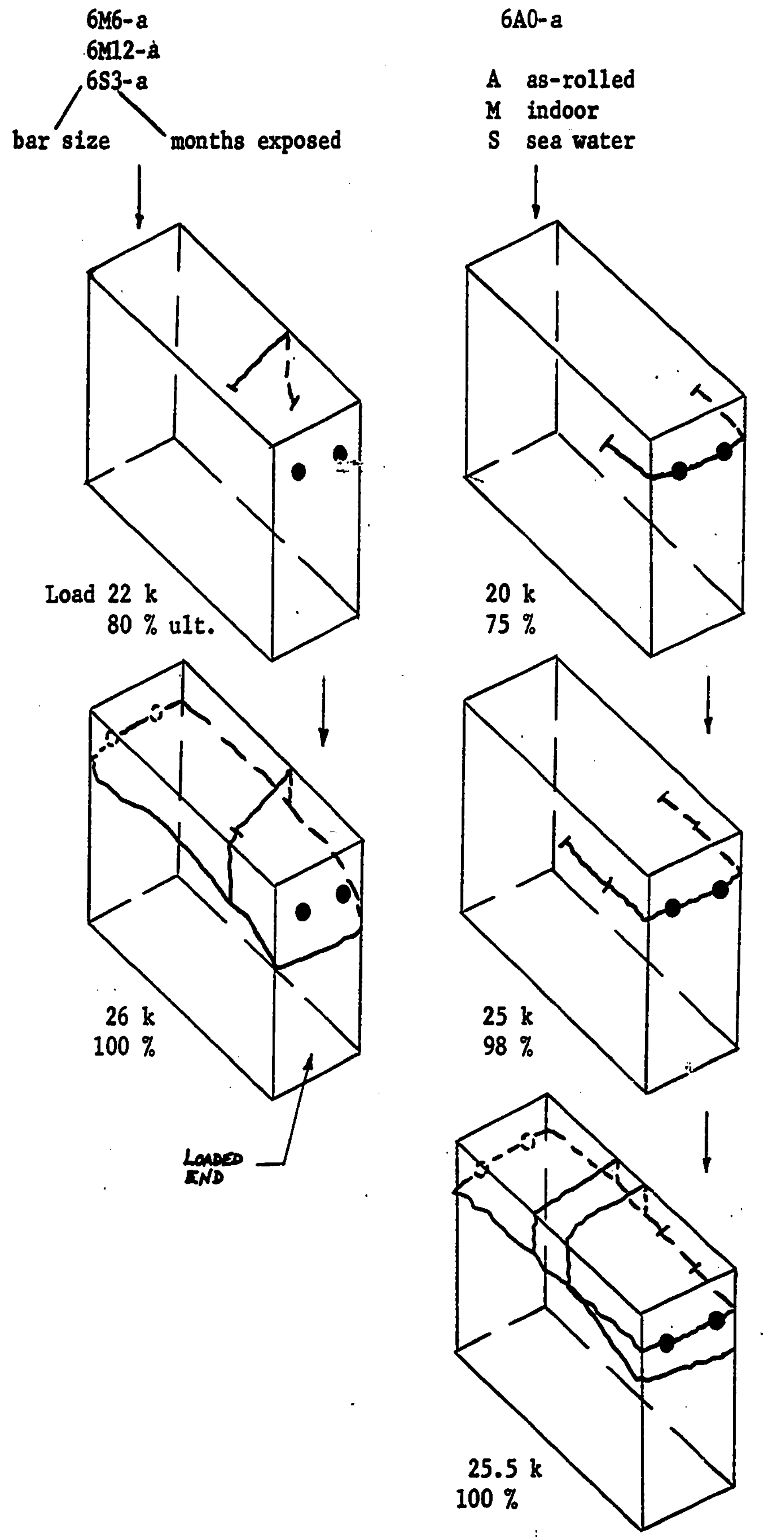


Fig. 5.24 Crack pattern for specimens with No. 6 double spaced bars.

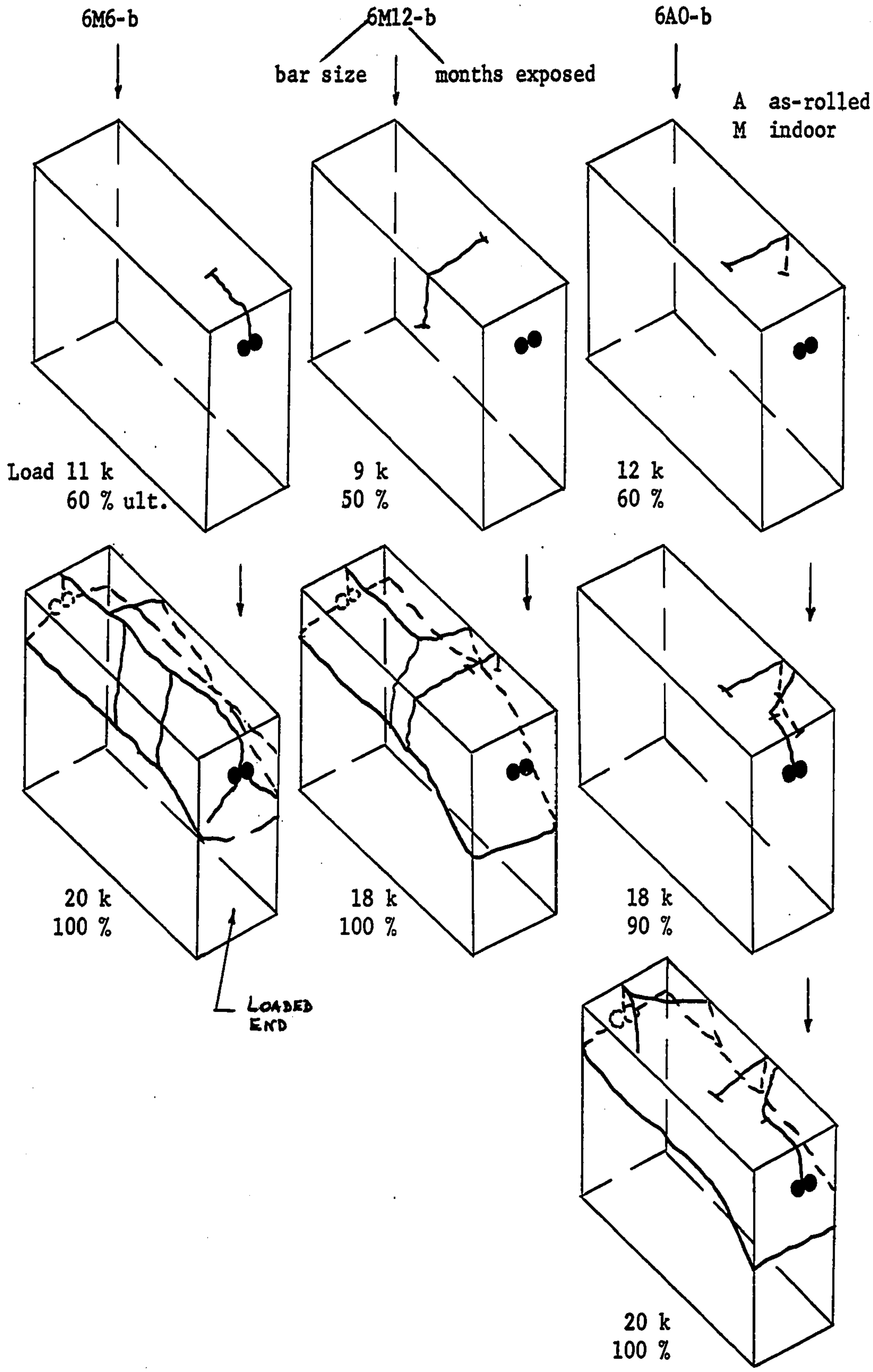


Fig. 5.25 Crack pattern for specimens with No 6 double adjacent bars.

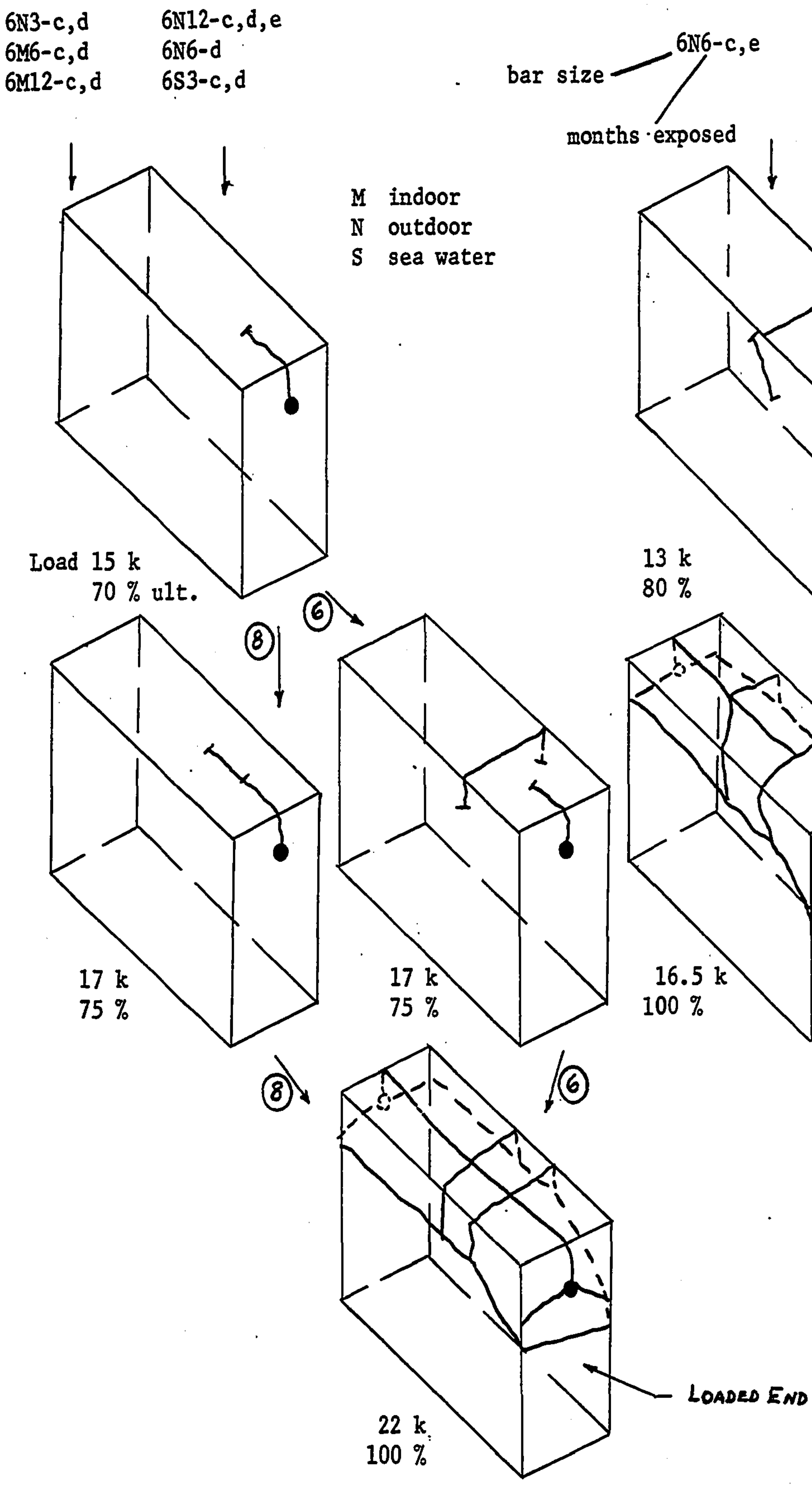


Fig. 5.26 Crack pattern for specimens with No. 6 single bar

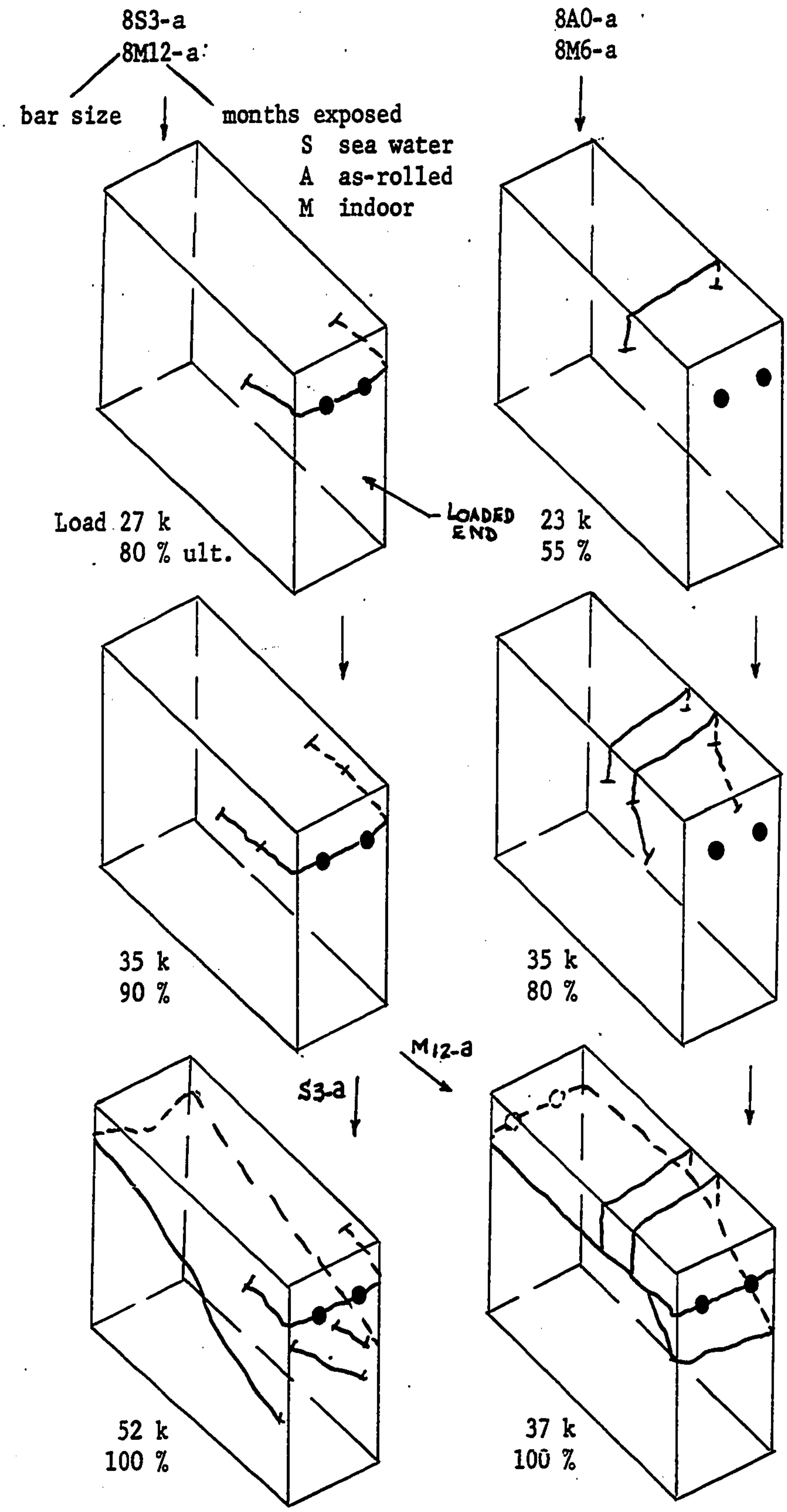


Fig. 5.27 Crack pattern for specimens with No. 8 double spaced bars.

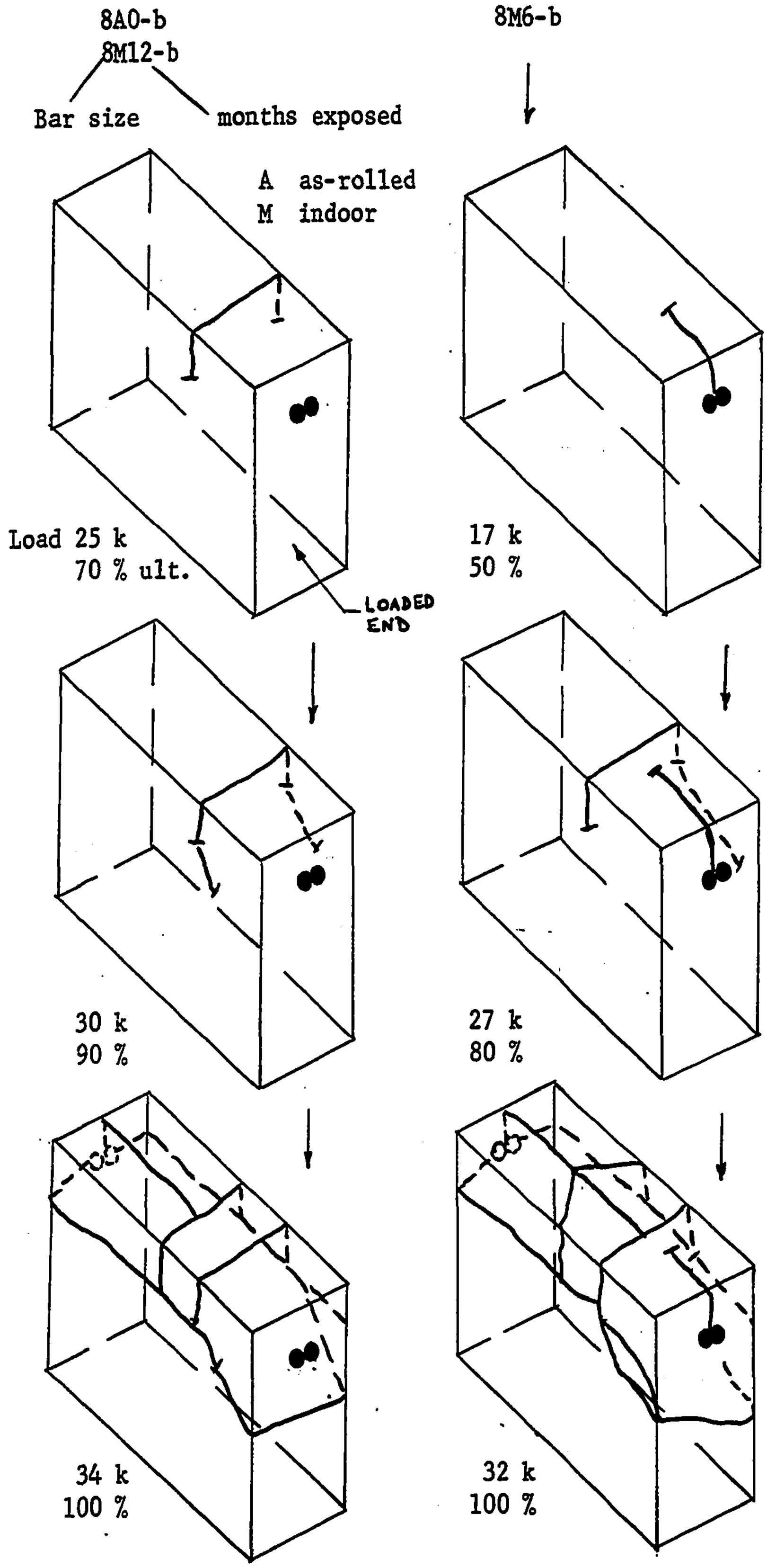


Fig. 5.28 Crack pattern for specimens with No. 8 double adjacent bars.

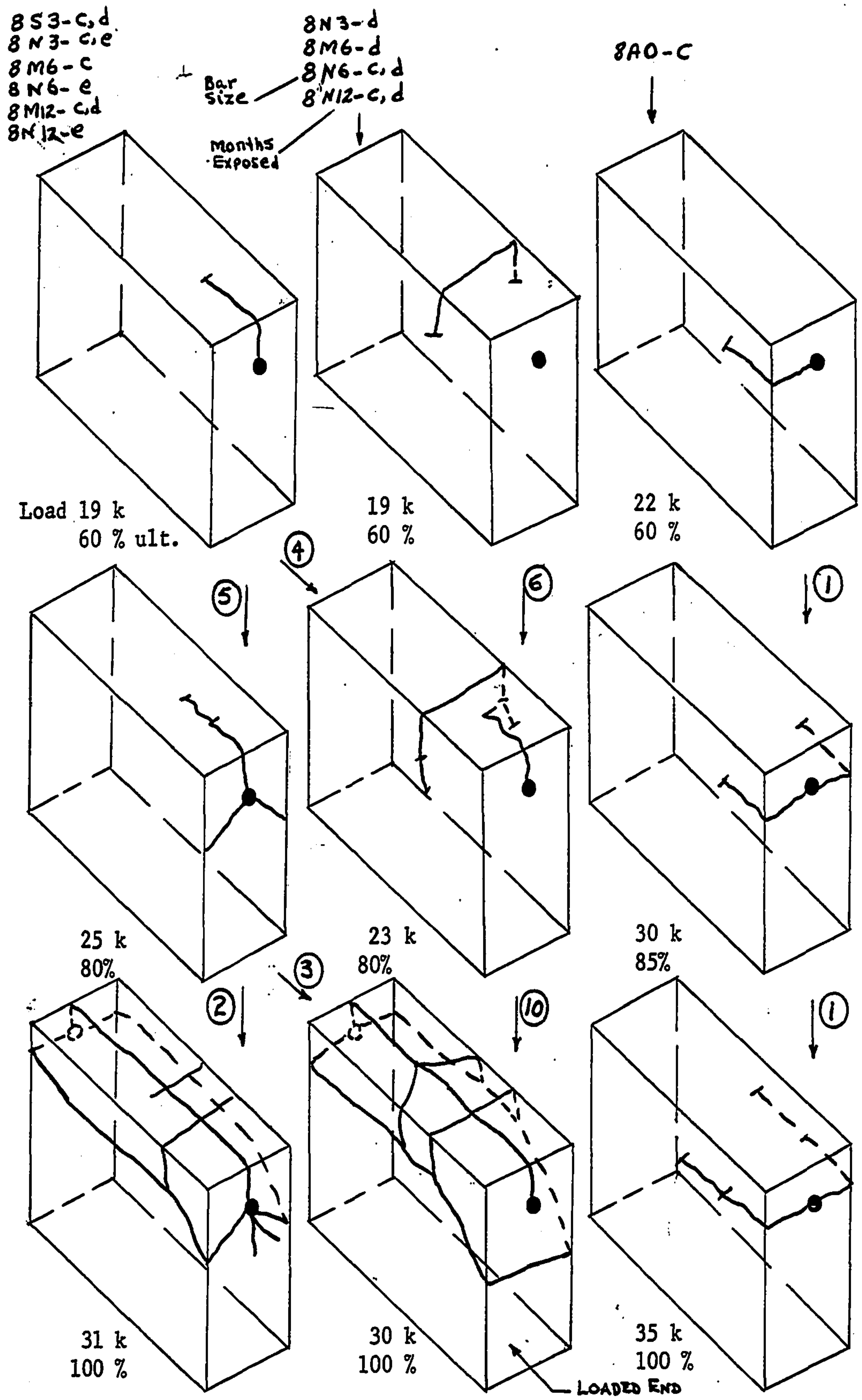


Fig. 5.29 Crack pattern for specimens with No. 8 single bar.

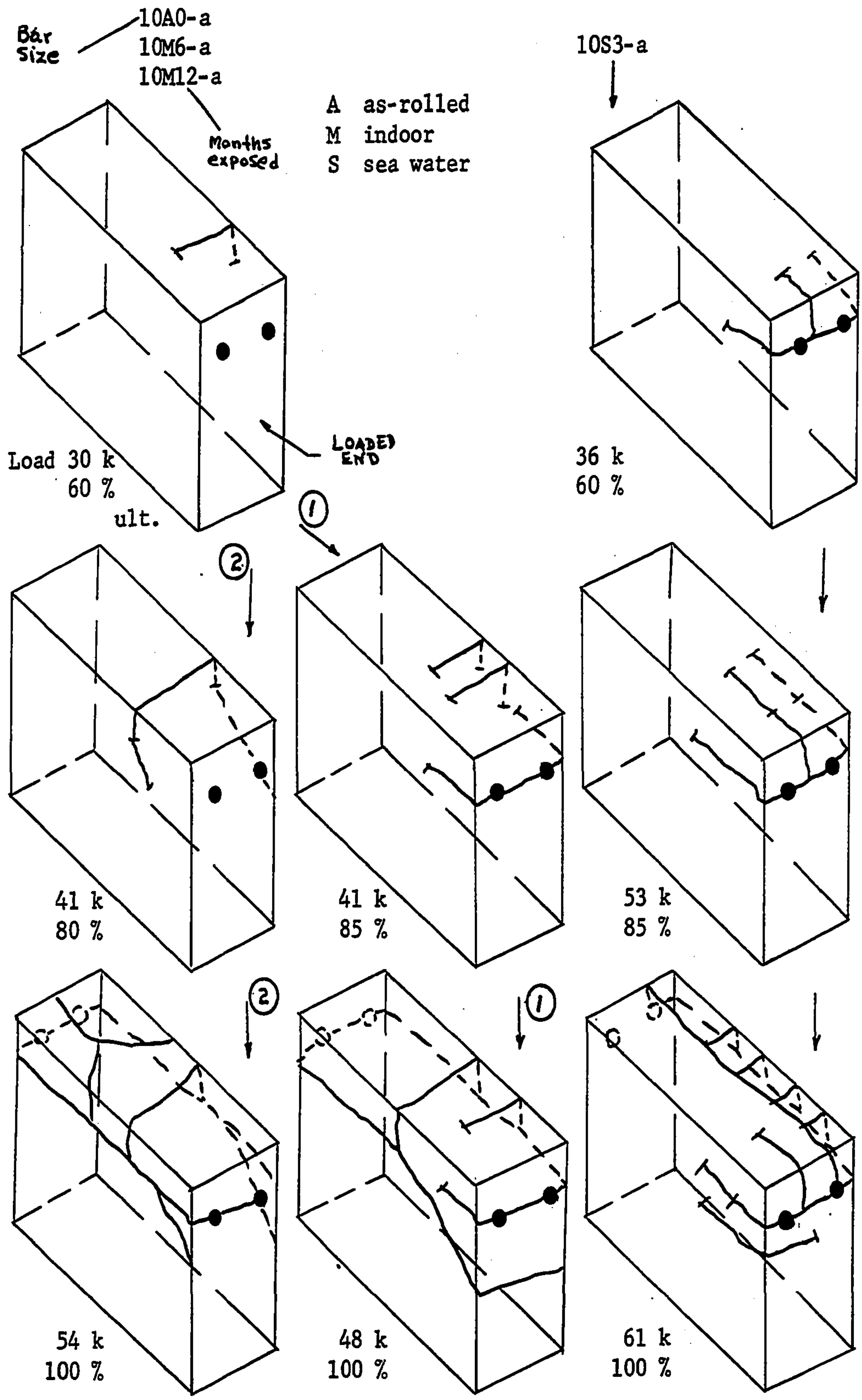


Fig. 5.30 Crack pattern for specimens with No. 10 double spaced bars.

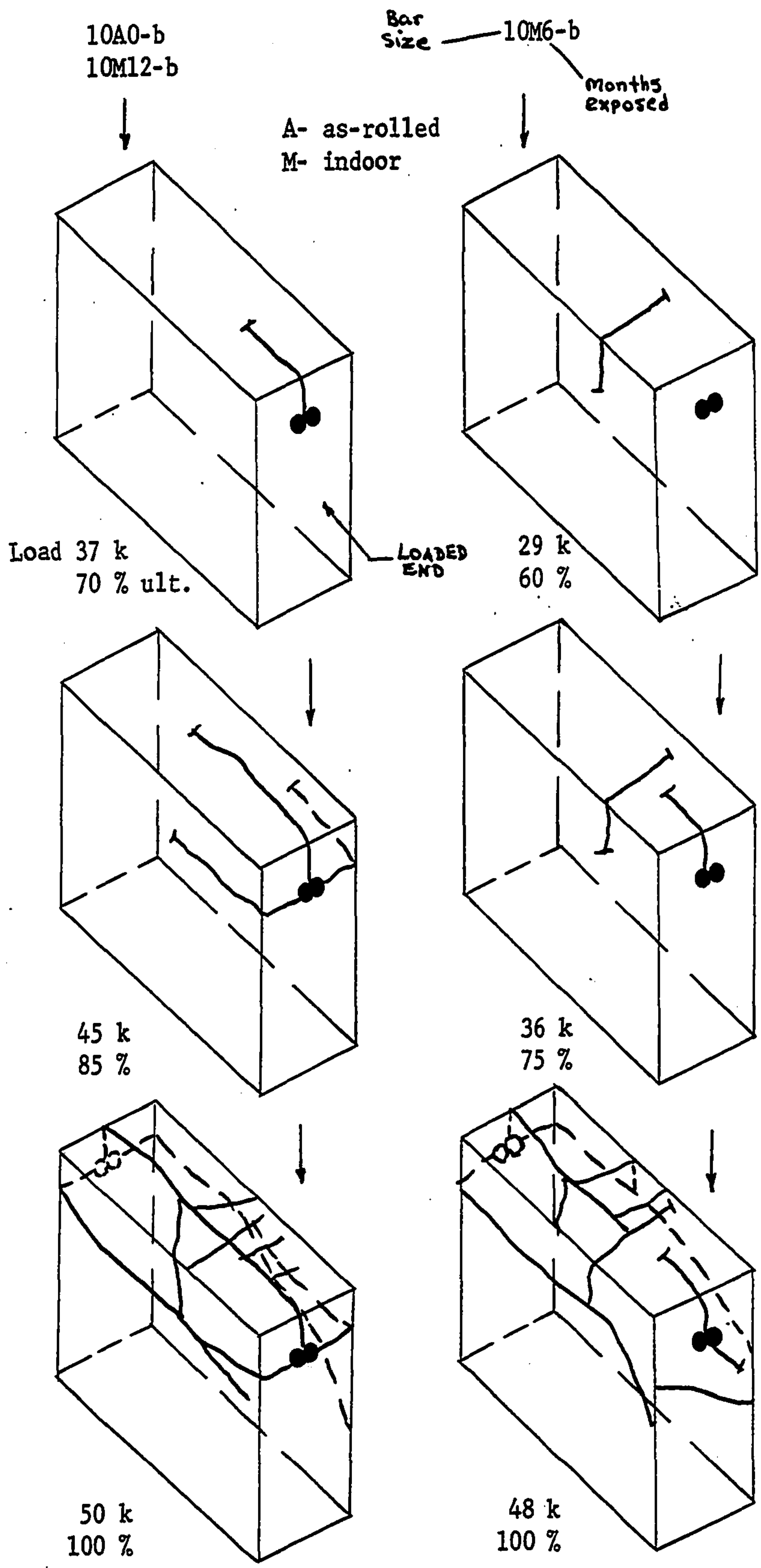


Fig. 5.31 Crack pattern for specimens with No. 10 double adjacent bars.

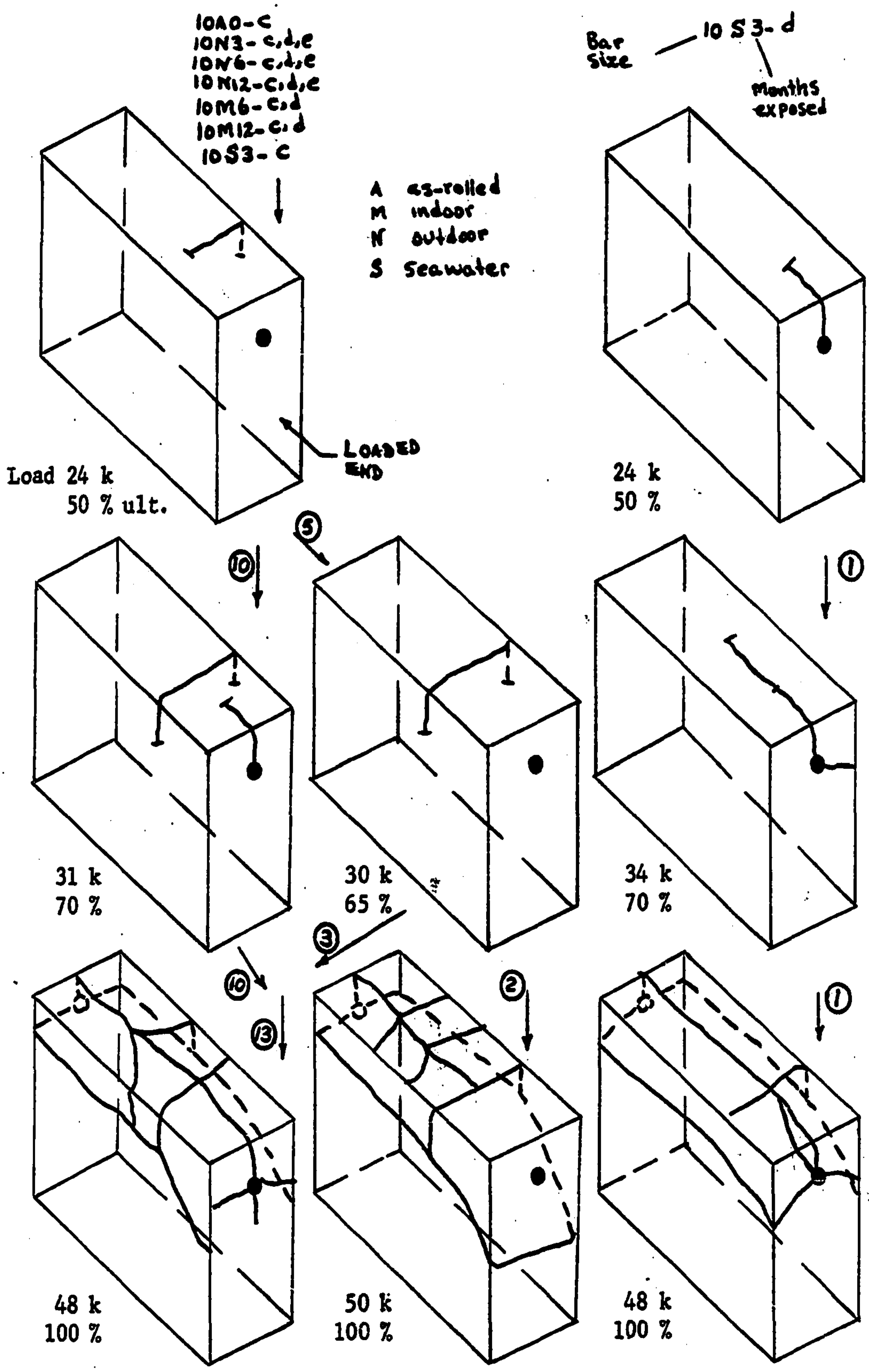


Fig. 5.32 Crack pattern for specimens with No. 10 single bar.

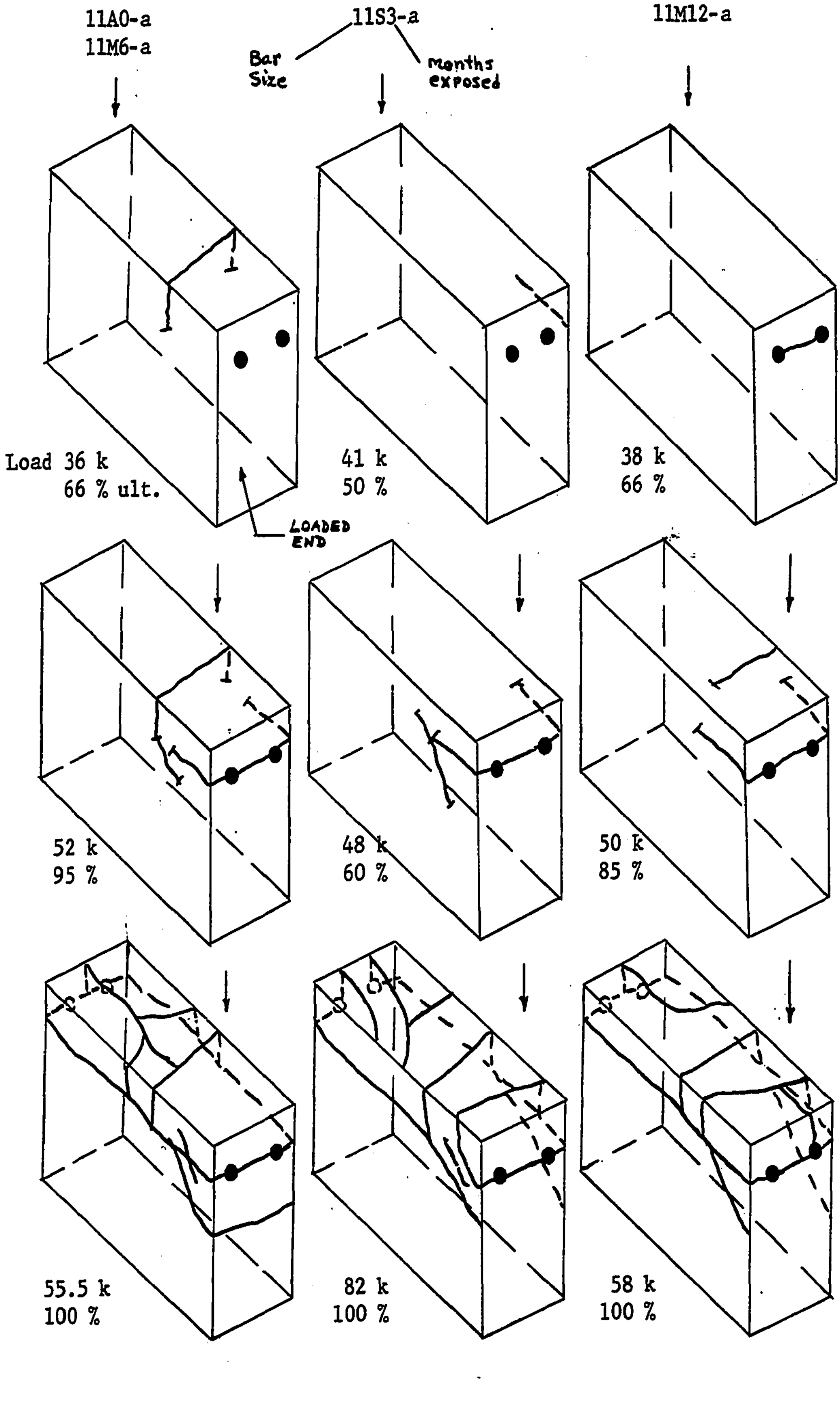


Fig. 5.33 Crack pattern for specimens with No. 11 double spaced bars.

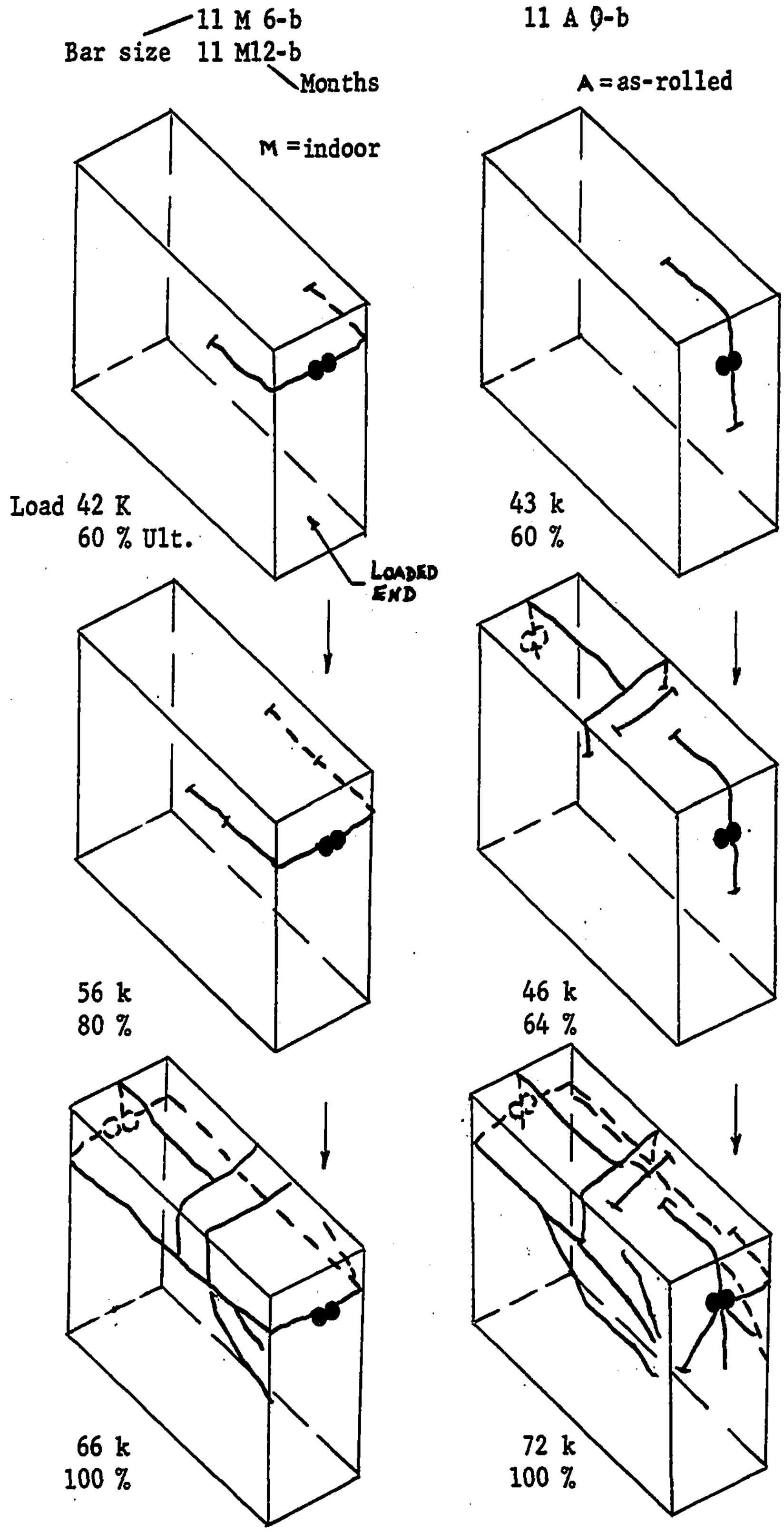


Fig. 5.34 Crack pattern for specimens with No. 11 double adjacent bars.

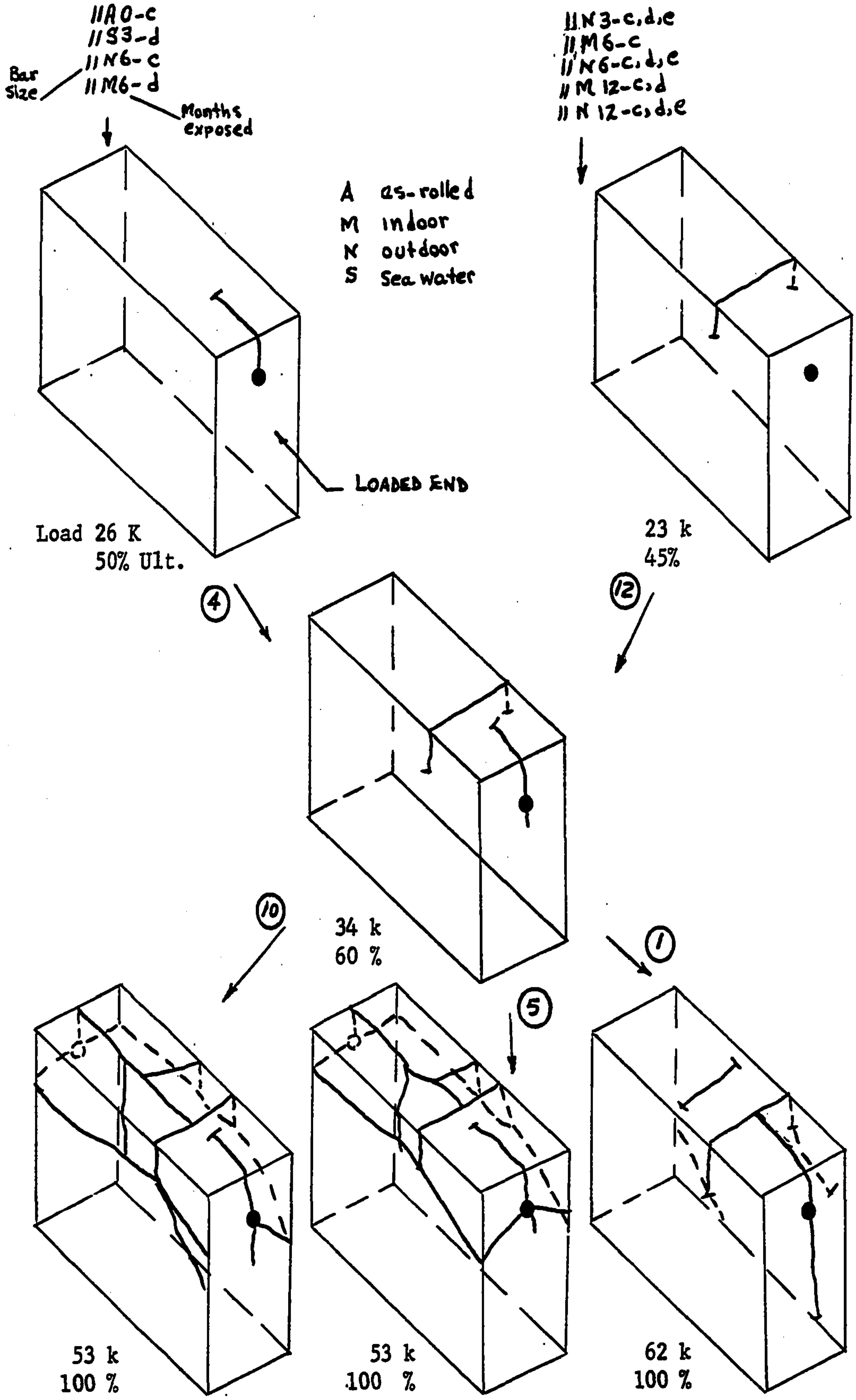


Fig. 5.35 Crack pattern for specimens with No. 11 single bar.

60 percent of the failure load of the specimens. The subsequent behavior of the specimens depended on the cracking pattern and on the manner and speed with which the cracks progressed. The cracking pattern of the single-bar specimens was somewhat different from those of the double-bar spaced or tied-adjacent specimens. The difference was in the order of appearence of the cracks, their locations and the final crack pattern. At the same time, some of the specimens tested with sea-water corroded reinforcing bars (single-bar and double-spaced bar specimens) failed by first the formation of a longitudinal splitting crack on the top of the specimens and then the occurence of a longitudinal crack on the sides at the level of the reinforcing bars. Eventually, a layer of concrete at the level of the bar was sheared off. The crack pattern of the specimens with web reinforcement was the same as the specimens without web reinforcement. The specimens without web reinforcement exploded at failure, whereas the specimens with web reinforcement popped open without explosion and the pieces of the specimen remained intact. (see Figs. A.1 and A.2 of Appendix A).

5.4 Corrosion

5.4.1 Quantity and Type of Crrosion

The corrosion of the reinforcing bars was removed by means of wire brushing and a chemical pickling process as described in Sec. 4.2.3.1. The quantity of corrosion and/or mill scale removed from each bar at the end of different exposure periods are tabulated in Table 4.4. The physical description of the bars after each period of exposure to different corrosive environments could bestbe described as follows:

(a) Three-Months Outdoor: A thin layer of corrosion partially covered the bar surface. There was more corrosion on bar deformations than in between the deformations. The corrosion had light brown color, it was soft and could be removed by hand rubbing. An insepection of the bar surface after the corrosion was removed revealed no pitting of the surface of the bars due to corrosion.

(b) Six-Months Outdoor: The bars were partially covered with a brown color rust, darker and firmer than the 3-months outdoor corrosion. It did not flake off by hand rubbing and there seemed to be more rust in between the deformations than the 3-months bars. There was no sign of pitting of the bar surface due to corrosion.

(c) Twelve-Months Outdoor: The corrosion on these bars had a reddish-brown color and could not be easily removed by hand. The corrosion was firmer and tighter than either of 3-or 6-months outdoor corrosion. No pitting of the bar surface was observed.

(d) Six-Months Indoor (Fresh Water): A dark brown, rough, non-uniform and corrugated, spot-like corrosion partially covered the bar surface. The rust came off by hand rubbing. In comparison with the 6-months outdoor bars, these bars had less corrosion on them. The bar surface was slightly pitted with corrosion.

(e) Twelve-Months Indoor (Fresh Water): The rust produced in this manner may be described as reddish-brown in color, rough and non-uniform in texture, and could not be easily removed by hand. The bars were slightly pitted due to rust.

(f) Three-Months Sea-Water: This was a light-brown crust of "loose", "flaky", soft corrosion covering the entire surface of each bar. It crumbled and came off loose by hand rubbing. Under the top light-brown layer of rust, there was a black layer of corrosion. After the corrosion was removed evidence of extensive pitting of bar surface was observed. (see Fig. A.5 of Appendix A).

In summary, the 3-,6-,and 12-months indoor and outdoor corrosion was nonuniform, partially covered the bars, and ranged in color from light-brown (early rust) to dark reddish-brown (late rust). There was but little pitting of bars due to corrosion. The 3-months sea-water rust was thick, covered the entire bar surface, and pitted the bars. Figs. A.4 and A.5 in Appendix A show the as-rolled, corroded and chemically cleaned reinforcing bars. Figs. 5.36 through 5.39 show the amount of corrosion and /or mill scale for various bar sizes.

5.4.2 Effect of Corrosion on Deformation Height, Length and Diameter of Deformed Bars

The influence and the seriousness of a given type and quantity of corrosion, created under three different environments, with respect to the bond characteristics of the deformed reinforcing bars was evaluated by means of the program of eccentric bond pullout specimens tested. However, the current ASTM standards impose some limitations on the effective amount of steel remaining after corrosion is removed, and set a minimum deformation height for modern deformed bars (Table 3.1). According to ASTM Specification, if the polished weight of the coupon becomes less than 94 percent of the theoretical weight, the bars have

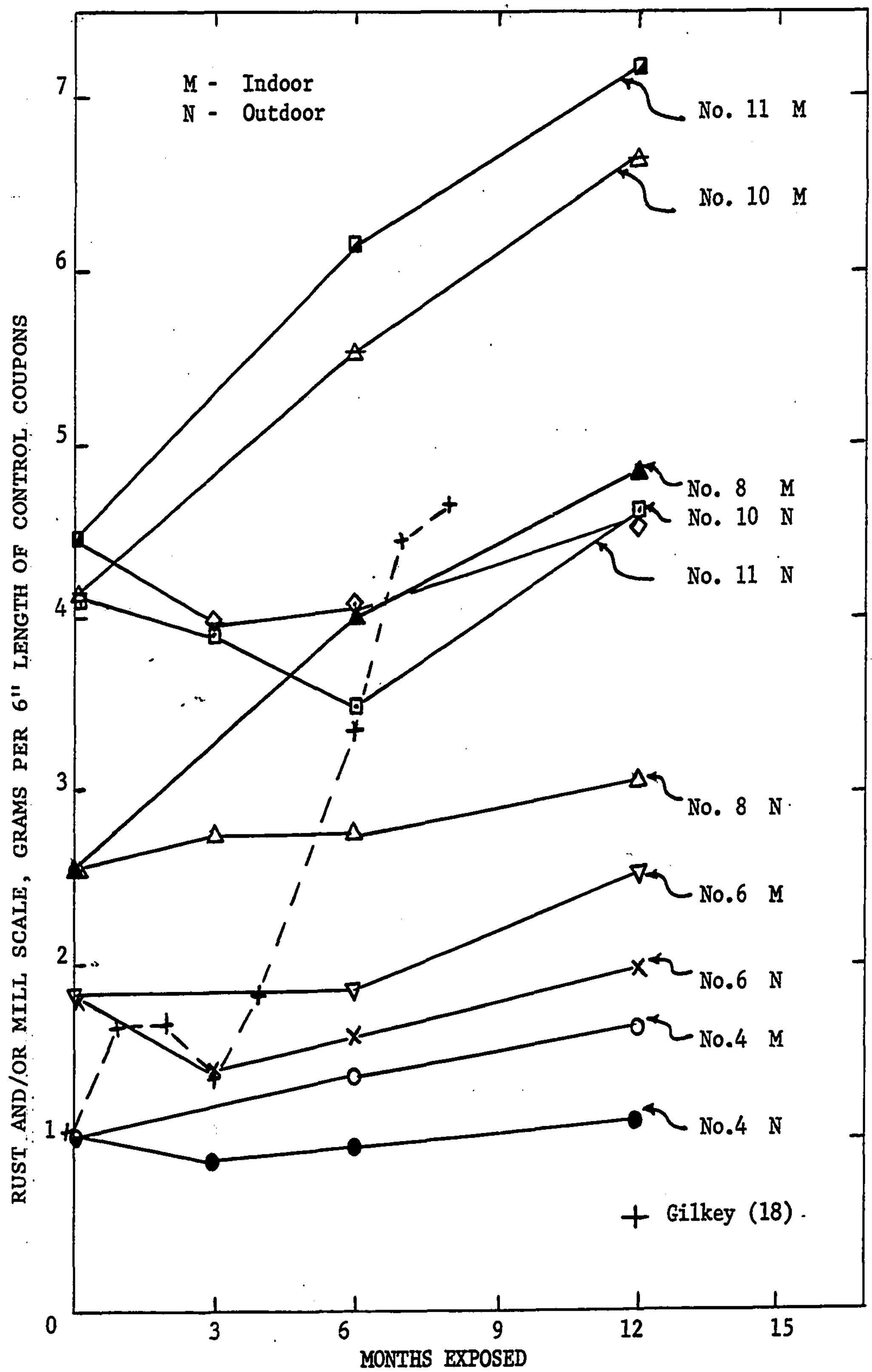


Fig. 5.36 Weight of corrosion for various exposure durations .

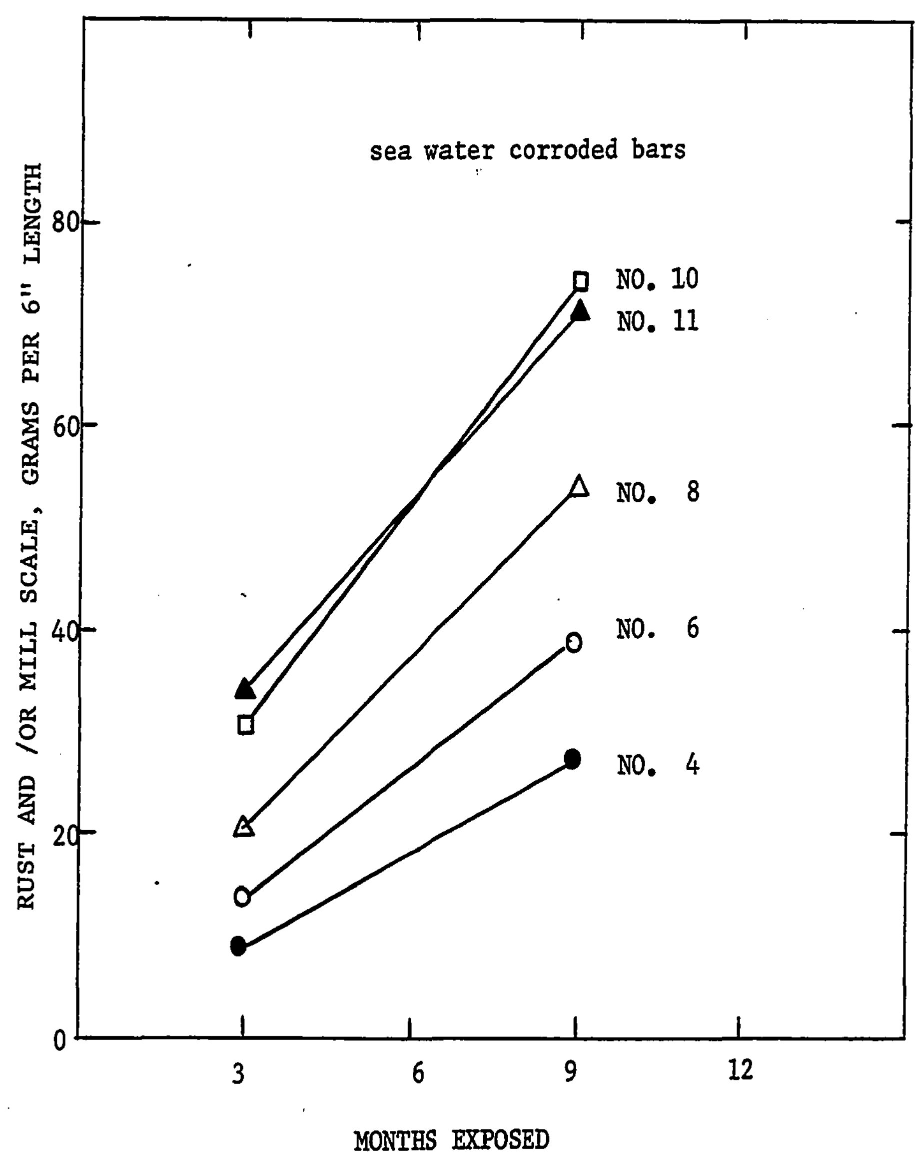


Fig. 5.37 Weight of corrosion versus months of exposure.

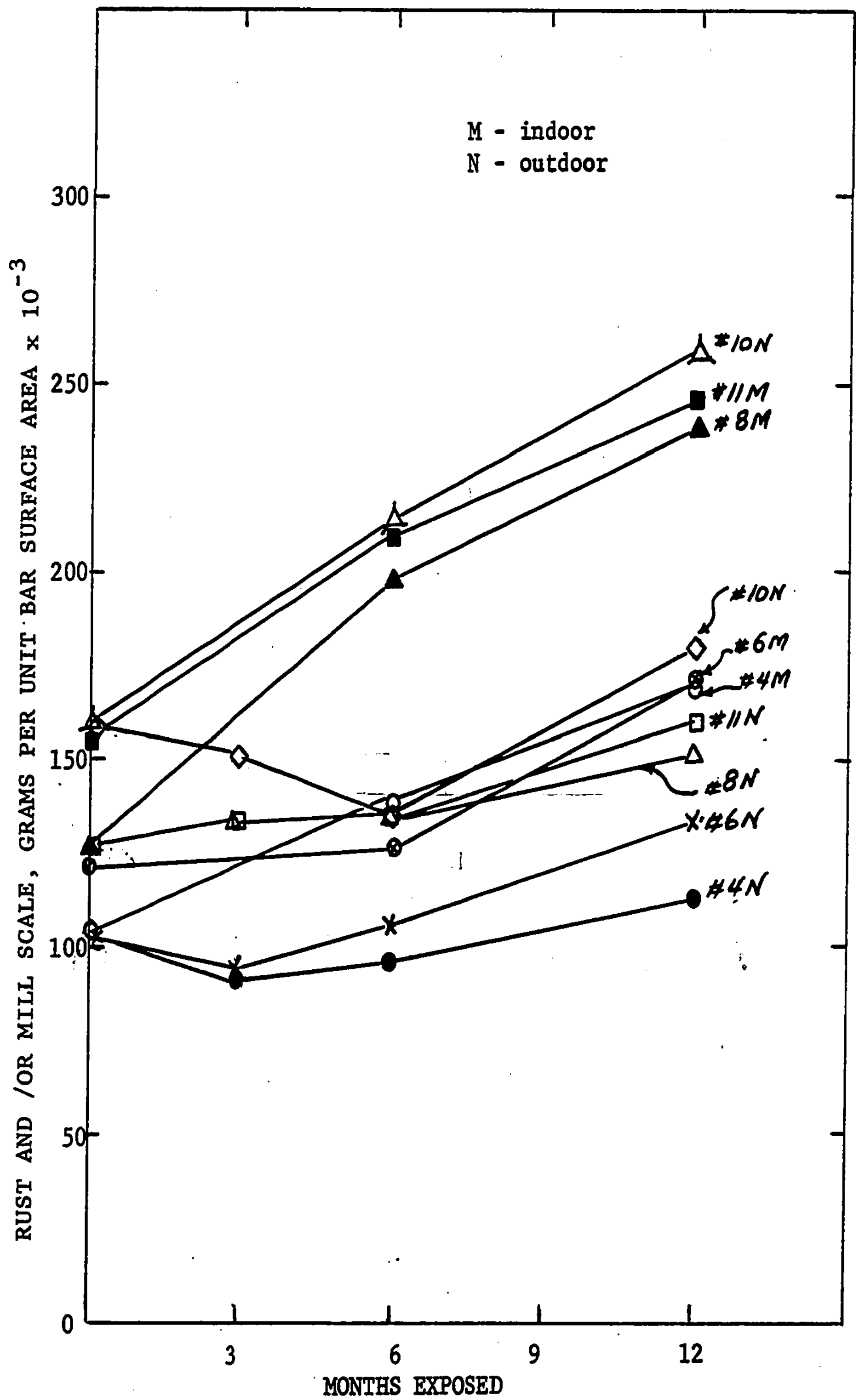


Fig. 5.38 Weight of corrosion per unit bar surface area for various corrosion environments & exposure durations.

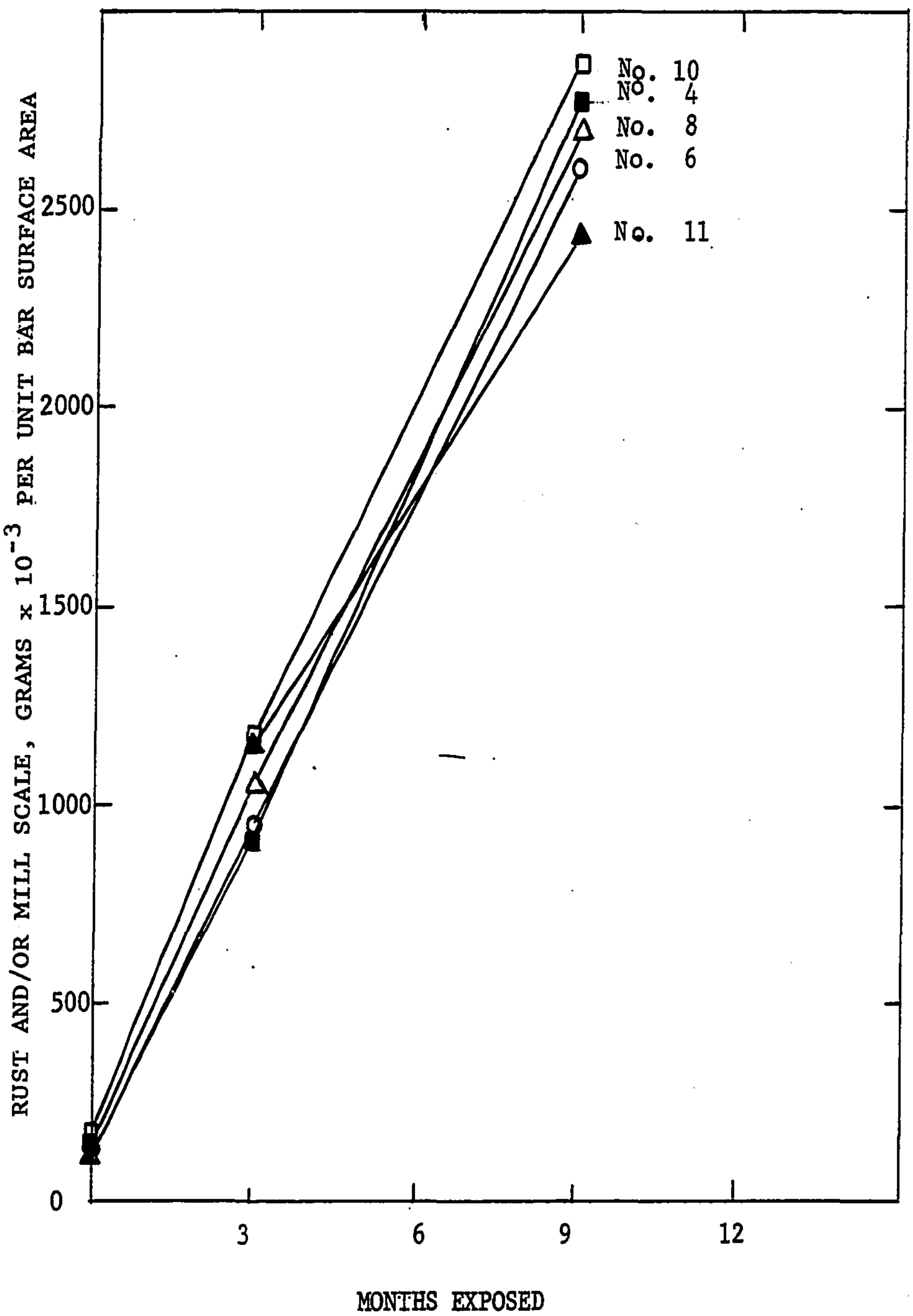


Fig. 5.39 Weight of corrosion per unit surface area of sea water corroded bars.

to be rejected regardless of their bond characteristics exhibited by the bond test. There was no significant change in the deformation height, bar length, bar diameter of the coupons exposed to sea-water (3 months), indoor and outdoor for periods of 3,6,and 12 months. There was, however, a slight reduction in final weights, but still much higher than the ASTM 94 percent. Table 5.1 is the tabulation of the initial and final dimensions of the bars corroded under sea -water for three months. These bars indicated a ratio of final weight to the theorectical weight of 94.7 to 97.7 for bar size ranging from No. 4 to No. 11.

5.4.3 Mill Scäle

The mill scale was chemically and mechanically removed from 86 control coupons of each bar size (Sec. 4.2.3.1). The control coupons were weighed before and after the removal of mill scale. Also the bar diameter, bar length and the deformation height were measured. These measurements are tabulated in Table A.2 of Appendix A. The mill scale was dark grey and represented an average of 0.5 percent of the initial weight of the coupons (6 inches long).

5.5 Tensile Strength of As-Rolled and Corroded Reinforcing Bars

One of the objectives of this study was to determine how far corrosion could progress, and by how much different degrees of corrosion would reduce the cross-sectional area of the bars, thereby reducing their ultimate strengths. In order to achieve this objective, 10 as-rolled bars of each of the bar sizes Nos. 4,6,8,10, and 11 were

TABLE 5.1

A COMPARISON OF THE INITIAL & FINAL BAR DIMENSIONS & WEIGHTS (3 MONTHS SEA WATER)

	Bar size				
	#4	#6	#8	#10	#11
D_f , inch	0.479	0.701	0.946	1.181	1.351
D_i , inch	0.487	0.708	0.960	1.190	1.362
L_f , inch	6.077	5.963	5.950	5.889	5.917
L_i , inch	6.001	5.961	5.958	5.954	5.946
H_f , inch	0.026	0.046	0.068	0.084	0.073
H_i , inch	0.026	0.048	0.067	0.087	0.072
W_f , grams	150.34	322.14	576.16	898.90	1143.37
$w_f = W_f / L_f$ lbs./in.	0.054	0.119	0.213	0.336	0.426
w_t lbs./ in.	0.057	0.125	0.224	0.348	0.436
$(w_f / w_t) \times 100$ %	94.7	95.2	95.1	96.5	97.7

$D_{f,i}$ final or initial diameter measured between the lugs.
$L_{f,i}$ & $H_{f,i}$ final and initial coupon length and lug height
W_f weight of rust and /or mill scale
w_t theoretical weight of steel = (0.2833 lb./ in.^{3})x A_s

tested in tension following the procedure described in Sec. 4.2.3.2. Also, as each eccentric bond pullout specimen was tested to failure, the corroded reinforcing bar was removed from the specimen and was tested in tension. The pullout specimens were designed to fail at steel stresses within the working stress. Table A.3 in Appendix A shows that, with the exception of No. 4 bars which yielded in the concrete, the steel stresses in the remainder of the bars were below their yield points (Table 3.1). The yield stress and the ultimate tensile stress of the as-rolled bars as well as the corroded bars are reported in Table 5.2. As it is seen from this table, the outdoor and indoor corrosions for periods of exposure of 3,6,and 12 months did not have much effect on the ultimate strength of the bars, while there was an average of 3.7 percent reduction in the ultimate strength of the bars corroded under sea-water.

Figures 5.40 through 5.45 show the stress-strain curves of as-rolled and corroded bars.

TABLE 5.2

COMPARATIVE STRENGTH OF CORRODED BARS

Type & Months Exposed	Steel Stress, f_s (ksi)		Average Stress (ksi)		f_s(ult.) / f_A(ult.)	Coeff. of Var. V %	
	y.p.	ult.	y.p.	ult.		y.p.	ult.
(No. 6.Bars As-rolled stress f_A : y.p.= 48.2, Ult.=80.2 ksi)							
N3	47.7	80.0					
"	46.6	80.7	46.4	80.4	1.002	1.6	0.3
"	46.4	80.4					
M6	50.4	83.2					
"	50.0	76.6					
"	48.9	82.3	49.5	80.4	1.002	1.5	3.1
"	49.1	82.7					
"	50.7	77.3					
"	48.9	80.9					
N6	48.4	82.5					
"	49.5	83.2	48.9	82.7	1.031	0.9	0.4
"	48.9	82.5					
M12	48.6	80.9					
"	47.3	81.1					
"	48.9	81.6	48.6	81.8	1.019	1.6	0.9
"	49.8	82.9					
"	48.4	82.5					
N12	46.4	74.3					
"	48.4	82.9	48.2	79.8	0.995	2.9	4.9
"	49.8	82.3					
S3	46.4	76.8					
(No. 8 bars As-rolled stress f_A: y.p.= 52.0, Ult.= 83.2 ksi)							
N3	50.6	81.4					
"	52.5	82.7	51.5	82.0	0.985	1.8	0.8
M3	51.4	79.7			0.958		
N6	51.6	83.8					
"	51.9	83.5	52.4	83.5	1.003	1.8	0.2
"	53.7	83.4					

N = outdoor, M= indoor, S= sea water f_A= as-rolled stress

TABLE 5.2 (Con't)

Type & Months Exposed	Steel Stress, f_s (ksi) y.p.	Steel Stress, f_s (ksi) ult.	Average Stress (ksi) y.p.	Average Stress (ksi) ult.	$\frac{f_s(ult.)}{f_A(ult.)}$	Coeff. of Var. V % y.p.	Coeff. of Var. V % ult.
M6	51.1	83.3					
"	52.9	84.0					
"	52.4	83.5	53.3	83.8	1.007	3.6	0.5
"	51.4	83.8					
"	55.9	84.5					
"	55.9	84.1					
M12	52.9	84.1					
"	53.7	84.3					
"	52.1	83.6	53.0	84.0	1.009	1.2	0.4
"	52.3	83.4					
"	53.5	84.3					
S3	51.1	80.2					
"	49.4	81.0	50.2	80.1	0.963	2.3	0.8
"	48.8	79.2					
"	51.6	80.0					
(No. 10 bars As-rolled stress f_A : y.p.= 47.4, ult.= 75.8 ksi)							
N3	44.0	74.0					
"	47.1	75.6	45.5	74.8	0.987	1.1	1.1
M6	48.0	68.4					
"	47.8	76.8					
"	48.4	77.0	47.9	75.2	0.992	1.3	4.7
"	48.1	76.4					
"	46.6	79.7					
"	48.6	77.0					
N6	44.8	74.8					
"	48.4	77.0	46.4	75.6	0.997	1.0	1.3
"	46.0	75.0					
M12	48.2	76.9					
"	43.5	75.7					
"	49.1	76.8	47.5	76.2	1.005	3.8	0.8
"	48.3	76.6					
"	48.1	75.3					
"	47.9	76.3					
N12	46.3	75.3					

N = outdoor, M= indoor, S= sea water, f_A = as-rolled stress

TABLE 5.2 (con't)

Type & Months Exposed	Steel Stress, f_s (ksi) y.p.	Steel Stress, f_s (ksi) ult.	Average Stress (ksi) y.p.	Average Stress (ksi) ult.	$\frac{f_s(\text{ult.})}{f_a(\text{ult.})}$	Coeff. of Var. V % y.p.	Coeff. of Var. V % ult.
S3	44.7	73.5					
"	42.3	71.0					
"	45.5	74.0	44.4	72.9	0.961	2.7	1.6
"	45.2	73.3					
(No. 11 bars As-rolled stress f_a: y.p. 48.1, ult.= 80.2 ksi)							
N3	49.4	81.4					
M6	47.8	81.1					
"	48.4	80.2					
"	49.3	81.0	48.9	81.1	1.011	1.4	0.8
"	49.0	82.2					
"	49.7	81.3					
N6	47.3	79.2					
M12	49.3	80.5					
"	46.9	78.9	47.6	79.7	0.994	2.6	0.8
"	46.6	79.7					
N12	46.7	78.7					
S3	47.0	78.0					
"	49.0	79.2	47.0	77.9	0.971	3.3	1.3
"	45.2	76.7					

N = outdoor, M = indoor, S= sea water

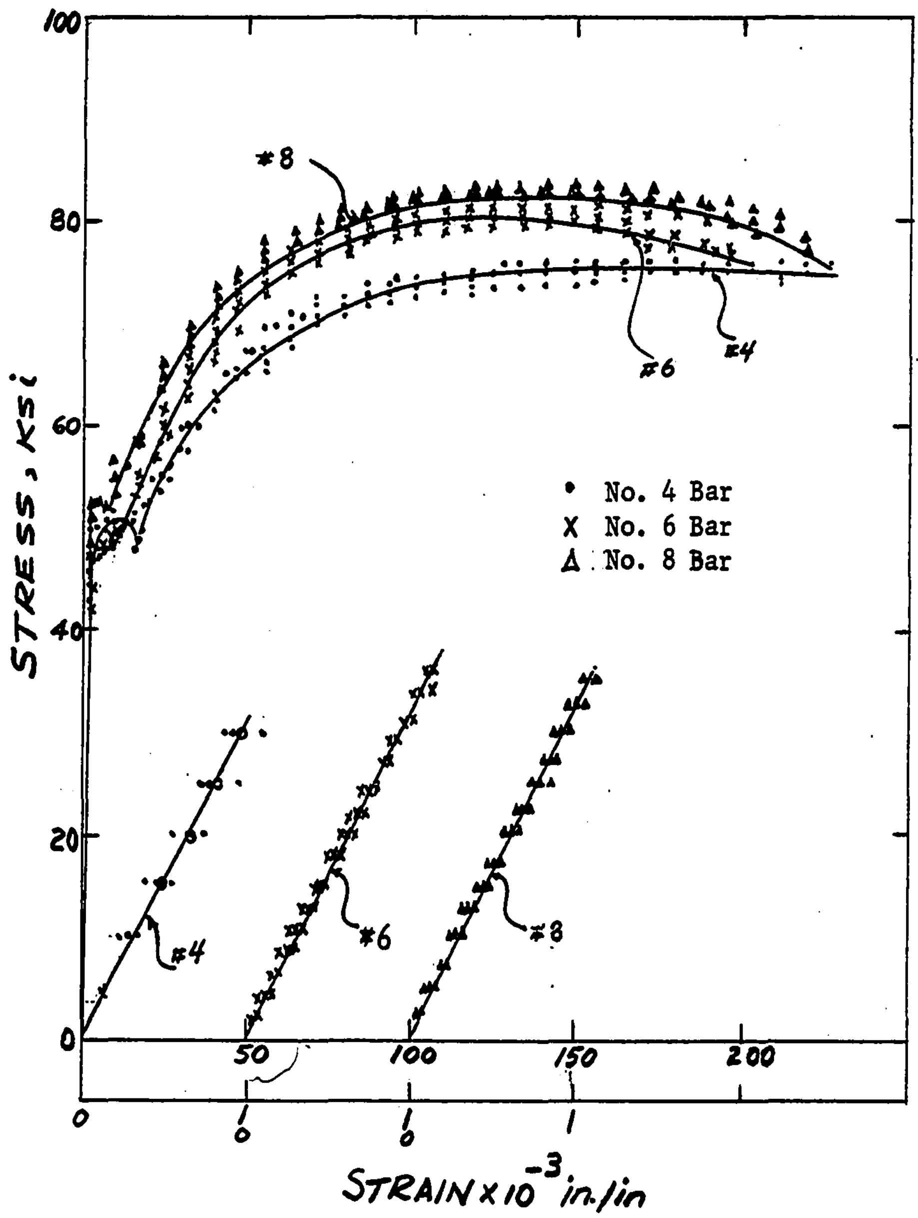


Fig. 5.40 Stress-strain curves for as-rolled

No. 4, 6 , & 8 bars.

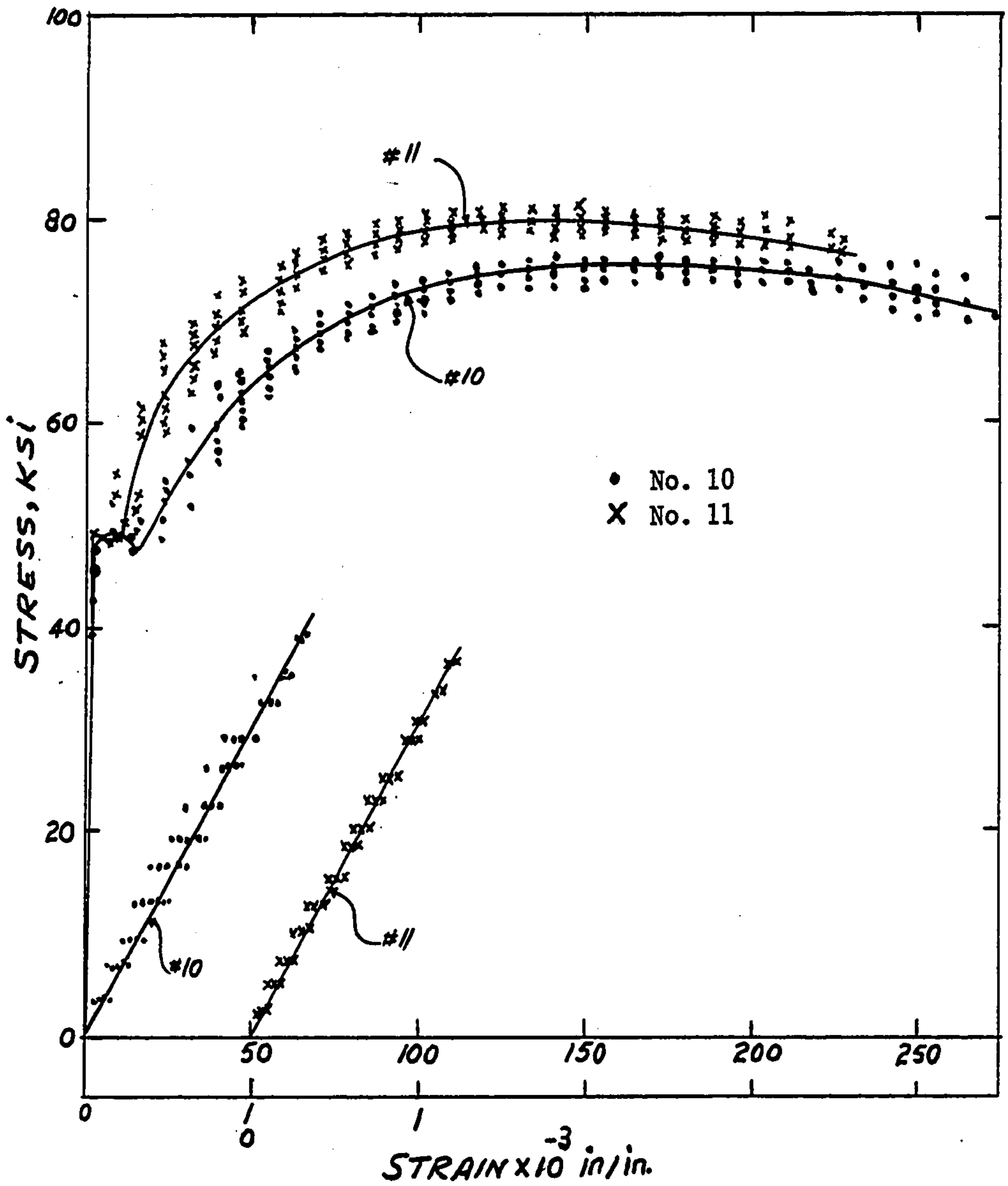


Fig. 5.41 Stress-strain curves, as-rolled No. 10 & 11 bars.

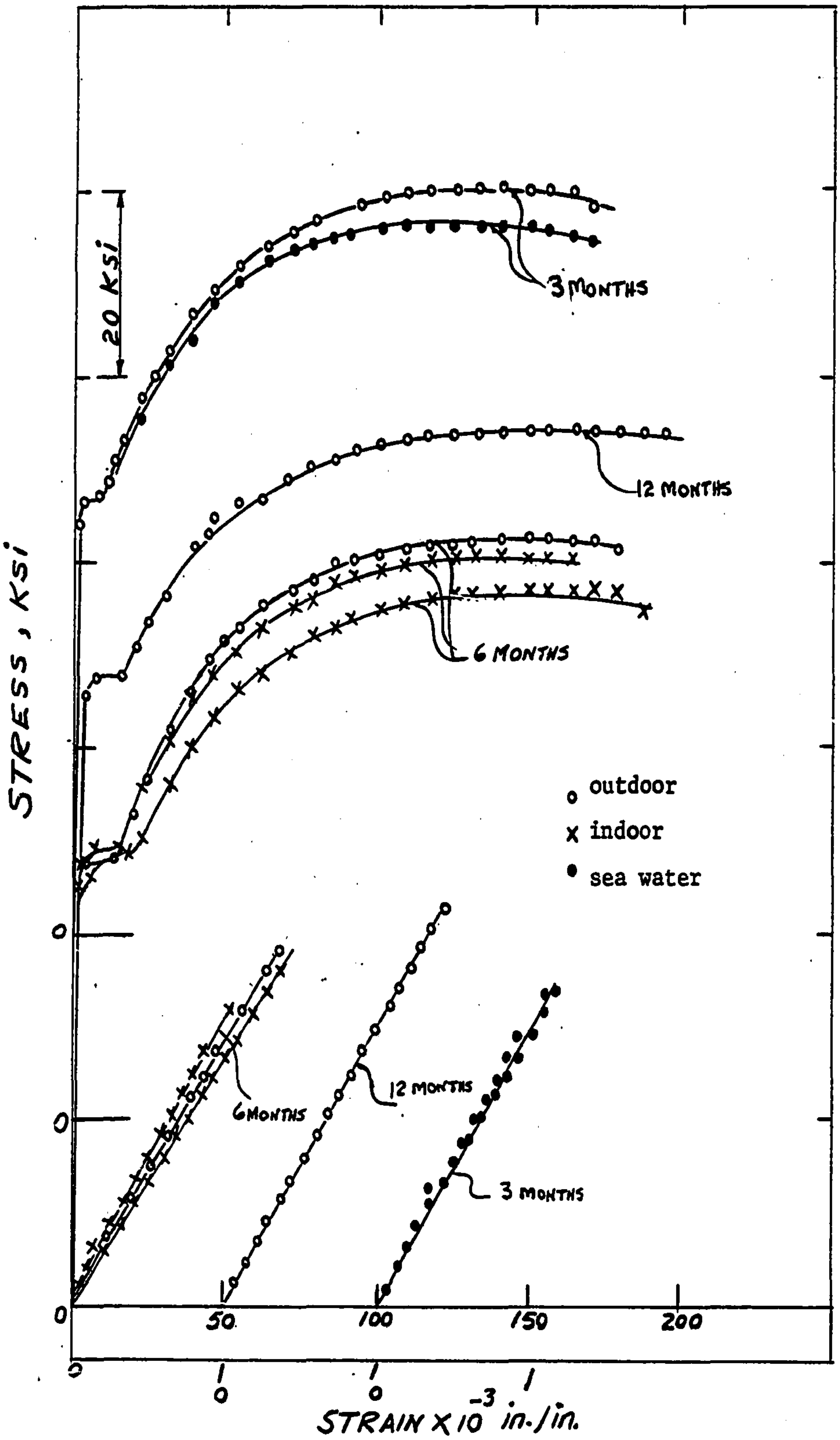


Fig. 5.42 Stress-strain curves for No. 6 corroded bars.

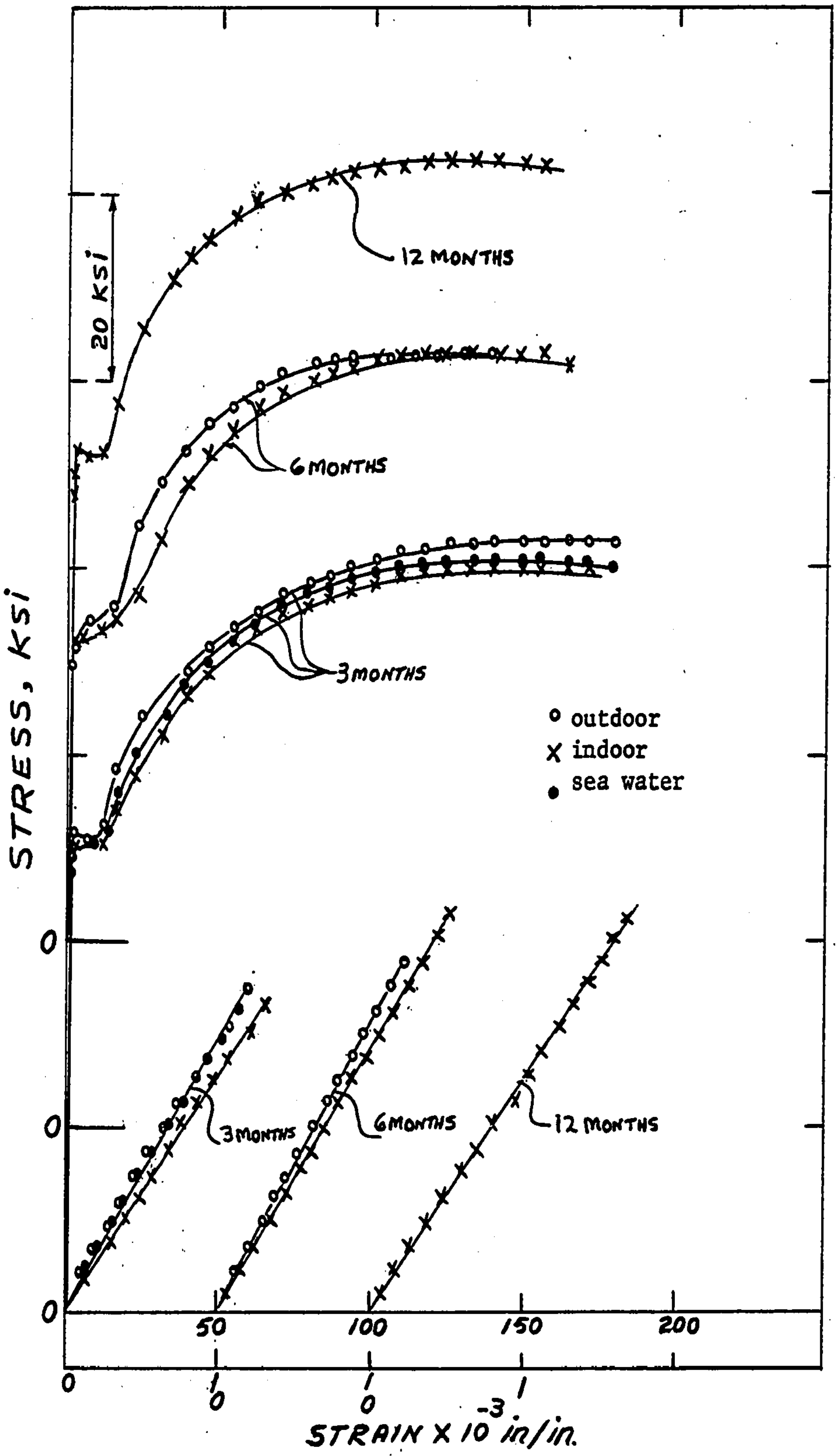


Fig. 5.43 Stress-strain curves for No. 8 corroded bars.

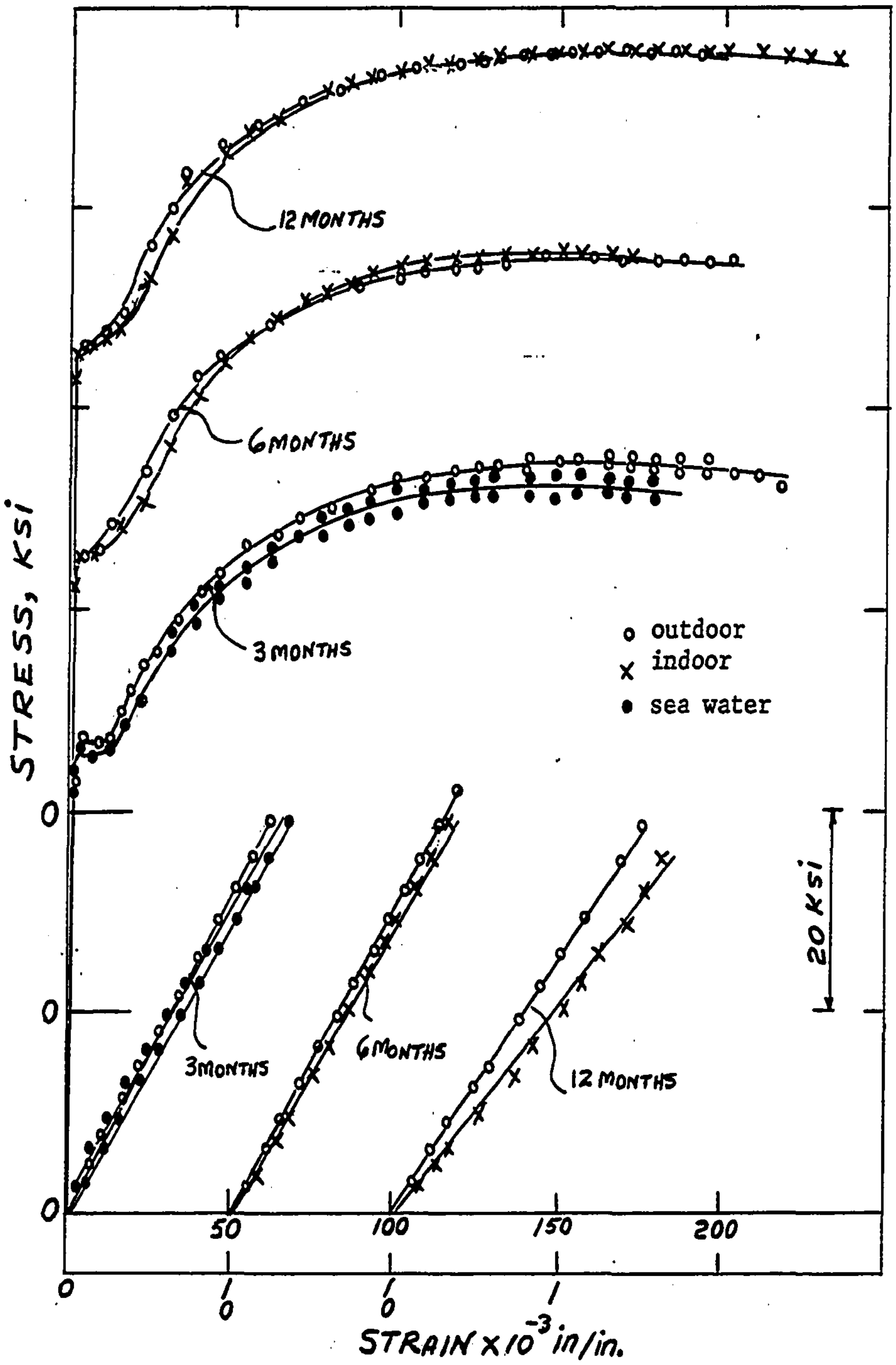


Fig. 5.44 Stress-strain curves for No. 10 corroded bars.

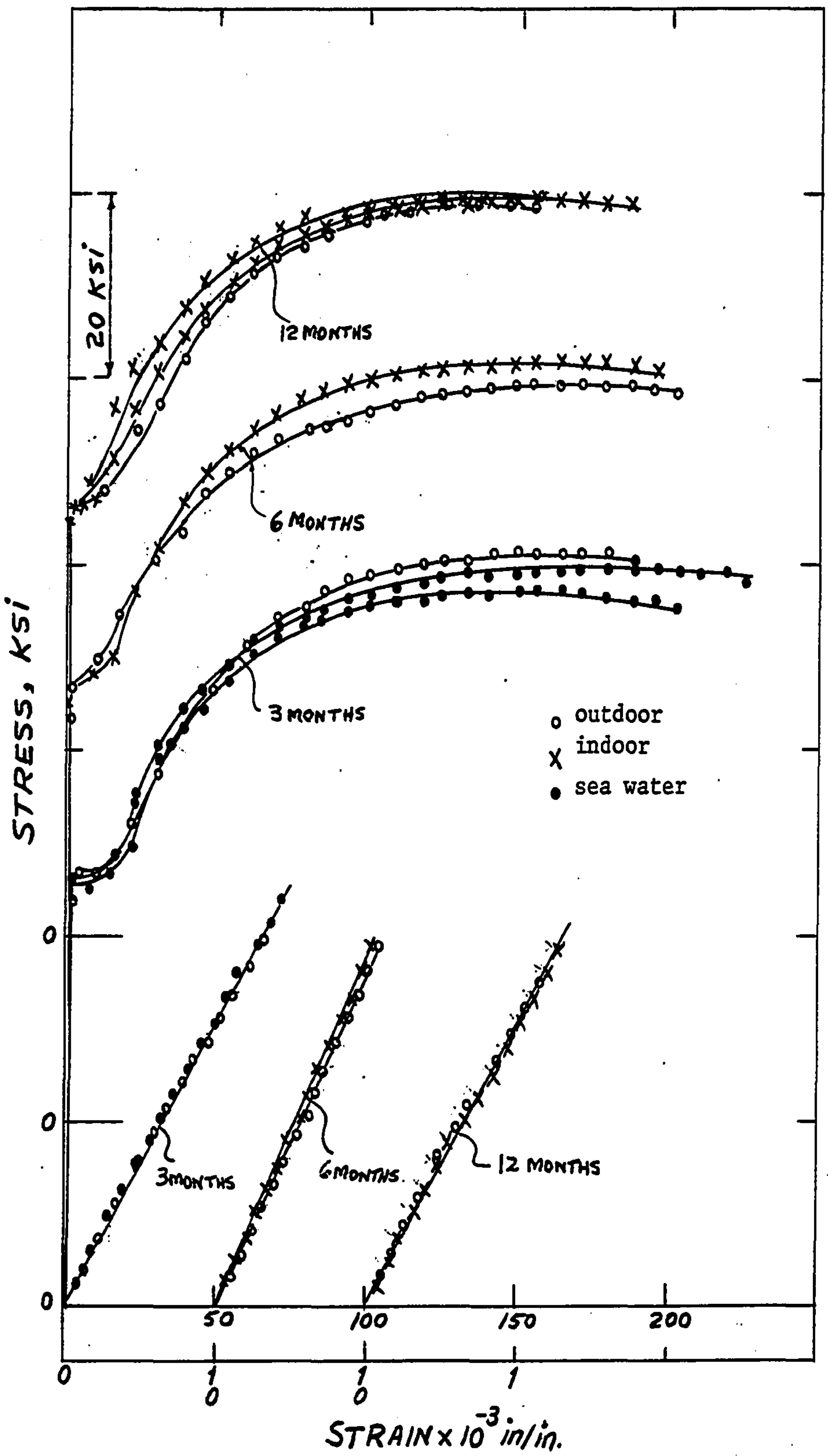


Fig. 5.45 Stress-strain curves for No. 11 corroded bars.

CHAPTER VI

DISCUSSION OF RESULTS

6.1 Introduction

The results of tests on 115 eccentric bond pullout specimens were reported in the previous chapter. In the following sections, the response of these specimens to the applied loads will be discussed in terms of bond stress versus the following variables:

1. Magnitude and degrees of surface corrosion produced under various corrosive environments,

2. Bar spacing and bar size,

3. Loaded-end and free-end slips,

4. Limiting shear stresses, and

5. Splitting behavior of the specimens.

The influence of web reinforcement on bond strength and the subsequent behavior of the eccentric pullout specimens is examined.

Since, the load carrying capacity and the mode of failure of the specimens depended on the cracking pattern, attention was focused on formation and the progression of cracks in the specimens. An attempt is made to determine the mechanism of failure of the specimens and to establish the influence of various degrees of corrosion and bar spacing on these mechanisms.

6.2 Bond Stress Versus Surface Corrosion

The correlation between bond properties of reinforcing bars and the various surface conditions studied were examined in three methods. First, Figs. 5.1 through 5.20 were plotted showing the corrected bond stress for a nominal concrete compressive strength of 3,000 psi versus the loaded-end slip for various bar sizes, bar spacings, and different reinforcing bar surface conditions. Three corrosive environments were studied: (a) outdoor, weather-rusted, (b) indoor, sprayed with tap water, (c) indoor, sprayed with simulated sea-water. Because of the numerous variables involved, it is difficult to deduce from these plots an over-all and specific correlation between bond stress and bar surface conditions. However, the plotted data of the above figures do seem to indicated the followings:

1. The initial portions of the bond stress loaded-end slip curves approach a straight line. But, generally, the curves beyond the linear portion are nonlinear and the rate of change of slip is higher than bond stress. In other words, slip was small for low loads, but as steel stress progressed toward the end of the bar, bond between reinforcing bar and concrete was broken over a longer portion of the bar and thus higher loaded-end slips were indicated.

2. The ultimate bond stress (at failure) developed by all of the five bar sizes exposed for 3-months outdoors and cast singly in eccentric pullout specimens was somewhat less than that indicated by the unexposed bars (as-rolled). The reduction in bond stress for No.4 and No. 6 bars was an average of eight percent and for Nos. 8,10, and 11 an average of 25 percent. The similar specimens tested with

6-and 12-months-outdoor corroded bars, as compared with as-rolled bars, offered an average of 20 percent loss in load carrying capacity for No. 4 and No. 11 bars and only an average loss of five percent for Nos. 6,8, and 10 bars. All the bars with 6-months-outdoor surface corrosion, with the exception of No. 4 bars, showed an average superiority of 20 percent in bond strength over the three months outdoor corroded bars. There was no substantial difference in bond stress developed by bars with 6-months-outdoor rust and the ones with 12-months-outdoor corrosion.

Therefore, it could be concluded that early state corrosion, such as the one found on the surface of the bars exposed to weather outdoor for a period of three months, would lower the bond resistance of bars larger than No. 6 bar. However, reinforcing bars corroded outdoor for a period of 6 months improved the load carrying capacity of the specimens by 20 percent. These conclusions should be viewed within the experimental limitations of this investigation, because there were only three eccentric pullout specimens of a kind per exposure period tested and one specimen with as-rolled bars.

3. A comparison of the ultimate bond stress developed by single-bar specimens with 6-months-and 12-months-indoor corroded bars indicated very small change in bond stress of the longer exposed bars. However, with the exception of nine percent increase in bond resistance of No.6 bars, there was an average loss of 10 percent in bond strength of bars with 6-and 12-months surface corrosion as compared with as-rolled bars. No substantial change in bond resistance was observed for outdoor and indoor corrosive environments.

4. For the reinforcing bars exposed to sea-water for three months, a reduction of 30 percent in bond stress was observed for single-bar specimens with No.4 and No. 11 bars, while for Nos. 6,8,and 10 bars an average increase of eight percent was indicated over that of as-rolled bars. As compared to the bars exposed outdoor for three months, Nos. 6,8, and 10 sea-water-corroded bars offered an average of 10 percent excess in bond resistance while No. 4 bars lost 25 percent and No. 11 bars gained three percent.

5. No trend was indicated for double-spaced bar and double-adjacent-tied bar specimens containing 6-and 12-months-indoor-rusted bars. However, the 3-months sea-water-corroded bars cast in double-spaced-bar specimens failed at higher loads than the unexposed bars. The excess in bond stress was not the same for all the bar sizes. The Nos. 4,10, and 11 bars offered bond stresses 34 percent higher, No. 8 bars 80 percent higher, and No. 6 bars five percent higher than the bond stresses developed in the similarly tested specimens with as-rolled bars.

In summary, the corrosive environments outdoor and indoor have similar effects on bond resistance of reinforcing bars. Bars exposed to sea water for 3 months showed higher bond stress for both single-bar and double-bar specimens. Early corrosion (3-months-outdoor) seemed to lower the load carrying capacity of the bars by as much as 25 percent for bars larger than No. 8 bar, while an additional time of exposure of three months improved the bond strength by 18 percent over the three months.

Figs. 6.1 and 6.2 show the ultimate bond stress as a percentage

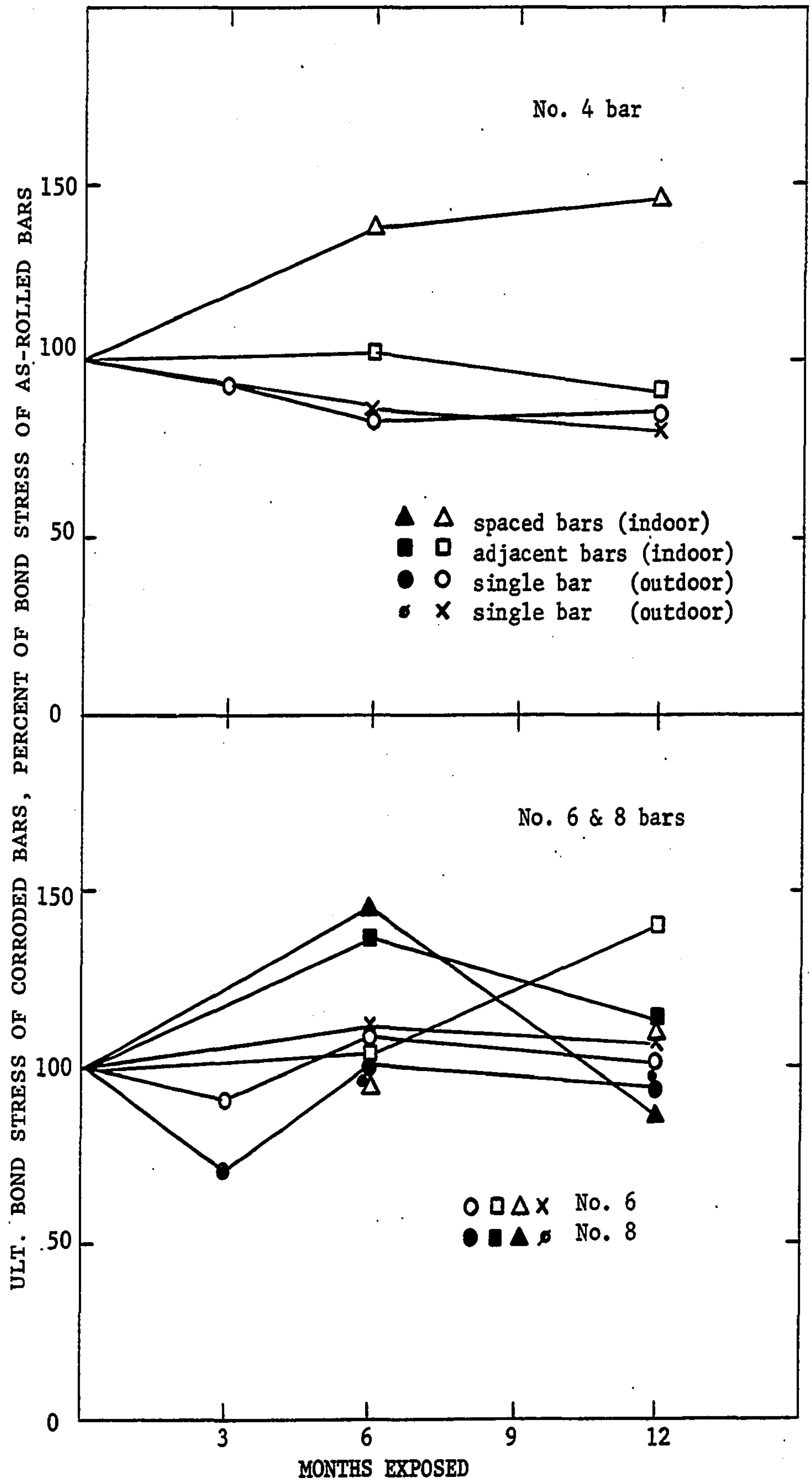


Fig. 6.1 Effect of corrosion on ultimate bond stress.

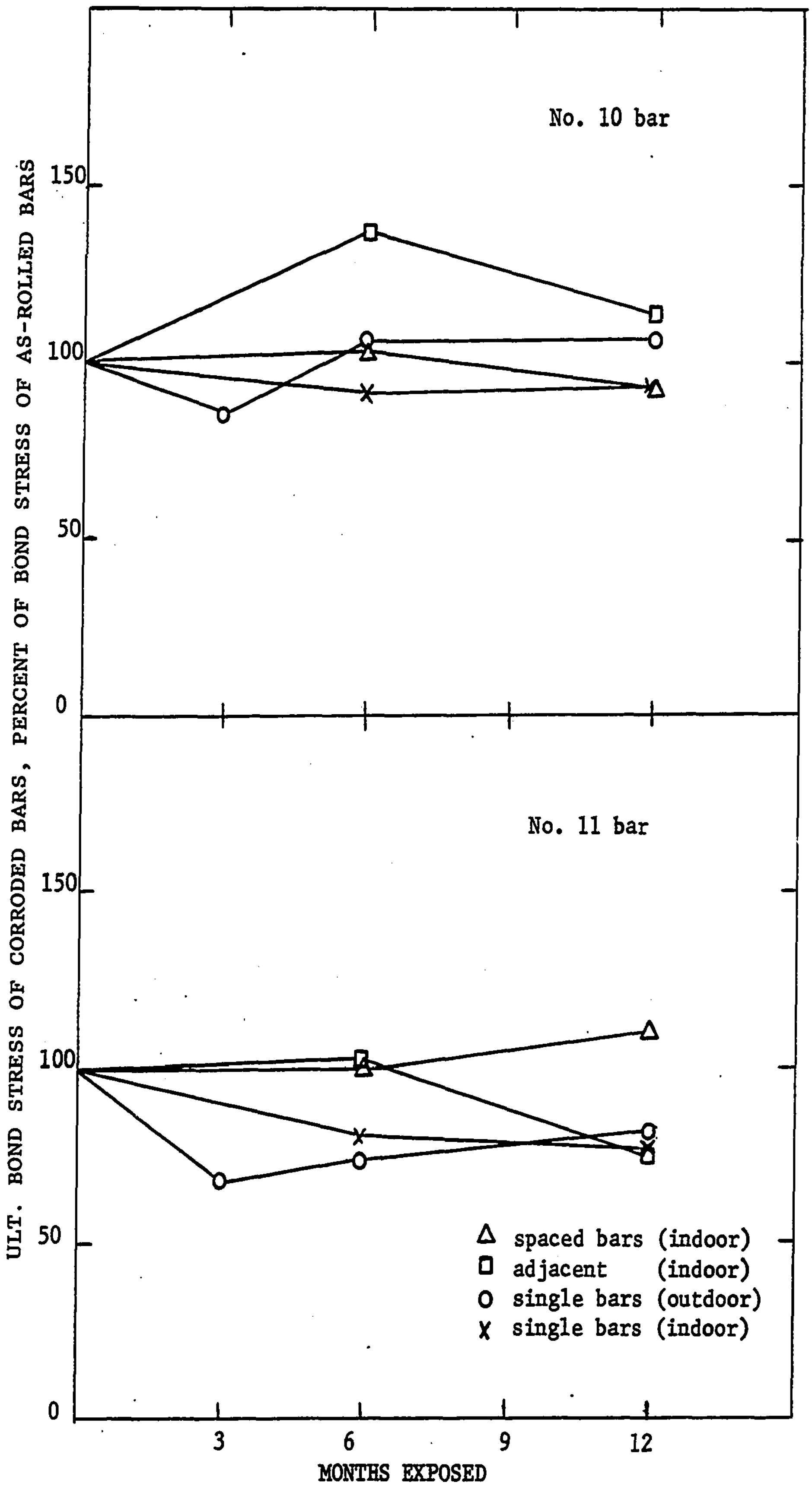


Fig. 6.2 Effect of corrosion on ultimate bond stress.

of bond stress of as-rolled bars plotted versus months of exposure to different corrosive environments.

The second method of correlation employed in the analysis of bond properties of reinforcing bars as influenced by different surface corrosions was the construction of plots showing the corrected bond stress at five arbitrary loaded-end slips versus the time of exposure. The loaded-end slip levels were 0.1×10^{-3}, 0.5×10^{-3}, 1.0×10^{-3}, 2.0×10^{-3} inch, and the ultimate slip at failure. These plots are shown in Figures 6.3 through 6.7. Although not on modern deformed reinforcing bars, the results of findings of Johnston and Cox (19) for No. 4 and No. 6 bars with transverse deformations are superimposed on Figures 6.3 and 6.4. As it is seen from these plots, the trends seem to be obscured within an experimental fluctuation.

The third method was an attempt to eliminate the above variations in results. The "average" bond stress values for each specimen were determined by totaling the bond stresses at above mentioned loaded end slip levels and dividing the sum by the number of slip levels. Figures 6.8 and 6.9 show the results. If the indication of No. 4 and No. 6 bars is disregarded, there seems to be a gradual and fairly well-defined downward trend for "average" bond stress of single-bar specimens with Nos. 8,10, and 11 bars with increase in time of exposure.

6.3 Effects of Corrosion on Reinforcing Bars

One of the objectives of this investigation was the evaluation of effects of corrosion on ultimate strength, height of deformations and the diameter of reinforcing bars after exposure to various corrosive environments for different periods of time. As it was discussed

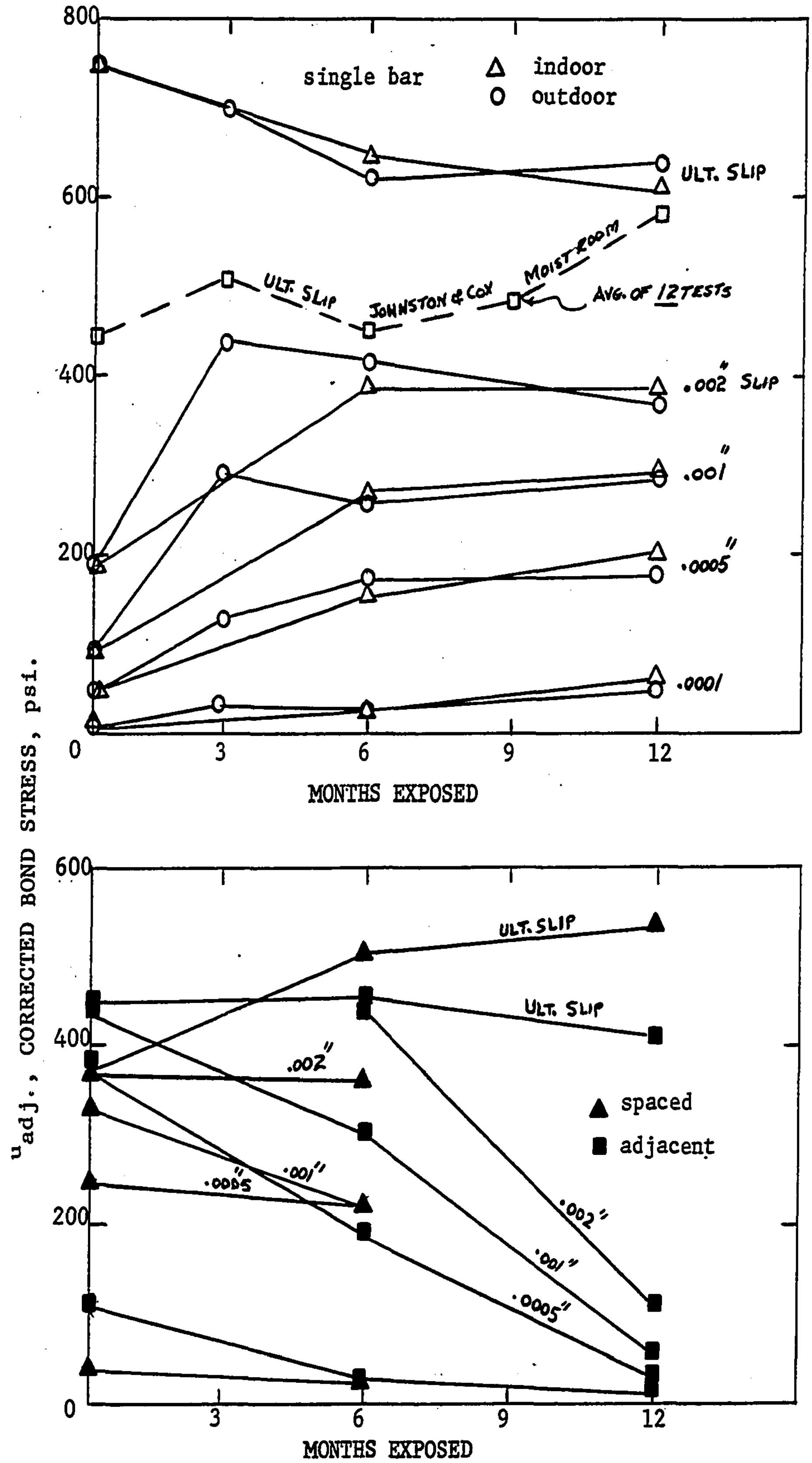
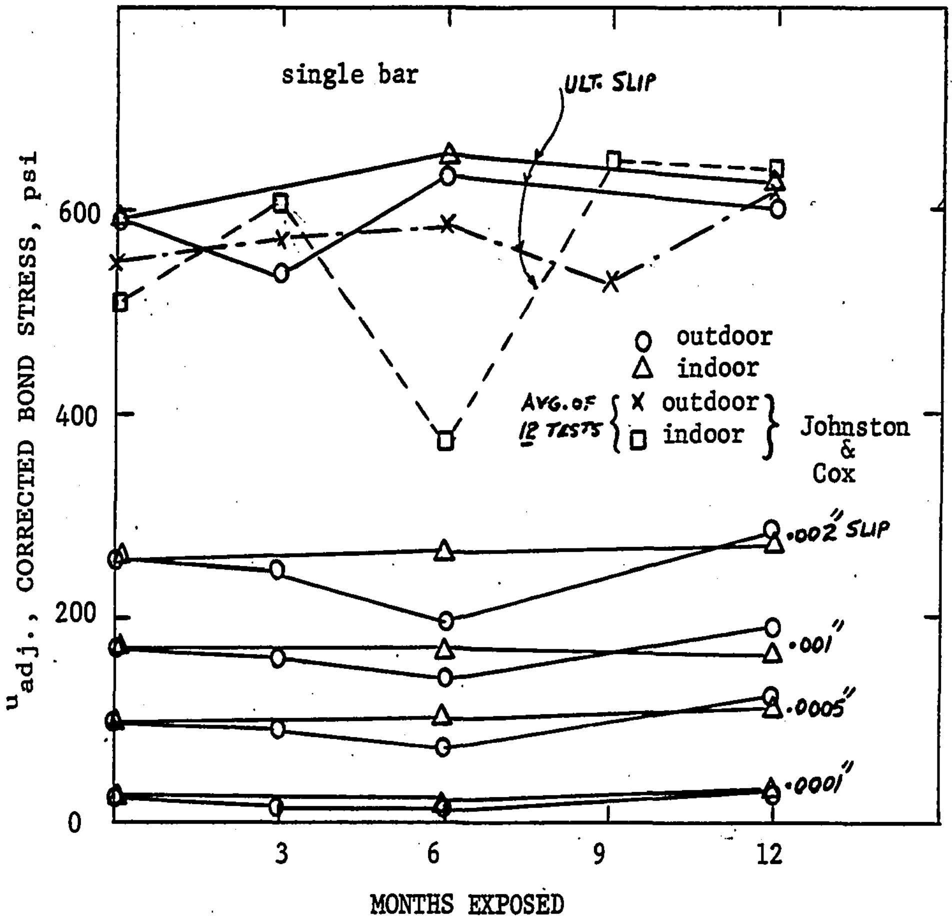


Fig. 6.3 Effect of corrosion on bond strength of No. 4 bars.



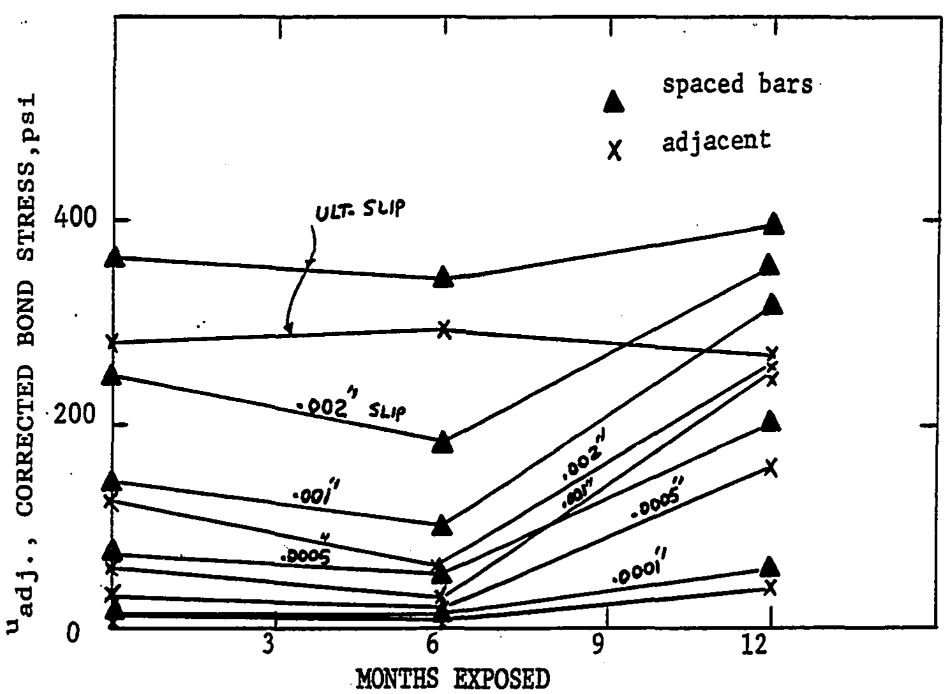
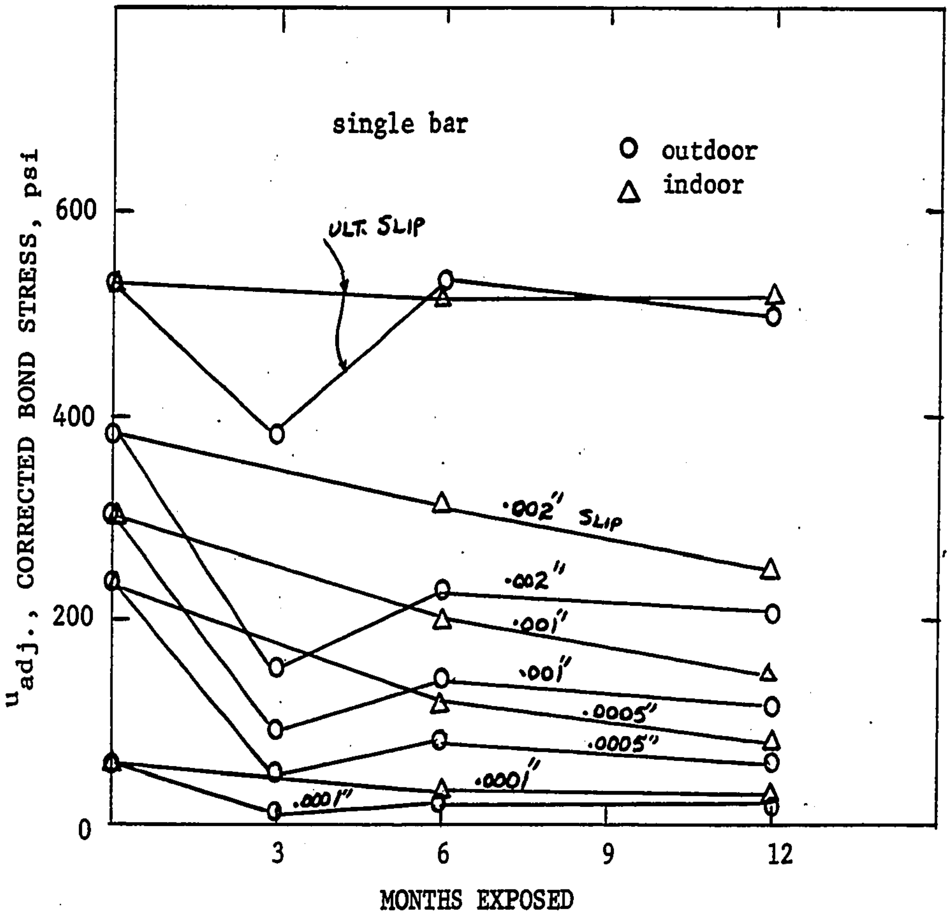


Fig. 6.4 Effect of corrosion on bond strength of No. 6 bars.



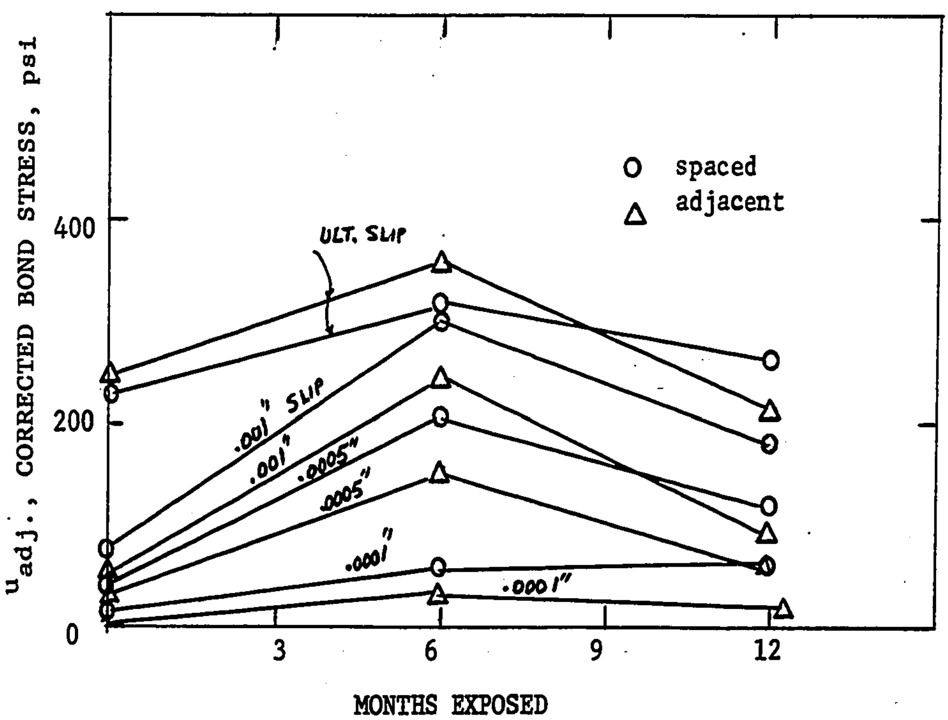


Fig. 6.5 Effect of corrosion on bond strength of No. 8 bars.

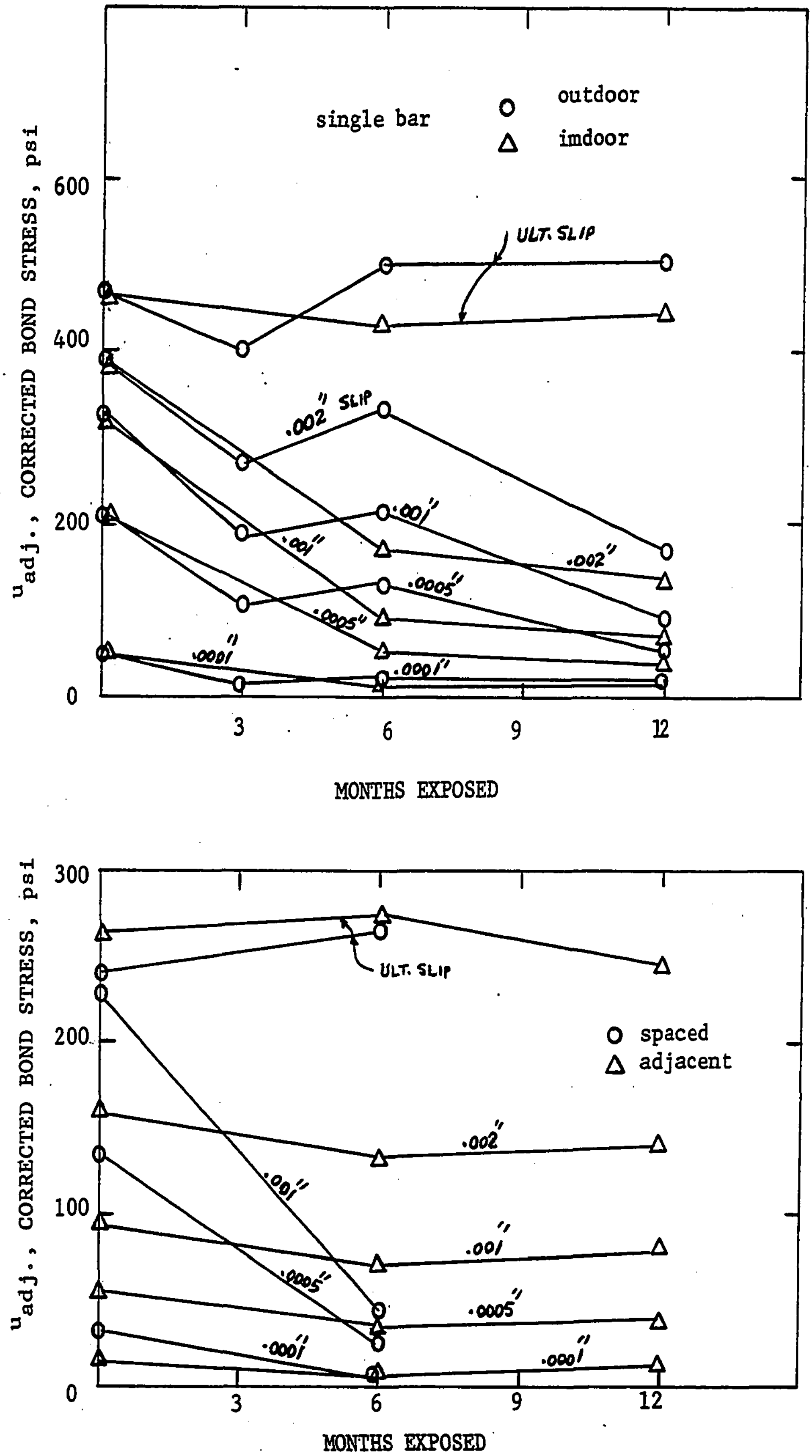
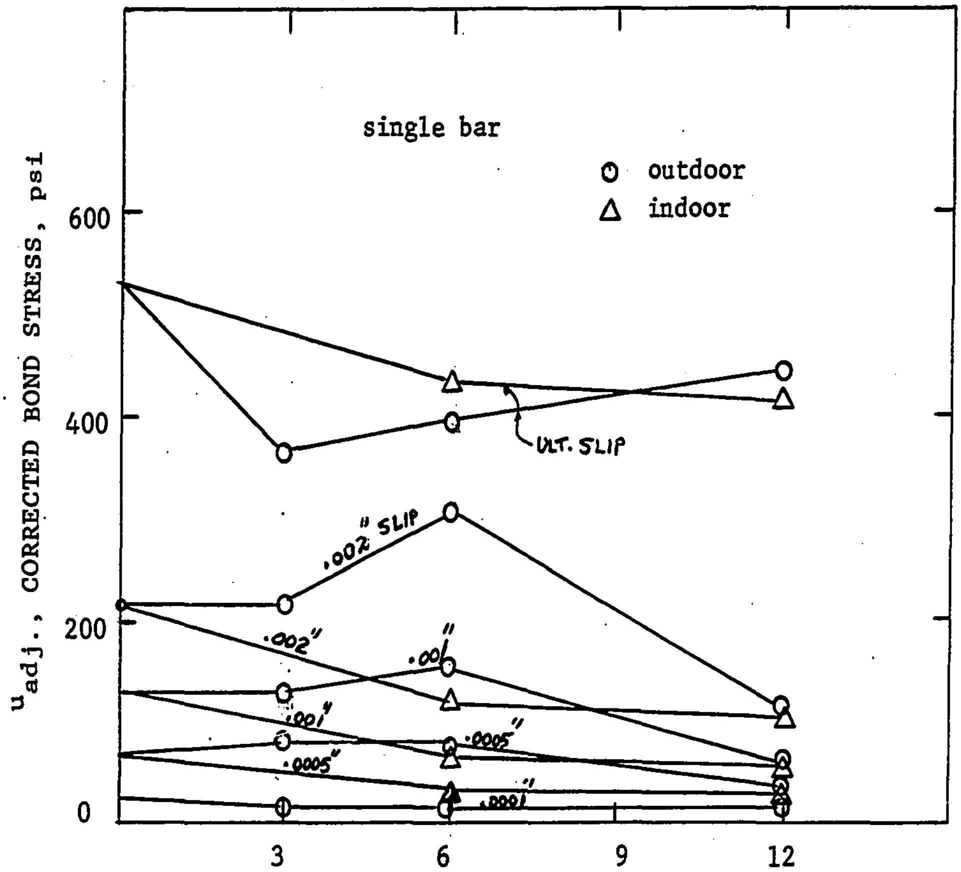


Fig. 6.6 Effect of corrosion on bond strength of No. 10 bars.



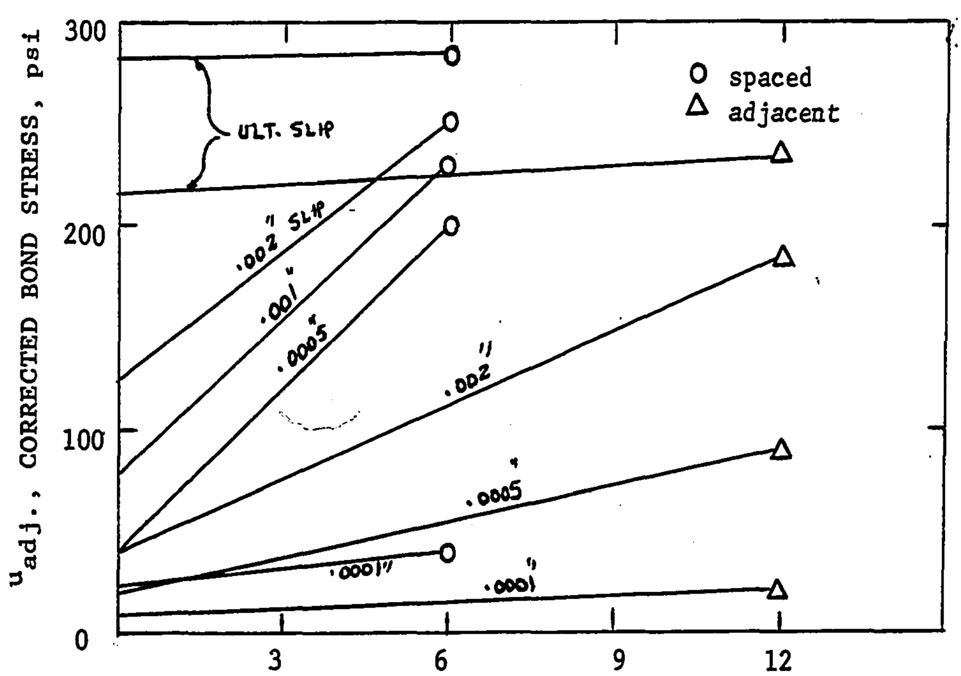


Fig. 6.7 Effect of corrosion on bond strength of No. 11 bars.

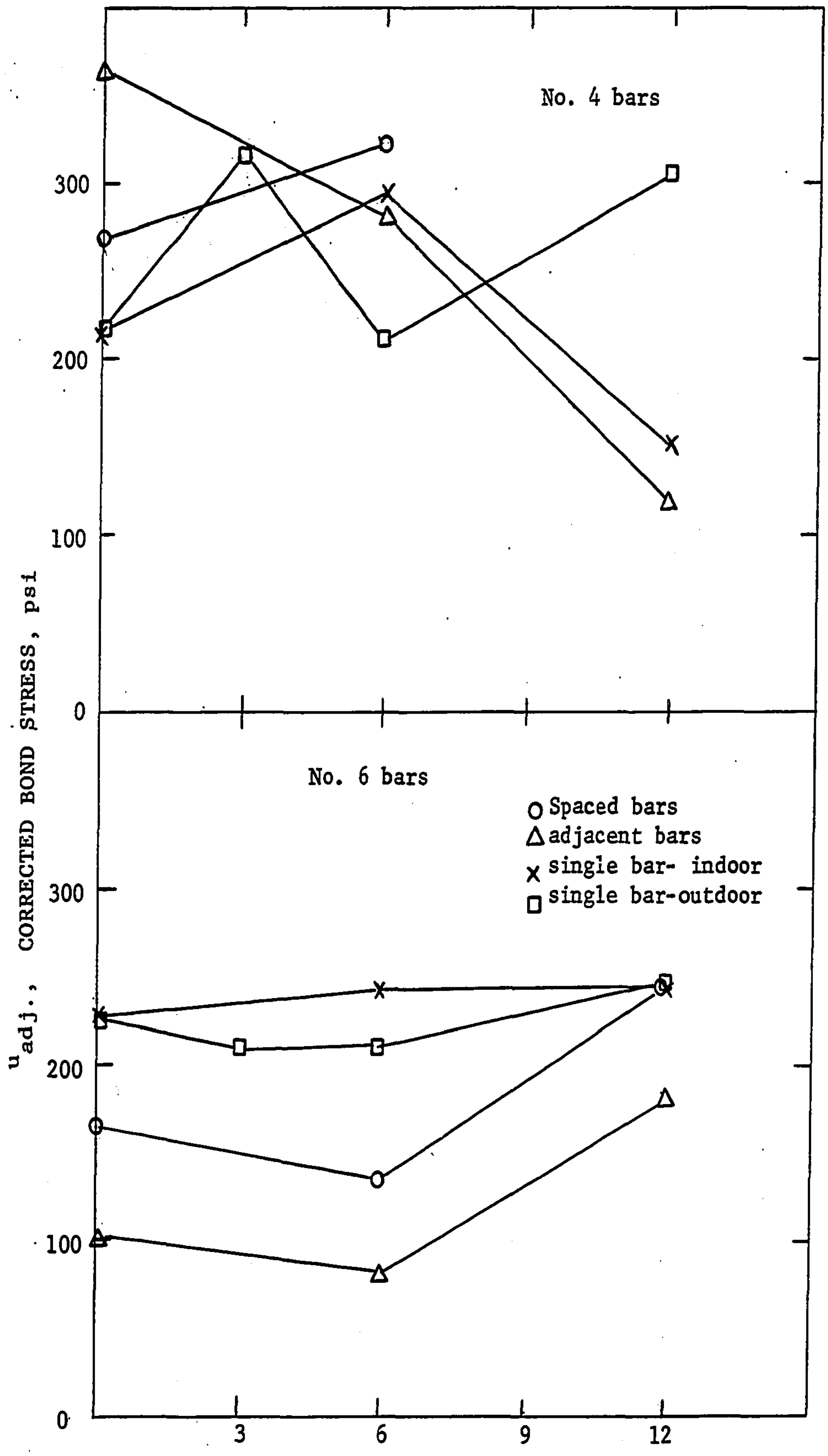


Fig. 6.8 Effect of corrosion on average bond stress.

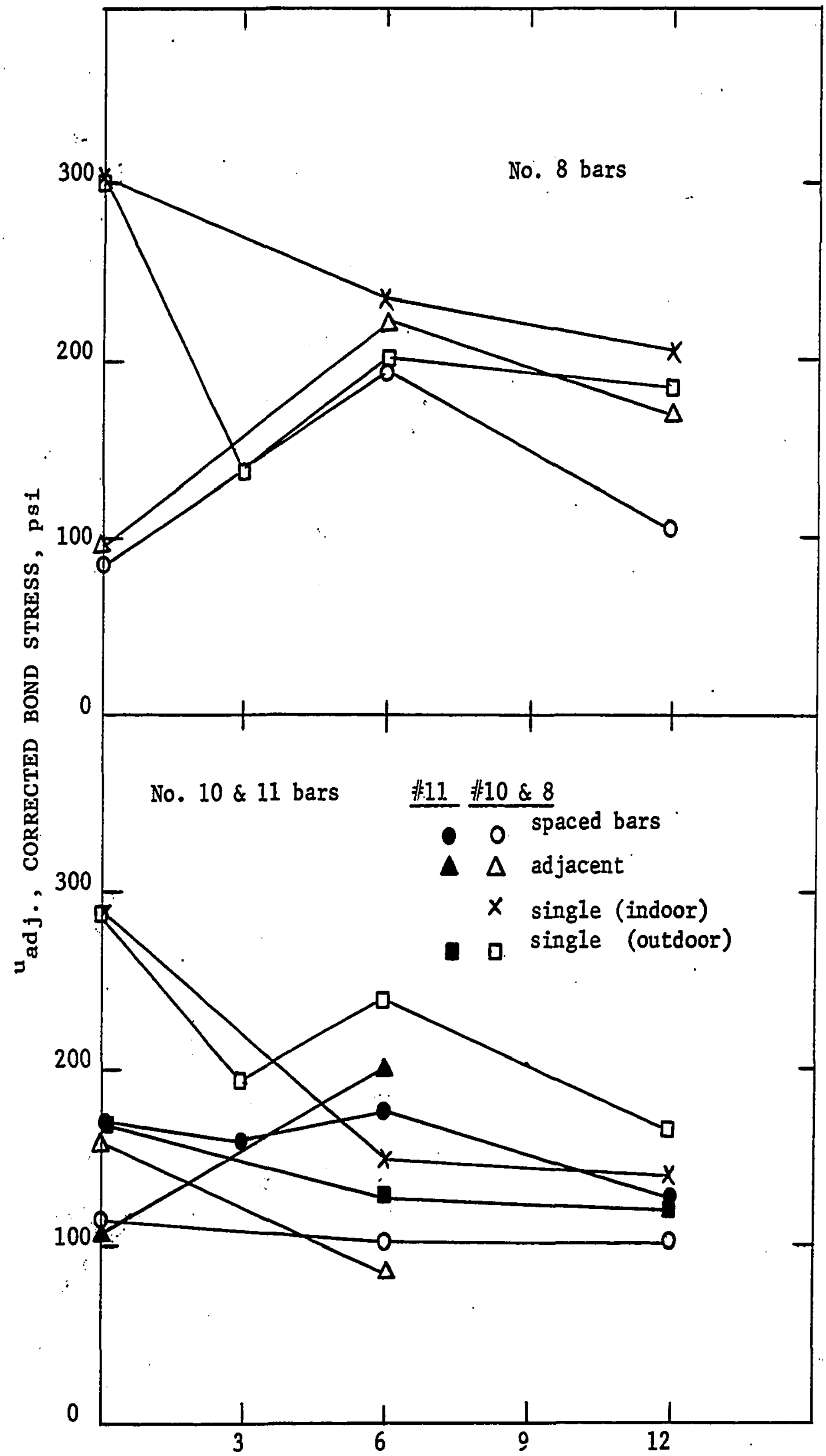


Fig. 6.9 Effect of corrosion on average bond stress of specimens with No. 8,10, & 11 bars.

in Sec. 1.1 and 2.2.1, the bond strength of the modern deformed bars is generated primarily by the mechanical interlocking of the deformations and concrete. Lutz (32) concluded that increasing the height of the deformations could cause a significant increase in the bond strength and slip resistance due to reduction of the bearing pressure on the deformations. With deformed bars ultimate load was much less dependent on the bar diameter than with plain bars. However, the bar diameter was a significant factor in the bond strength, especially when the reinforcement was confined. Any loss in bar diameter due to corrosion, however, means a reduction in the effective amount of steel and thus a lower ultimate strength for the bars.

The results of tensile tests on reinforcing bars with as-rolled and corroded surface conditions were reported in Table 3.1 and Table 5.2. From the test results the following conclusions were reached:

1. The indoor and outdoor corrosions for durations of 3,6,and 12 months did not affect the ultimate strength of the bars as compared with the strength of the as-rolled bars.

2. The ultimate strength of 3-months sea-water corroded bars was reduced by an average of 3.7 percent in comparison with the ultimate strength of unexposed bars.

After corrosion was removed, the final deformation height, the diameter, and the weight of the corroded bars were measured..The measurements, as compared to the initial values, seemed to indicate the followings:

1. There was no significant change in the deformation height, bar diameter (measured between the deformations), and bar length of

the coupons exposed to sea water (3 months), indoor and outdoor for 3,6, and 12 months.

2. Although a slight reduction in the weights was observed, the polished weights were much higher than the 94 percent of the theoretical weight limitation set by ASTM standards (Sec. 5.4.2).

6.4 Bond Stress Versus Bar Spacing and Bar Size

One of the objectives of this study was to investigate the splitting type of bond failure of deformed reinforcing bars through the effects of different bar spacings on bond strength. It was observed from Table A.3 in Appendix A that the load carrying capacity of Nos. 6, 8, and 10 bars with as-rolled surface condition and cast in double spaced bar-specimens was an average of 15 percent higher than that of closer bar spacing (adjacent tied), while the specimens with Nos. 4 and 11 bars showed 30 percent loss in bond strength. The 6-and 12-months-indoor corroded bars yielded similar results for different bar spacings. Bar sizes Nos. 4,6,and 8 carried an average of 13 percent more load than as-rolled bars as bar spacing increased. But a reduction of 26 percent was indicated for No. 11 bars.

Tests by Ferguson, et al. (10) and Lutz (32) have indicated that the resistance of closely spaced bars is lower because of the creation of a plane of weakness along the transverse axis of the bars. The results of this investigation partially support this conclusion. The variability in the results may be attributed to the fact that one specimen per bar size per degree of surface condition was tested for each bar spacing. Therefore, it seems that probably several specimens of a kind were needed if a trend of the effect of bar spacing on load carrying capacity of the specimens was to be identified with assurance. Nevertheless, the eccentric pullout specimens with three bar diameters bar spacing generally developed slightly better bond action than the specimens with adjacent-tied bars. There was also a dissimilarity of the cracking patterns as the bar spacing increased. The specimens with adjacent-tied bars failed as a splitting crack on top ran along the bar to the free end of the bar, while the failure of the specimens with spaced bars was a combination of splitting and diagonal tension cracks.

Plots of ultimate bond stress versus bar diameter for different corrosive environments and various time of exposure are shown in Figures 6.10 through 6.12. A least-square polynomial fit indicated that bond stress decreased quadradically with increasing bar diameter.

6.5 Bond Stress Versus Loaded and Free End Slips

The bond stress loaded-end slip curves were prepared for each of the eccentric pullout specimens and are presented in Figs. 5.1-5.20. The free-end slips were generally small and are not shown on these plots. Table A.3 of Appendix A shows the ultimate free-end slips as well as the ultimate loaded-end slips which were extrapolated from the above curves.

The slip values were averaged for the companion specimens of a kind and they were grouped in three arbitrary categories. The results are tabulated in Table 6.1. A slip level of 0.01 inch for the loaded-end was chosen because the past investigators have set this as a maximum tolerable crack width in a flexural member.

In general, the single-bar specimens containing the 3-months sea-water corroded bars indicated slightly higher ultimate loaded-end

TABLE 6.1

PERCENTAGE OF THE SPECIMENS FAILED AT DIFFERENT ULTIMATE

LOADED & FREE ENDS SLIP LEVELS

F R E E   E N D

	Single Bars	Double spaced bars	Double adjacent bars
Slip $\leq$ 0.00025 inch	53 %	80 %	67 %
0.00025 $<$ Slip $\leq$ 0.00050 in.	15	15	20
Slip $>$ 0.00050	32	5	13

L O A D E D   E N D

	Single bar	Double spaced bars	Double adjacent bars
Slip $\leq$ 0.005 inch	17 %	63 %	54 %
0.005 $<$ Slip $\leq$ 0.010 inch	40	37	46
Slip $>$ 0.010 inch	43	0	0

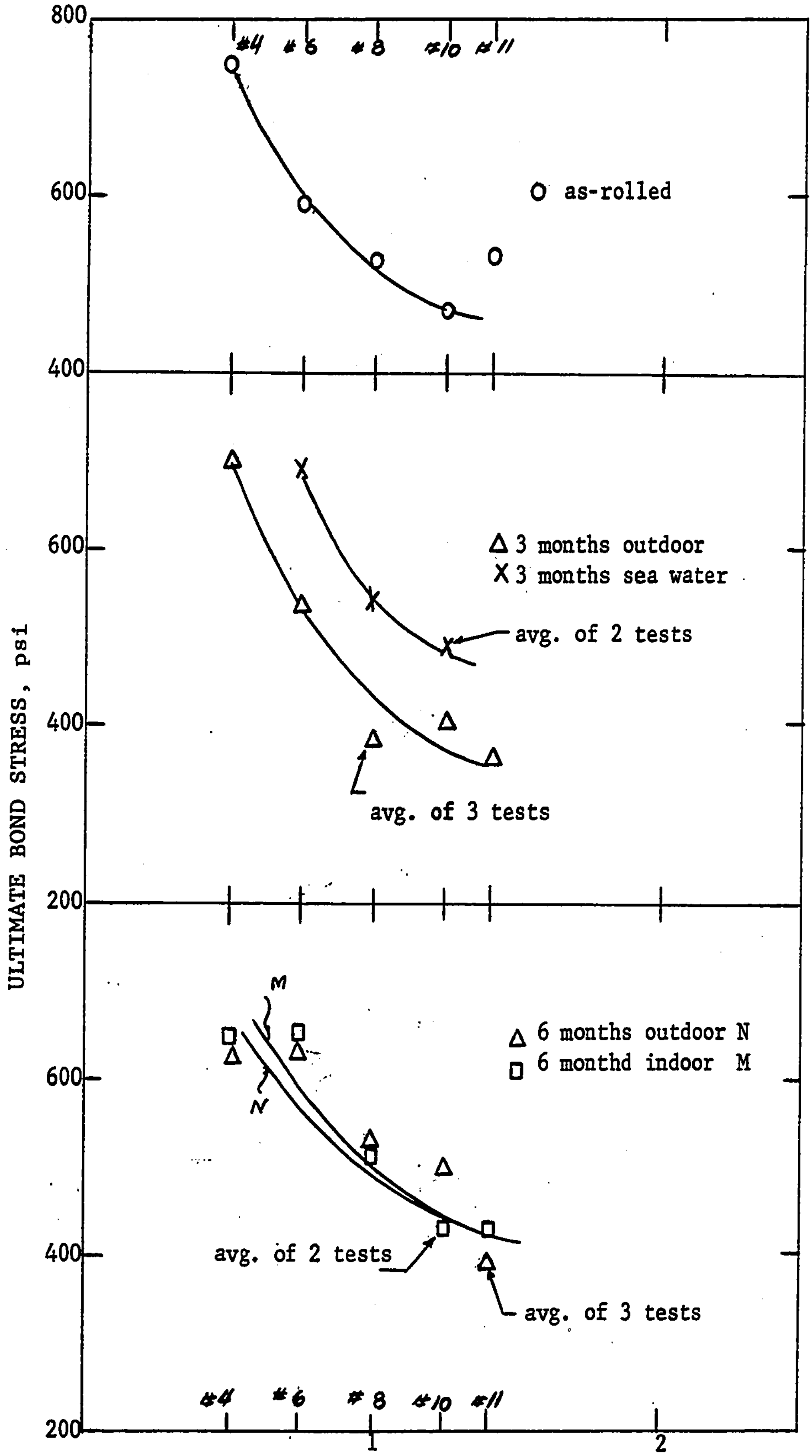


Fig. 6.10 Effect of bar diameter on ultimate bond stress of single bar specimens.

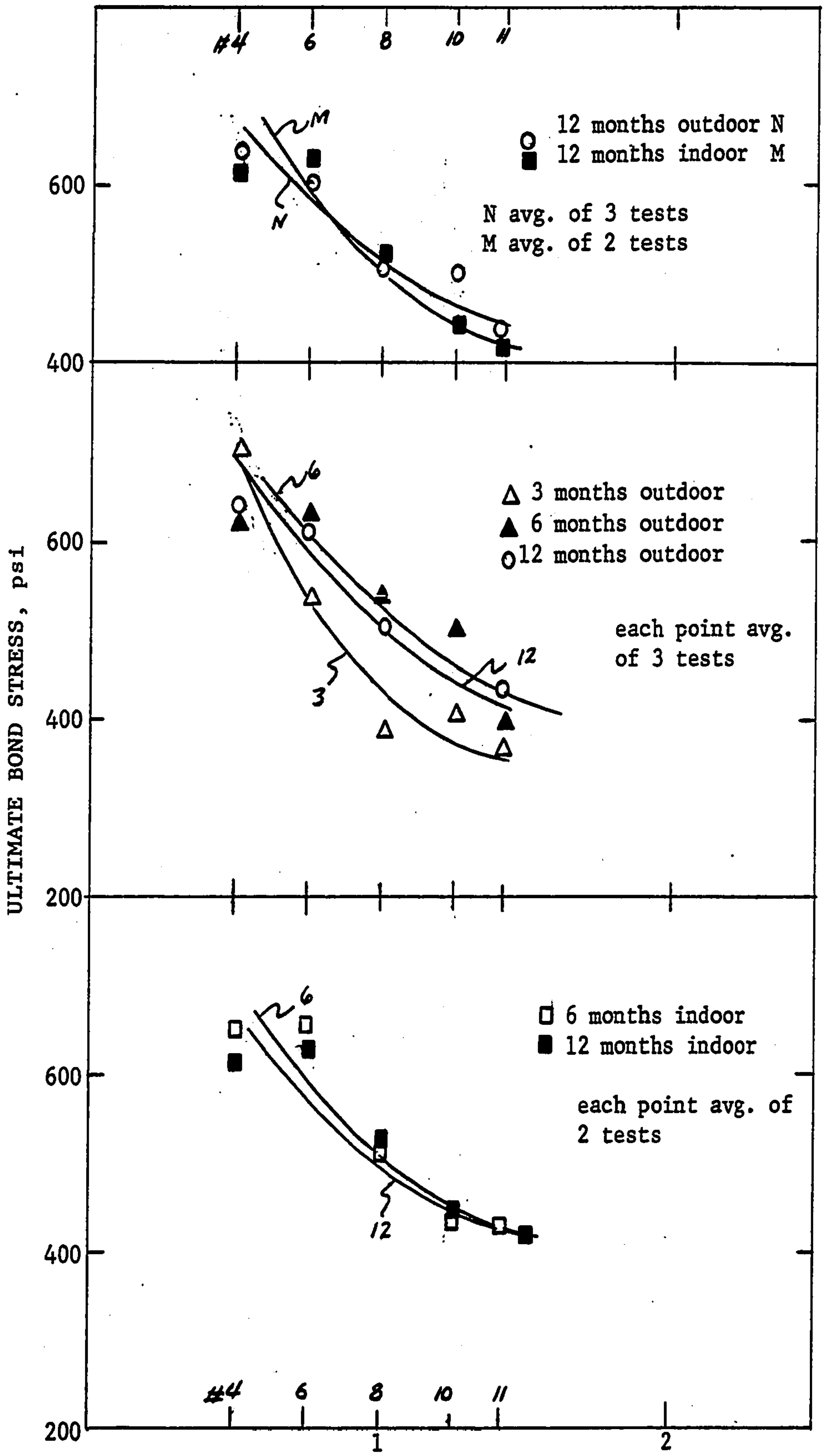


Fig. 6.11 Effect of bar diameter on ultimate bond stress of single bar specimens.

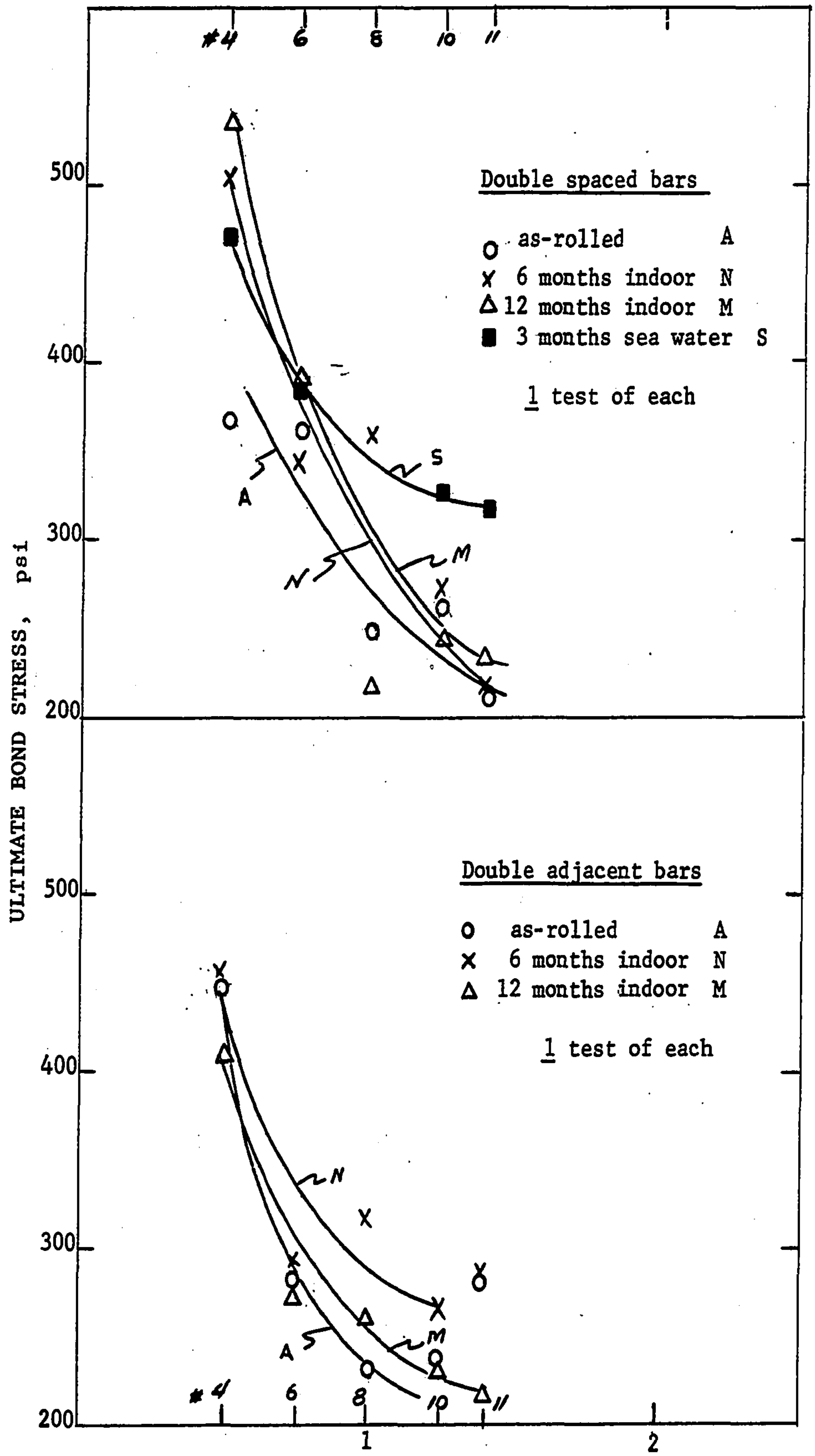


Fig. 6.12 Effect of bar diameter on ultimate bond stress of double spaced and double adjacent bars specimens.

slip as compared with other bar surface conditions studied. However, the free-end slip for these bars was considerably higher than for the remainder of the bars. This was due to the fact that most of the specimens with sea-water corroded bars split on top (along the bar). Apparently splitting was the means by which some of the unevenness in bond stress distribution was smoothed out and the entire bond between reinforcing bar and concrete was lost, leading to higher free end slips. In order to investigate the possibility of existance of any correlation between loaded-end slip and bar diameter as well as bar surface conditions and the time of exposure, Figurs 6.13 through 6.15 were prepared. No over-all trend is observed. However, the data indicated that the loaded-end slip decreases with increase in bar diameter. This is evident in Figures 6.13 and 6.15 for as-rolled and 3-months sea water corroded bars.

6.6 Bond Stress Versus Limiting Shear Stresses

The interrelationship between bond stress and shear stress will now be examined by means of a comparison of current shear and bond stress equations with the results of the eccentric pullout specimens.

Flexural cracking changes the bond stress distribution in a reinforced concrete flexural member and a corresponding lare change in shear stress results. Commonly, two kinds of bond stresses are considered to exist in a flexural member:

1. Flexural bond, due to shear and given by the equation:

$$u = \frac{V}{\Sigma o\ jd}$$

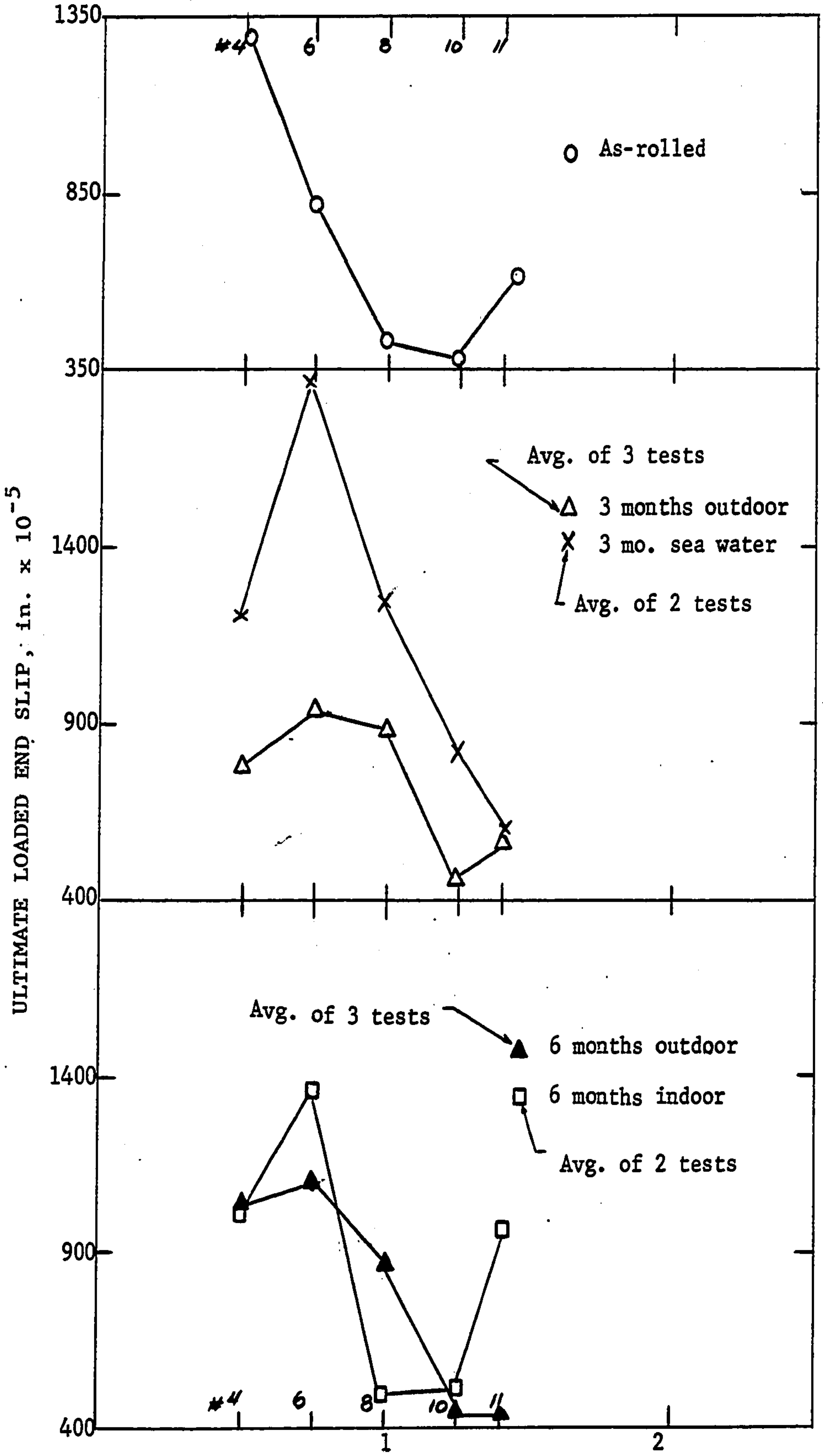


Fig. 6.13 Effect of bar diameter and corrosion on ultimate loaded end-slip of single bar specimens.

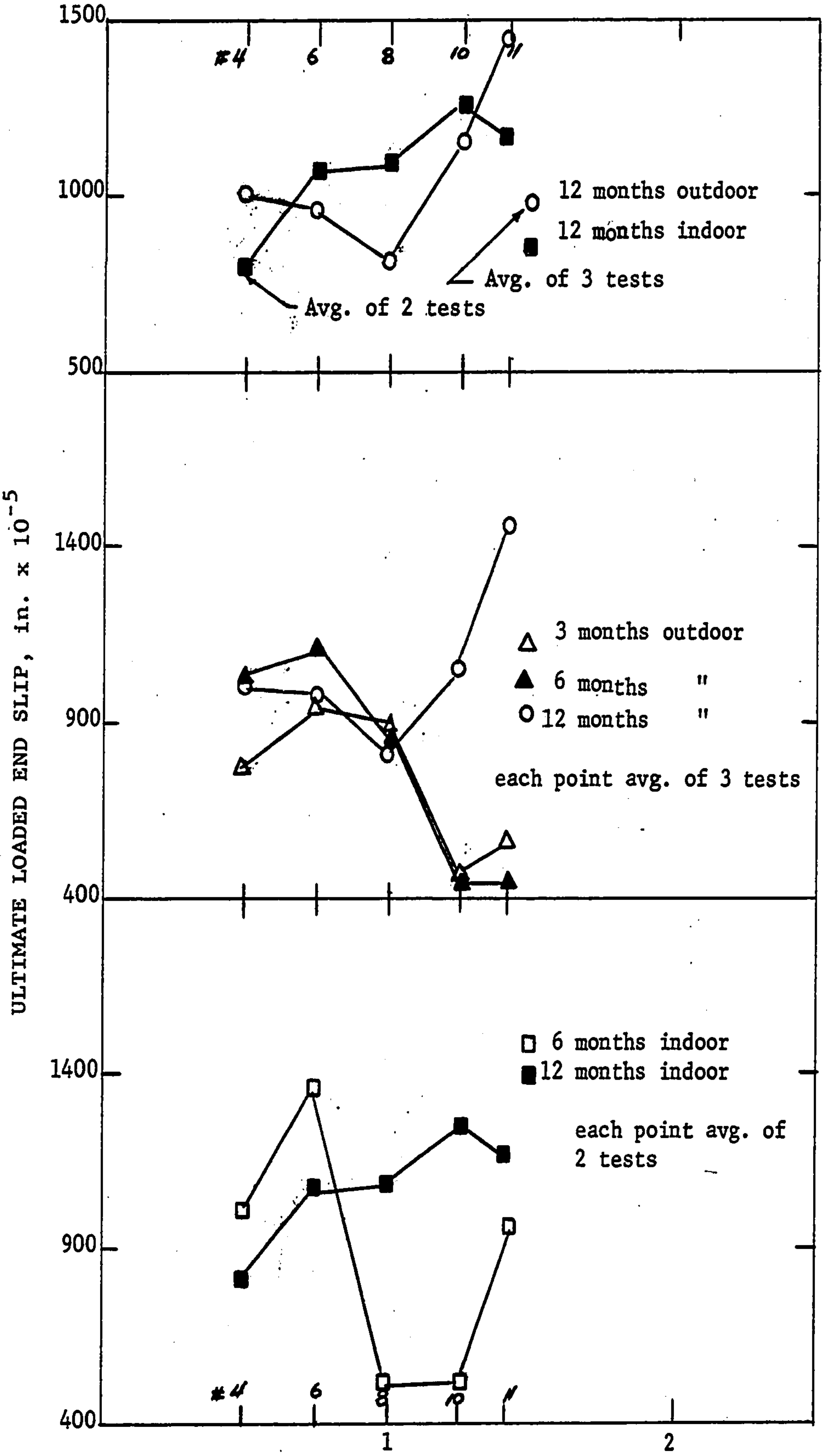


Fig. 6.14 Effect of bar diameter and corrosion on ultimate loaded end-slip of single bar specimens.

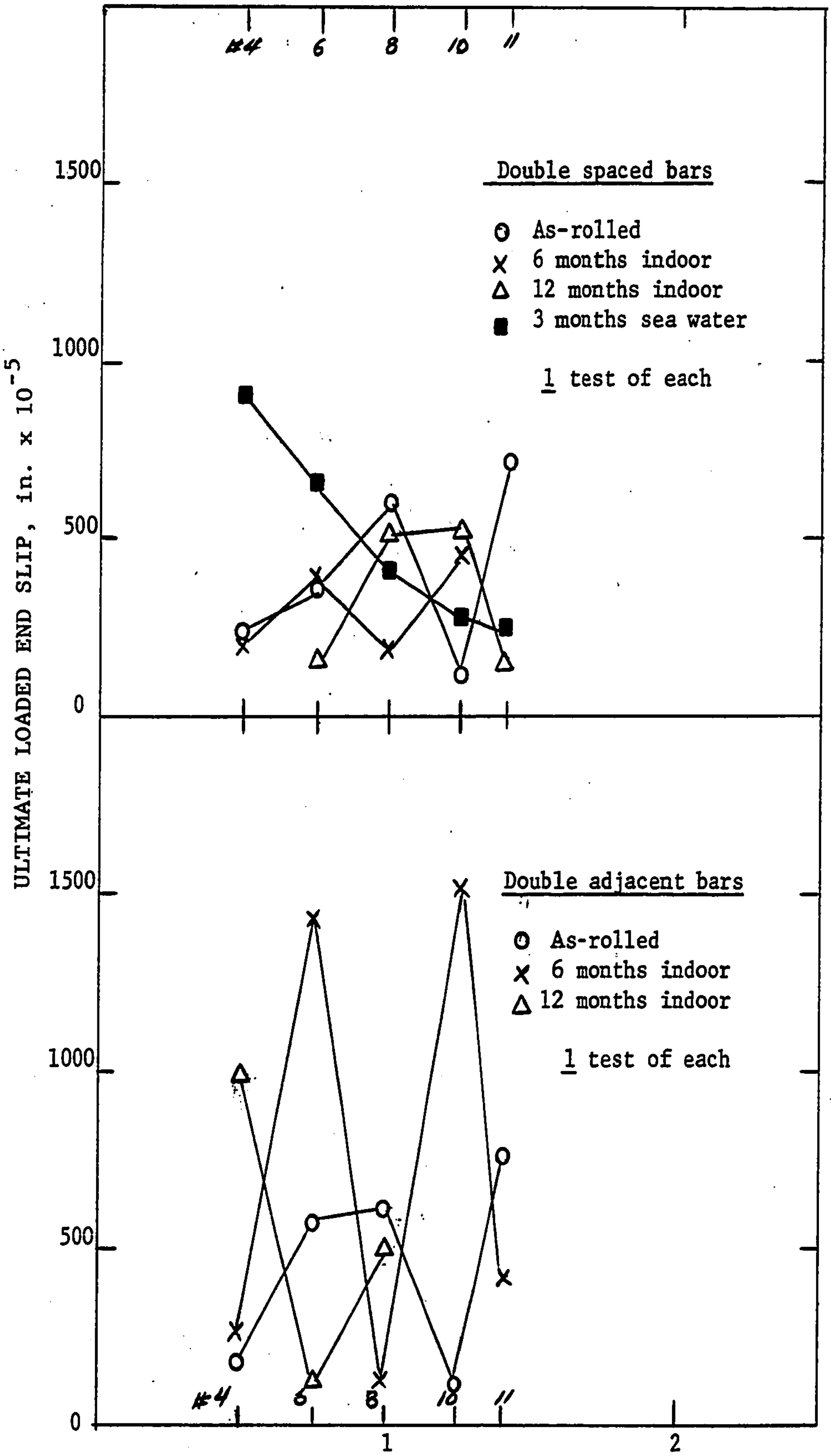


Fig. 6.15 Effect of bar diameter and corrosion on ultimate loaded end slip of double bar specimens.

$$\text{or} \qquad u = \frac{V}{N\pi D\ jd} \qquad (1)$$

where u is bond stress, V is shear, N is the number of bar.

2. Anchorage bond stress:

$$u = \frac{A_s\ f_s}{\Sigma o\ L''}$$

$$\text{or} \qquad u = \frac{A_s\ f_s}{N\pi D\ L''} \qquad (2)$$

where A_s is bar area, D is bar diameter, f_s is steel stress, and L" is the embedment length of bar in concrete.

If the nominal ultimate shear stress, as a measure of diagonal tension, is combined with equation (1) the following equation results:

$$v = \frac{V}{b\ jd}$$

$$u = \left(\frac{b}{N\ D}\right) \times \left(\frac{v}{\pi}\right) \qquad (3)$$

where b is the width of the flexural member.

The parameter $\frac{b}{N\ D}$ relates bond stress and shear and the ratio b/N is considered by some investigators to be a good measure of the lateral spacing of reinforcing bars. Figures 6.16 through 6.18 exhibit the relationship between $\frac{b}{N\ D}$ parameter and bond stress existing at various limiting shear stresses imposed by the 1963 ACI Building Code. The results of this ivestigation are compared with the eccentric bond tests of Ferguson, Turpin, and Thompson (10). The test points in Fig. 6.16 are the average values of either two or three tests. Fig.6.17 shows the values of bond stress for the individual tests, while the

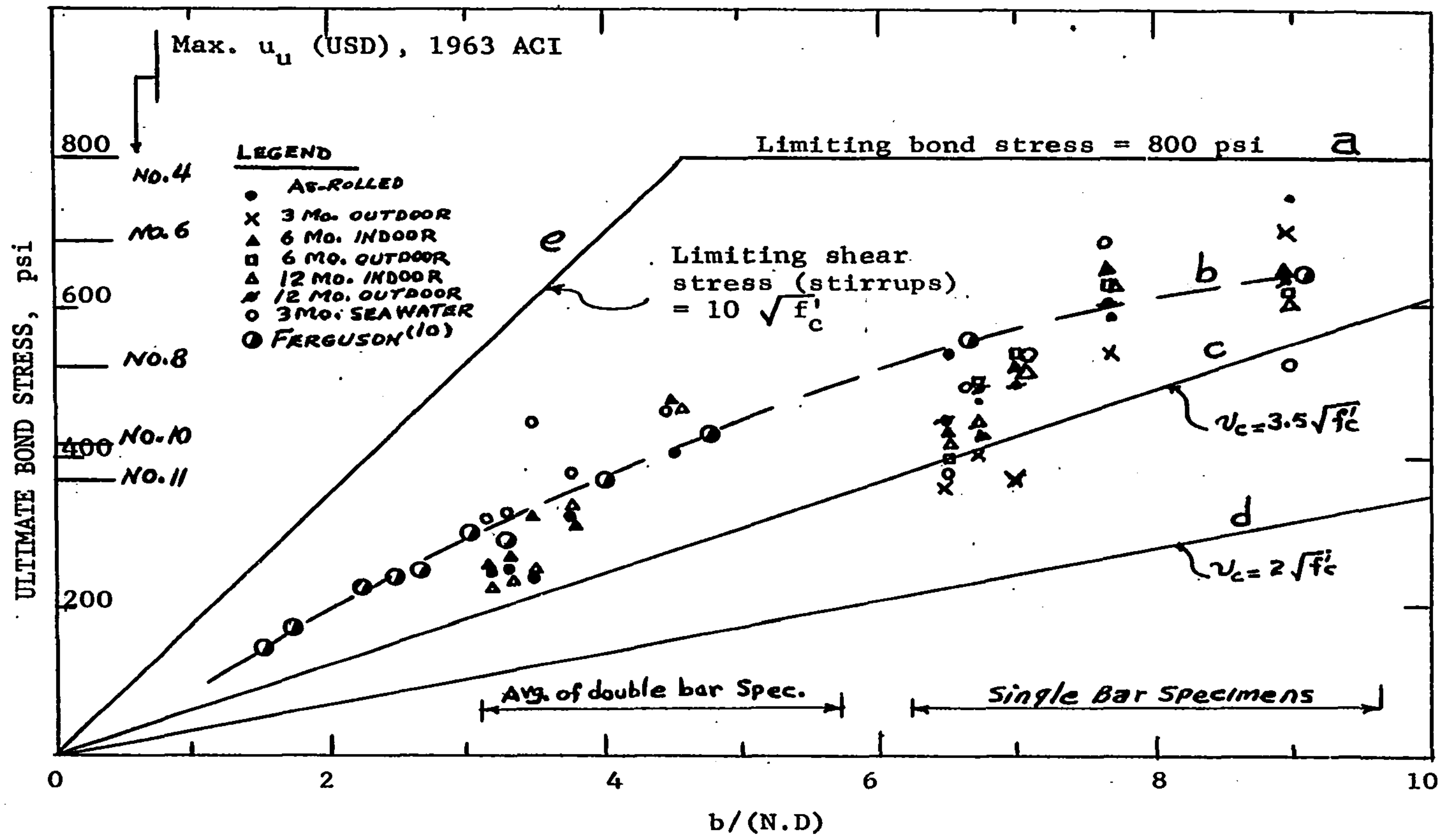


Fig. 6.16 Ultimate bond stress at various limiting shear stresses & test results of Ferguson as a function of b/(N.D). f'_c = 3,000 psi

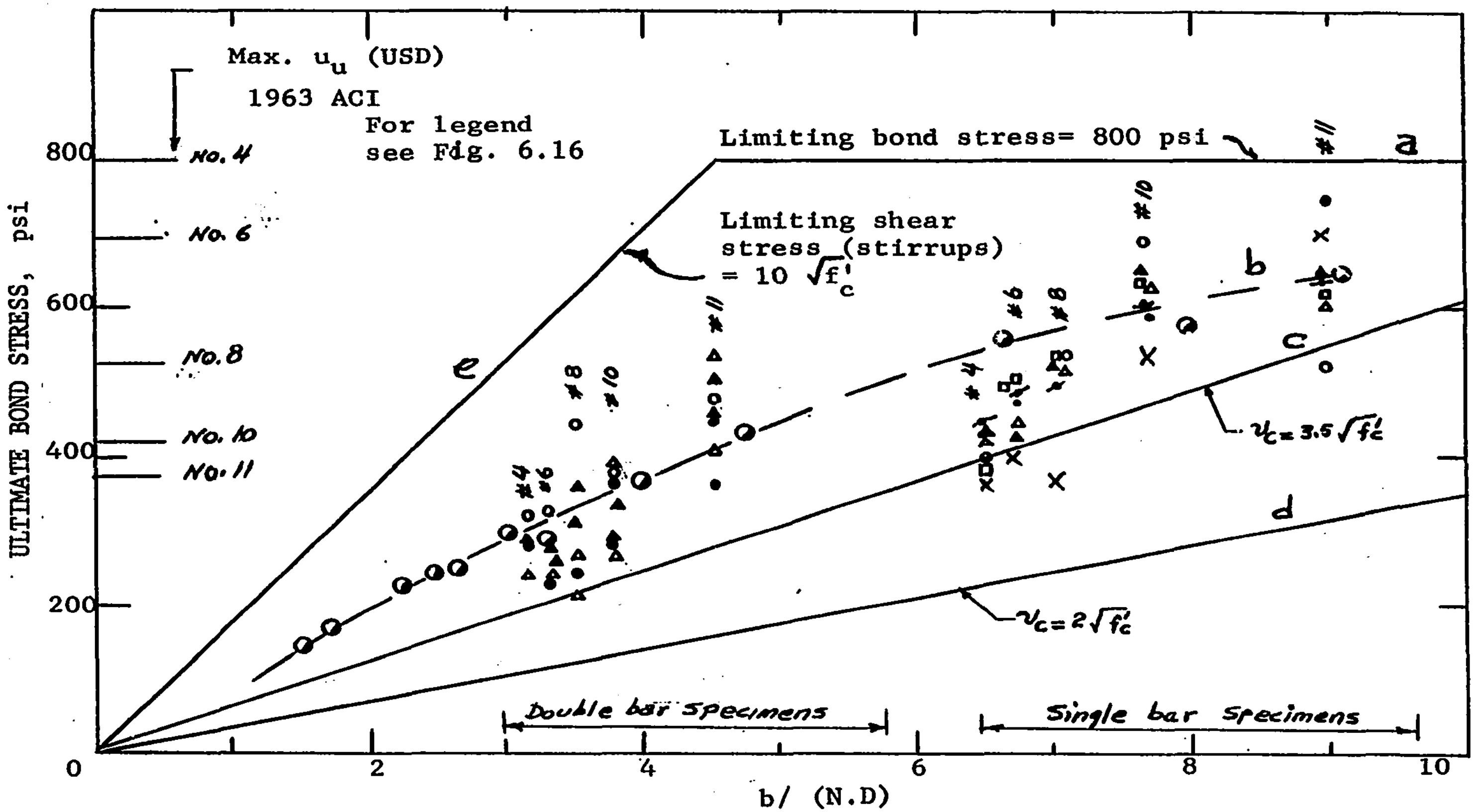


Fig. 6.17 Ultimate bond stress at various limiting shearing stresses & test results of Ferguson as a function of b/(N.D). f'_c = 3,000 psi.

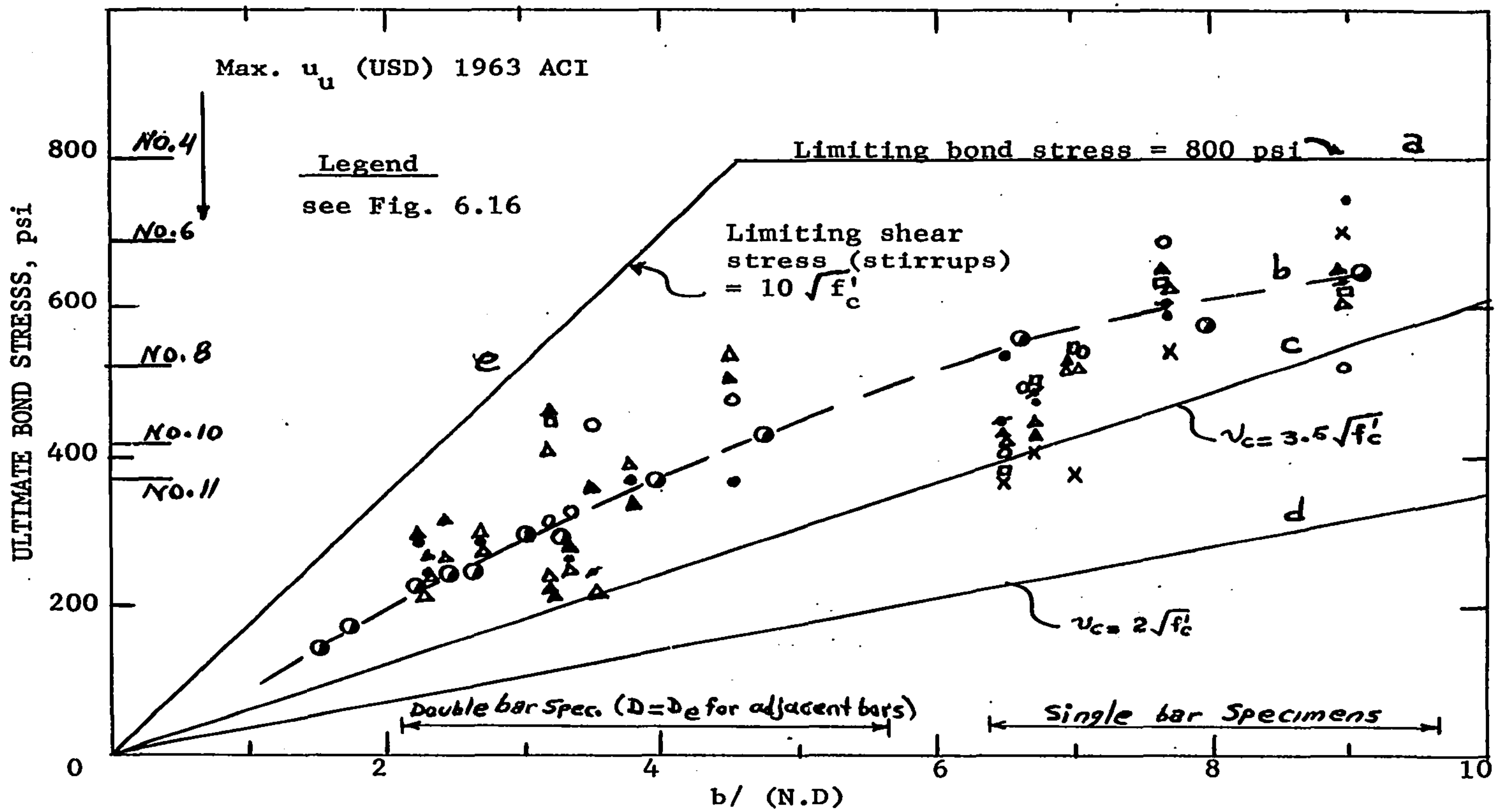


Fig. 6.18 Ultimate bond stress at various limiting shearing stresses & test results of Ferguson as a function of b/ (N.D). f'_c = 3,000 psi.

points for double-adjacent bar tests shown in Fig. 6.18 were calcualted according to Sec. 804-f of the 1963 ACI Code, using an equivalent bar diameter.

In an eccentric bond pullout test normal cracking occurs as in a flexural member without the presence of the shear forces which exist in such members. The formation of diagonal cracks and the presence of shear forces influence the bond stress distribution or even cause bond failure as a result of shear failure. "line a" in Figs. 6.16 - 6.18 represents the ultimate allowable bond stress set by the Code. When no stirrups are used, such as the specimens of this study, the bond failure stresses (Line a) are well above the bond stresses which exist at the limiting shear stress v_c (lines c & d). Therefore, shear failure should occur before bond failure. The test results of this research study with various surface corrosion, and the test values of eccentric pullout tests by Ferguson, et al. fall between the above lines (lines a,c,and d) for the b/(ND) values shown.

When web reinforcement is used, the Code limiting shear stresses are represented by line e. For low values of b/(ND) the maximum shear stress according to the Code (Line e) closely predicts the experimental failure stresses.

In connection with Figs. 6.16- 6.18, the following points were observed:

1. Line b was constructed from the test results of the study conducted by Ferguson, et al. (10) on eccentric bond pullout specimens with No. 4 and No. 6 modern deformed reinforcing bars. Their data were corrected to a concrete compressive stength of 3,000 psi. The

test points of the present investigation for No. 4 and No. 6 bars follow the trend of line b, while the points for Nos. 8,10, and 11 bars with high b/(ND) ratios (single bar specimens) fall below the line. In the low range of b/(ND), the test values scatter evenly about line b.

2. Curve b seems to question the validity of the Code limitation of one bar diameter or one inch minimum clear spacing rule. There is no break anywhere on the curve that could set a logical minimum.

3. For concrete compressive strength of 3,000 psi and no web reinforcement the allowable shear limitation of $3.5\sqrt{f'_c}$ is the only limitation needed upon the minimum spacing of bars, other than that imposed by the size of the aggregate. Therefore, very small spacings are as strong as needed in bond for current code requirements.

4. The expressions and data shown in Figs. 6.16-6.18 are for one row of tensile reinforcement; the use of two or more rows of bars might alter the situation.

5. The variation in test results for bars with various surface corrosions was high. Therefore, it was difficult to identify a specific trend. However, within the limitations of this investigation, it may be concluded that the bar surface condition had little or no influence on the interrelationship between bond stress and the parameter b/(ND).

6. The eccentric bond pullout specimens of this study were designed to fail by splitting of concrete. The results obtained substantiate the fact that where splitting is possible, the present maximum permissable bond stress of up to 800 psi set by the Code appears to be high. However, this trend has not been established by the test data presented herein since embedment lengths were purposely held

below that which would be required to cause yielding of the bars.

7. The limiting shear and bond equations in the Code are conservative to various degrees.

6.7 Influence of Splitting and Web Reinforcement on Bond Stress

It has been pointed out by the past investigators that when splitting is not prevented by external forces, special reinforcement, or a large mass of concrete, such splitting appears to lower the overall bond resistance. With this in mind, the specimens of this study were desgined to fail by splitting of concrete.

Generally speaking, the failure mode of the specimens tested fall into three categories:

1. The first group includes all the specimens with two bars and a clear bar spacing of three bar diameters. A horizontal crack occured on the loaded end of the specimen at the level of the bars. Later, one or two transverse flexural cracks formed on the top near the loaded end of the specimen. Failure occured when a layer of concrete over the bars splitted loose after the horizontal crack on the loaded end propagated on the sides. In some of the specimens the final failure was accompanied with a longitudinal split on top originating from the previously formed transverse flexural cracks on top. Figures 5.21, 5.24, 5.27, 5.30, and 5.33 show the sequential crack pattern for these specimens.

2. The second type of failure was associated with the eccentric bond pullout specimens with two adjacent-tied bars. The ultimate

crack pattern of these specimens primarily depended on the location and the type of the firstcrack.

(a) If the first crack was a flexural transverse crack on the top of the specimen, then one or two more transverse cracks occured on the top. These cracks extended vertically on the side and later turned into diagonal tension cracks pointing toward the mid-height of the loaded end of the specimen. As the loading continued, a longitudinal crack along the bar originated from one of the transverse cracks. The specimen suddenly failed when the diagonal cracks reached the loaded end and the longitudinal split on the top progressed to the free end of the bar. Figs. 5.25, 5.28, and 5.34 show this type of failure.

(b) If the first crack was a vertical one originating between the adjacent bars and progressing upward on the loaded end and along the bar on top of the specimen, then a transverse flexural crack appeared on the top. The specimen failed by splitting of concrete. This type of failure is seen in Figs. 5.22, 5.25, 5.28, and 5.34.

3. The crack pattern of the single-bar specimens was the third distinct behavior. The splitting failure of these specimens was preceded by a vertical crack developing on the loaded-end and extending lengthwise to the free-end of the bar. Figures 5.23, 5.26, 5.29, 5.32, and 5.35 show this type of failure.

As previously mentioned, the eccentric bond pullout specimens tested were designed to fail by splitting of concrete. Therefore, no binding was used in the specimens. It was observed that this led to a bond failure by cracking of the concrete transversely (flexural cracks) and longitudinally (splitting). The anchorage zone at each instance of loading extended from loaded end of the bar to the location of the maximum steel stress. However, due to diagonal tension cracking, splitting, and slip of the reinforcing bar, the maximum steel stress progressed toweard the free end of the bar with increase in load.

Ferguson, et al. (1,10,33) concluded that reinforcement, which was able to restrain the progress of the longitudinal cracking, improved the bond strength. Evaluating the beneficial effect of stirrups in beams is difficult because of the effect of stirrups on shear strength. On the other hand, the eccentric bond pullout specimens are considered to be a better means of evaluating the influence of binding reinforcement as was done by Ferguson. In the present investigation three single-bar specimens with Nos. 8,10, and 11 bars, and 12-months outdoor surface corrosion, were tested with stirrups binding the main reinforcing bar. The results indicated improvement of bond strength with stirrups, but not to the extent observed by Ferguson, et al. (10). As compared to the bars with similar surface condition, No. 8 bars developed 24 percent, No. 10 bars 32 percent, and No. 11 bars 11 percent higher bond resistance. There was little indication of the difference in the progress of the splitting or flexural cracks in specimens with and without binding reinforcement. The transverse reinforcement apparently did little or nothing to inhibit the formation and the progress of the cracks. The failure of the specimens with stirrups binding the main reinforcement was sudden, but not explosive as was the case with specimens without stirrups.

It has been suggested (32) that the contribution of web reinforcement to ultimate load can best be expressed by the variable:

$$\frac{D_v^3 L''^2 D}{s}$$

which is the total amount of reinforcement in embedment length times the product of the bar diameter and embedment length (D_v = stirrups diameter, s = the stirrup spacing, D = bar diameter, and L" = embedment length). The exponent on D_v reflects a compromise between the area and the moment of inertia as pertinent variables. To determine the ultimate load, Lutz (32) plotted the ultimate load per unit length (P_u / L") against the term:

$$\frac{D^2}{L''}\left[1 + k\frac{D_v^3 L''^2}{s D}\right]$$

A value of k=1 seemed to correlate best with the data. A plot of Lutz's data, corrected to a concrete compressive strength of 3,000 psi and the test results of three specimens with web reinforcement of this investigation is given in Fig. 6.19. It should be observed that Lutz's results were obtained from 18 eccentric pullout tests on two bars in two embedment lengths with just two stirrup sizes and one size of concrete block. Therefore, the quantitative conclusions shown on Fig. 6.19 are preliminary. However, these quantities illustrate how bar diameter, embedment length, and the binding of web reinforcement can influence the ultimate bond strength.

6.8 Test Ultimate Strength Versus Calculated Ultimate Strength

The test ultimate load capacity of the eccentric bond pullout specimens based on a concrete compressive strength of 3,000 psi

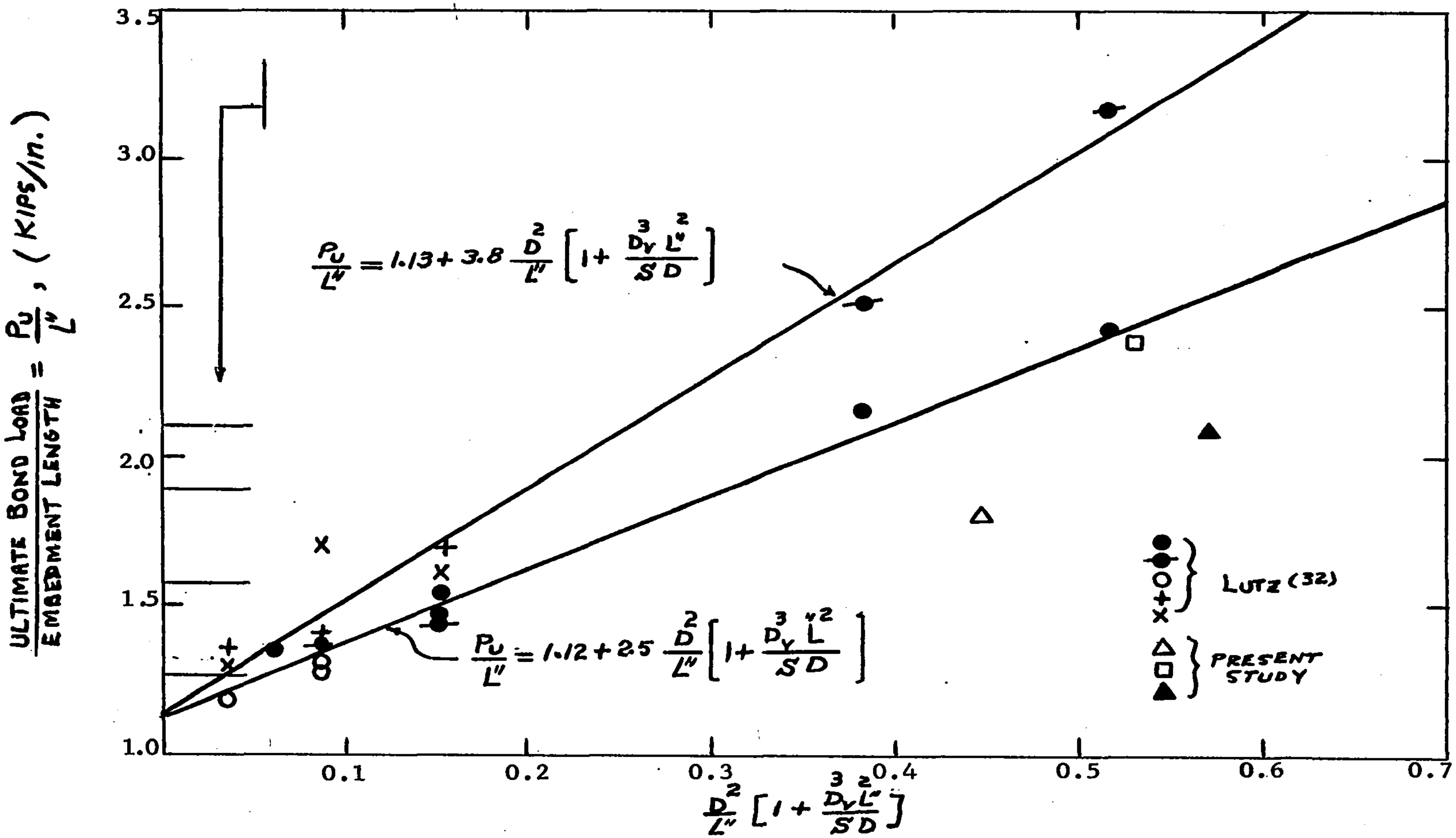


Fig. 6.19 Relationship for ultimate bond load in eccentric pullout tests of Lutz and the present investigation.

and the expected (calculated) capacity of the specimens are tabulated in Table 6.1. The theoretical strength, in kips, was determined according to the 1963 ACI Building Code from the development length equation:

$$u = \frac{T}{\sum oL''} \qquad (a)$$

If the Code's allowable ultimate bond stress (equation b) for deformed reinforcing bars conforming to ASTM A-305-56T is substituted in (a), equation (c) results:

$$u = \frac{9.5\sqrt{f'_c}}{D} \leqq 800 \text{ psi} \qquad (b)$$

$$T = (1.634) \times L'' \qquad (c)$$

where T is the calculated ultimate strength. It should be noted that for f'_c = 3,000 psi equation (b) implies that ultimate strength is independent of the bar diameter for No. 6 to No. 11 bars but it is linearly dependent on bar diameter for smaller bars.

The ratios of the test ultimate strength to the calculated ultimate strength are given in Table 6.2. A study of these ratios indicated that 45 percent of all the single-bar specimens (with as-rolled and corroded bars) failed at loads between 2 to 19 percent higher than the calculated loads and 55 percent of those specimens developed loads between 5 and 20 percent lower than expected. Eighty percent of the specimens with double spaced bars exhibited load carrying capacities of about 9 to 55 percent higher than the calculated capacities, while the ramaining 20 percent failed at loads values between 1 and 11 percent lower than the expected loads. The ultimate

TABLE 6.2

TEST ULTIMATE STRENGTH VS. CALCULATED ULTIMATE STRENGTH

Spec. No.	$T_{adj.}$ (kips)	$T_{cal.}$ (kips)	$\frac{T_{adj.}}{T_{cal}}$	Spec. No.	$T_{adj.}$	$T_{cal.}$	$\frac{T_{adj.}}{T_{cal.}}$	Spec. No.	$T_{adj.}$	$T_{cal.}$	$\frac{T_{adj.}}{T_{cal.}}$
4A0-a	11.2	12.2	0.92	6N6-c	19.8	23.1	0.86	8M12-d	32.7	31.0	1.05
-b	13.4	11.9	1.13	-d	21.6	23.1	0.93	8N12-c	34.4	31.0	1.11
-c	11.4	12.2	0.93	-e	22.2	23.3	0.95	-d	28.7	31.2	0.92
6A0-a	24.4	23.3	1.05	8M6-a	44.2	32.1	1.38	-e	27.0	31.0	0.87
-b	18.9	23.2	0.81	-b	38.2	31.2	1.22	10M12a	47.4	39.6	1.20
-c	19.8	23.3	0.85	-c	31.2	31.4	0.99	b	45.2	39.4	1.15
8A0-a	29.6	31.0	0.95	-d	31.2	31.2	1.00	c	42.7	39.4	1.08
-b	27.7	31.0	0.89	8N6-c	30.7	31.2	0.98	d	42.2	39.4	1.07
-c	31.7	31.0	1.02	-d	31.9	31.0	1.03	10N12c	57.7	39.4	1.46
10A0-a	51.5	40.2	1.28	-e	32.0	31.6	1.01	d	50.1	39.6	1.26
-b	65.7	39.4	1.67	10M6-a	52.2	39.2	1.33	e	37.7	39.4	0.96
-c	45.8	39.6	1.16	-b	51.2	39.4	1.30	11M12a	56.3	44.1	1.28
11A0-a	51.3	44.1	1.16	-c	45.5	39.4	1.15	b	51.3	44.1	1.16
-b	69.3	45.3	1.53	-d	37.7	39.6	0.95	c	50.6	44.3	1.14
-c	64.6	44.3	1.46	10N6-c	47.1	39.4	1.19	d	49.2	44.1	1.11
4N3-c	9.1	11.9	0.76	-d	43.0	39.4	1.09	11N12c	50.8	44.1	1.15
-d	10.0	12.1	0.83	-e	54.1	39.4	1.37	d	51.4	44.1	1.16
-e	12.9	12.6	1.02	11M6-a	50.8	44.1	1.15	e	56.7	44.1	1.29
6N3-c	18.9	23.5	0.80	-b	68.8	44.1	1.56	4S3- a	17.0	11.9	1.43
-d	17.5	23.3	0.75	-c	51.4	44.1	1.16	c	8.1	11.9	0.68
-e	17.5	23.1	0.76	-d	52.4	44.1	1.19	d	7.6	12.1	0.63
8N3-c	26.3	31.0	0.85	11N6-c	43.6	44.1	0.99	6S3-a	25.5	23.3	1.09
-d	21.9	31.2	0.70	-d	51.4	44.1	1.16	c	25.1	23.1	1.09
-e	20.6	31.2	0.66	-e	48.1	44.1	1.09	d	21.3	23.3	0.91
10N3-c	37.7	39.2	0.96	4M12a	15.9	11.9	1.34	8S3-a	53.4	31.0	1.72
-d	42.0	39.2	1.07	b	12.2	11.9	1.02	c	29.8	31.0	1.09
-e	35.7	39.4	0.91	c	8.7	11.9	0.73	d	34.7	31.0	1.12
11N3-c	53.0	44.1	1.20	d	9.5	11.9	0.80	10S3-a	63.1	39.6	1.59
-d	42.8	44.1	0.97	4N12c	9.5	12.1	0.78	c	48.5	39.4	1.23
-e	35.0	44.1	0.79	d	10.6	12.1	0.88	d	46.6	39.6	1.18
4M6-a	14.8	11.8	1.25	e	9.5	11.9	0.80	11S3-a	75.7	43.9	1.72
-b	13.4	11.8	1.14	6M12a	26.4	23.3	1.13	c	42.0	44.1	0.95
-c	9.6	11.9	0.81	b	18.1	23.3	0.78	d	47.6	44.1	1.08
-d	11.3	12.1	0.93	c	19.1	23.3	0.82				
4N6-c	9.3	12.2	0.76	d	23.1	23.3	0.99				
-d	9.2	11.9	0.77	6N12c	21.5	23.3	0.92				
-e	9.5	12.1	0.78	d	16.3	23.3	0.70				
6M6-a	23.4	23.7	0.99	e	22.8	23.3	0.98				
-b	19.1	21.8	0.88	8M12a	26.1	31.2	0.84				
-c	22.1	23.3	0.95	b	31.2	31.0	1.01				
-d	22.1	21.8	1.01	c	29.0	31.0	0.93				

A= as-rolled
M= indoor
N= outdoor
S= sea water
a= double spaced bars
b= double adjacent "
c,d,e= single bar specimens.

strength of 73 percent of the specimens with double adjacent bars was 9 to 42 percent in excess of the theoretical failure loads. The remaining 27 percent of the specimens failed at loads 11 to 18 percent lower than the calculated loads. A detailed comparison of the test ultimate strength with calculated ultimate strength for various bar sizes and bar spacings is presented in Table 6.3.

6.9 Analytical Analysis of the Bond Data

An attempt was made to express quantitatively the ultimate load carrying capacity of the eccentric bond pullout specimens of this study by considering the influence of embedment length, concrete compressive strength, bar diameter, and the amount of concrete width per bar (b/ND), Sec. 6.6. Since no binding reinforcement was used, its effect was not included in the regression analysis which follows. It would be desirable if the influence of bar surface corrosion on ultimate load could be incorporated in the ultimate load expression. However, the results were inconclusive in that they did not exhibit a specific trend. Therefore, the contribution of surface rust (ill or favorable) to ultimate load was not considered.

A multiple linear regression analysis of the bond data indicated that the following equation best fitted the ultimate bond strength of the specimens of this investigation:

$$P_u = 29.3\ (L''\sqrt{f'_c}\,) + 15{,}256\ (D)^2 - 460\left[NDL''\left(\frac{b}{ND}\right)\right]^{0.5962} \qquad (6.1)$$

The coefficient of correlation was 92, 92, and 70 percent for the first, second, and the third term, respectively. The coefficient of multiple

TABLE 6.3

COMPARISON OF TEST ULT. STRENGTH WITH CALCULATED

ULT. STRENGTH OF BOND SPECIMNS

Bar Size		16 single-bar specimens per bar size.			4 double spaced-bar specimen per bar size.			3 adjacent-bar specimens per bar size.		
		No. of Specimens	Percent of Tot. Spec.	$T_{cal} - T_{adj.}$ % of $T_{cal.}$	No. of Specimens	Percent of Tot. Spec.	$T_{cal} - T_{adj.}$ % of $T_{cal.}$	No. of Specimens	Percent of Tot. Spec.	$T_{cal} - T_{adj.}$ % of $T_{cal.}$
No.4	$T_{adj.} < T_{cal.}$	15	94	20	1	25	8	-	-	-
No.4	$T_{adj.} > T_{cal.}$	1	6	2	3	75	34	3	100	9
No.6	$T_{adj.} < T_{cal.}$	14	88	13	1	25	1	3	100	18
No.6	$T_{adj.} > T_{cal.}$	2	12	5	3	75	9	-	-	-
No.8	$T_{adj.} < T_{cal.}$	9	56	12	2	50	11	1	33	11
No.8	$T_{adj.} > T_{cal.}$	7	44	5	2	50	55	2	67	11
No.10	$T_{adj.} < T_{cal.}$	4	25	5	-	-	-	-	-	-
No.10	$T_{adj.} > T_{cal.}$	12	75	19	4	100	35	3	100	37
No.11	$T_{adj.} < T_{cal.}$	4	25	7	-	-	-	-	-	-
No.11	$T_{adj.} > T_{cal.}$	12	75	18	4	100	33	3	100	42

correlation was 95 percent. The above equation was derived from the corrected ultimate loads for a concrete compressive strength of 3,000 psi (Table A.3). As a check on the validity of equation 6.1 the actual concrete compressive strengths of all the specimens and the other parameters were substituted in the equation. The results are tabulated in Table 6.4 along with the ultimate loads predicted by Lutz's (32) ultimate load equation for specimens with no web reinforcement and the equation derived by Lutz with Ferguson's (10) beam data. In general, the ultimate strengths given by these equations are lower than the ultimate loads predicted by equation 6.1. Equation 6.2 is Ferguson's ultimate load equation and equation 6.3 is Lutz's expression for ultimate load.

$$P_u = 66.67L''b + 17,000 \qquad (6.2)$$

$$P_u = 19.1\,L''\sqrt{f'_c} + 3,400\,D^2 \qquad (6.3)$$

TABLE 6.4

ULTIMATE BOND STRENGTH FROM EQUATIONS 6.1, 6.2, & 6.3

NO WEB REINFORCEMENT

Ultimate Load per Specimen (kips)														
Spec. No.	Test	Eq. 6.1	Eq. 6.2	Eq. 6.3	Spec. No.	Test	Eq. 6.1	Eq. 6.2	Eq. 6.3	Spec. No.	Test	Eq. 6.1	Eq. 6.2	Eq. 6.3
4A0-a	12	15	11	12	6N6-c	16	15	18	14	8M12c	30	30	26	24
-b	14	14	11	11	-d	18	15	18	14	d	34	30	26	24
-c	12	11	11	11	-e	18	15	18	14	8N12c	34	28	26	23
6A0-a	25	24	18	17	8M6-a	37	31	26	21	d	28	28	26	23
-b	20	24	18	17	-b	32	30	26	20	e	29	31	26	25
-c	21	20	18	17	c	29	27	26	22	10M12a	48	49	35	31
8A0-a	31	36	26	24	d	29	27	26	22	b	46	49	35	31
b	29	36	26	24	8N6-c	27	26	26	21	c	41	40	35	30
c	35	32	26	25	-d	28	26	26	21	d	41	40	35	30
10A0-a	55	51	35	33	-e	34	31	26	25	10N12c	57	41	35	30
-b	55	42	35	26	10M6-a	49	46	35	29	d	50	41	35	30
-c	49	44	35	32	-b	48	46	35	29	e	40	44	35	32
11A0-a	54	60	40	36	-c	50	45	35	33	11M12a	58	59	40	36
-b	72	60	41	37	-d	41	45	35	33	b	53	59	40	36
-c	62	48	40	34	10N6-c	50	43	35	32	c	54	52	40	37
4N3-c	10	12	11	12	-d	46	43	35	32	d	52	52	40	37
d	11	12	11	12	-e	62	46	35	34	11N12c	57	55	40	38
e	12	10	11	10	11M6-a	55	61	40	37	d	58	55	40	38
6N3-c	19	19	18	17	-c	52	50	40	35	e	60	52	40	36
-d	15	15	18	14	-d	53	50	40	35	4S3-a	18	14	11	11
-e	15	15	18	14	11N6-c	48	54	40	38	c	8	12	11	11
8N3-c	28	31	26	25	-d	56	54	40	38	d	8	12	11	12
-d	24	31	26	25	-e	50	51	40	36	6S3-a	27	25	18	18
-e	25	30	27	26	4M12a	16	14	11	11	-c	27	20	18	18
10N3-c	43	46	35	34	b	12	14	11	11	-d	23	20	18	18
-d	46	45	35	33	c	11	14	11	13	8S3-a	52	34	26	23
11N3-c	56	51	40	36	d	12	14	11	13	-c	29	28	26	23
-d	46	52	40	37	4N12c	12	14	11	13	-d	34	28	26	23
4M6-a	14	12	11	10	d	11	11	11	11	10S3-a	61	48	34	30
-b	12	12	11	10	e	12	14	11	13	-c	50	50	34	31
-c	10	12	11	12	6M12a	27	23	18	17	-d	48	50	34	31
-d	10	9	11	10	b	18	23	18	17	11S3-a	82	61	40	37
4N6-c	10	12	11	12	c	24	25	18	20	-c	45	52	40	37
-d	10	12	11	11	d	29	25	18	20	-d	51	52	40	37
-e	10	11	11	11	6N12c	27	25	18	20					
6M6-a	23	23	18	17	d	16	19	18	17					
-b	20	23	18	17	e	23	19	18	17					
-c	22	19	18	17	8M12a	27	35	26	24					
-d	22	19	18	17	b	32	35	26	24					

A=as-rolled, N=indoor
M=outdoor, S=sea water
a=double spaced bars
b=double adjacent "
c,d,e= single bar

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

The purpose of this investigation was to obtain fundamental information on the influence of corrosion and bar spacing on bond properties of intermediate grade reinforcing bars. An experimental program was conducted on 115 eccentric bond pullout specimens. The specimens were subjected to monotonic load to collapse. The type of loading used, Fig. 3.1, was devised to obtain a bond condition undisturbed by load points, reactions or stirrups. The specimens were cast with single bar, two bars spaced three bar diameters, and two bars adjacent-tied (Sec. 3.2.1).

The following conclusions were derived from the analysis of the test results:

7.1.1 Bond Stress Versus Surface Corrosion

1. As compared to unexposed bars, and with the exception of Nos. 6,8,and 10 sea-water corroded bars, the results indicated some reduction in the ultimate bond strength of the single-bar specimens cast with 3-,6-, and 12-months-outdoor and indoor corroded bars. However, due to the limited number of specimens tested in this study, the quantitative values of the loss of bond strength should be

considered preliminary.

2. No trend was indicated for double-spaced-bar specimens and double-adjacent-tied-bar specimens containing the 6- and 12-months rusted bars.

3. Double-spaced-bar specimens with 3-months sea-water rusted bars failed at higher loads than the specimens with as-rolled bars.

7.1.2 Effects of Corrosion on Reinforcing Bars

1. As compared to the as-rolled bars, the ultimate tensile strength of indoor and outdoor corroded bars was not affected by the different degrees of surface corrosion studied.

2. The ultimate tensile strength of 3-months sea-water rusted bars was reduced by an average of 3.7 percent.

3. There was no substantial change in the deformation height, bar diameter, and bar length due to corrosion.

4. Although a slight reduction in the weights of control coupons (indoor and outdoor corroded) was observed, the polished weights were much higher than the 94 percent of the theorectical weight limitation set by ASTM standards. The polished weights of sea-water rusted bars were between 94.7 and 97.7 percent of the theoretical weight as bar size increased from No. 4 to No. 11.

7.1.3 Bond Stress Versus Bar Spacing

1. The ultimate resistance of closely spaced bars was slightly lower than the spaced bars.

2. The mode of failure of the specimens changed with bar spacing. The specimens with two adjacent-tied bars failed when a longi-

tudinal crack formed on the top and progressed along the bar to the free end of the bar. The failure of the double-spaced-bar specimens, however, was as a result of longitudinal cracks on the sides of the specimen at the level of the bars.

3. For concrete compressive strength of 3,000 psi and no web reinforcement, bond stresses calculated up to the maximum indicated by the shear limitation of $3.5\sqrt{f'_c}$ set by the 1963 ACI Code should be the only limitation upon the spacing of bars, other than that imposed by the size of the aggregates (Sec. 6.6).

7.1.4 Bond Stress Versus Slip

A. Loaded End

1. Half of the single-bar specimens tested failed at ultimate loaded-end slips of less than 0.01 inch and the other half failed at slips greater than 0.01 inch (0.01 inch slip has been assumed by bond investigators as a crack control measure).

2. For specimens with double spaced and double adjacent bars, the observed ultimate loaded-end slip was 0.001 inch or less in slightly over half of the tests and 0.010 inch or less in all the tests.

3. Loaded-end slip at failure decreased with increase in bar diameter.

4. The single-bar specimens containing the 3-months sea-water corroded bars failed at slightly higher loaded-end slip than the remainder of the specimens.

5. No trend of influence of surface corrosion on loaded-end slip was observed.

B. Free End

1. For the specimens with single bar, double spaced bars, and double adjacent-tied bars, the free-end slip was 0.0005 inch or less in 68, 95, and 87 percent of the tests, respectively (outdoor and indoor corroded bars).

2. The specimens with 3-months sea-water rusted bars developed considerably higher free-end slip than specimens with other surface corrosion.

7.1.5 Ultimate Bond Strength

1. The ultimate bond strength of the eccentric bond pullout specimens of this investigation, with no web reinforcement, could be expressed as:

$$P_u = 29.3\ (L''\sqrt{f'_c}) + 15{,}256\ D^2 - 460\left[DL''\ln\left(\frac{b}{ND}\right)\right]^{0.5962}$$

with a coefficient of multiple correlation of 95 percent. (P_u =load per specimen, pounds; L"= embedment length, inches; f'_c= concrete compressive strength, psi; D= bar diameter, inches; b= specimen's width, inches, N = number of bars per specimen).

2. The presence of web reinforcement was found to increase the ultimate bond strength to a significant degree (Sec. 6.7). However, due to the limited number of specimens tested with web reinforcement, this effect was not included in the ultimate bond strength equation.

7.2 Recommendations

This investigation, due to the limited number of tests and the variety of variables, did not result in a conclusive and unique solution to the problem of corrosion on reinforcing bars, its effects on bond properties of bars, and the influence of bar spacing on the ultimate strength. However, it did result in a comprehensive understanding of the problems involved.

The following recommendations are suggested for future research:

1. The effect of corrosion on bond strength of reinforcing bars needs to be investigated for long duration of exposure to corrosive environments. It is the author's belief that, since the load carrying capacity of the modern deformed bars is primarily dependent on lug action (bearing of lugs upon concrete), if reinforcing bars are exposed to a corrosive environment (such as sea water) for a long period of time the lug height will be considerably reduced due to corrosion, thereby impairing the bond strength of such bars.

2. The influence of a wide range of the variable $\frac{b}{ND}$ ratio on bond strength of eccentric bond pullout specimens deserves further research.

3. Knowledge of the actual splitting forces developed by deformed bars and the resistance of concrete members to splitting forces are needed if an adequate bond theory is to be established.

4. Influence of different sizes of web reinforcement on bond strength, with various stirrups spacings, requires further study.

5. The effect of two or more rows of tensile reinforcement on bond strength has yet to be explored.

BIBLIOGRAPHY

1. Ferguson, P. M., et al., "Development Length of High Strength Reinforcing Bars," ACI Journal, Vol. 49, pp. 887, 1962.

2. Mathey, R. G., et al., " Investigation of Bond in Beam and Pullout Specimens with High-Yield-Strength Deformed Bars," ACI Journal, Vol. 57, pp. 1071, 1961.

3. Hadley, H. M., "Tests of Beams Reinforced with 'Bundled bars'," Civil Engineering, Vol. 11, No. 2, pp. 90-93, February 1941.

4. Hadley, H. M., "Bundled Reinforcement," ACI Journal, Vol. 49, pp. 157-159, October 1952.

5. Boase, A. J., "Brazil's Wonder Hotel and Casico," Engineering News-Record,Vol. 136, No. 2, pp. 112-116, January 10, 1946.

6. Anon, "Bundle Reinforcing Saves Materials," Engineering News-Record, Vol. 140, No. 14, pp. 509-510, April 1, 1948.

7. Walker, W. T., "Laboratory Tests of Spaced and Tied Reinforcing Bars," ACI Journal, Vol. 47, No. 5, pp. 365-372, January 1951.

8. Hanson, N. W., et al., "Concrete Beams and Columns with Bundled Reinforcement," Journal of the Structural Division, Proceedings, ASCE, Paper 1818, Vol. 84, No. ST6, pp. 1818-1 , 1818-23, October 1958.

9. Steiner, F. D., "Suggested Applications for Bundled Bars," ACI Journal, Proceedings Vol. 64, pp. 213-214, April 1967.

10. Ferguson, P. M., et al., "Minimum Bar Spacing as a Function of Bond and Shear Strength," ACI Journal, Proceedings Vol. 50, pp. 869-887, June 1954.

11. Chamberlin, S. J., et al., "Bond Between Concrete and Steel and the Spacing of Reinforcement," Iowa Engineering Experiment Station, Report No. 26, 1955-56.

12. Yee, B. L., "A Comparison of the Bond Strength of Bundled Bars and Spaced Bars when Embedded in Concrete," A.M.S.E. Thesis, University of Oklahoma, pp. 35, 1965.

13. ACI Committee 408, "Bond Stress- The State of the Art," Vol. 63, pp. 1161-1188, ACI Journal, November 1966.

14. Baldwin, J. W., "Bond of Reinforcement in Lightweight Aggregate Concrete," Preliminary Report, University of Missouri, March 1965.

15. Withey, M. O., "Tests on Bond Between Concrete and Steel in Reinforced Concrete Beams," Bulletin 321, University of Wisconsin, Engineering Series, Vol. 5, No. 5, 1909.

16. Abrams, D. A., "Tests on Bond Between Concrete and Steel," Bulletin 71, University of Illinois, Experiment Station, 1913.

17. Shank, J. E., "Effect of Bar Surface Conditions in Reinforced Concrete," Ohio State Engineering Experiment News, p. 9, June 1934.

18. Gilkey, H. J., et al., "Bond Tests on Rusted Bars," Proceeding Highway Research Board, Vol. 19, pp. 149-163, 1939.

19. Johnston, B. and Cox, K. C., "The Bond Strength of Rusted Deformed Bars," Proceedings ACI, pp. 57-72, September 1940.

20. Kemp, E. L., et al., "Effect of Loose Rust and Mill Scale on the Bond Characteristics of Deformed Reinforcing Bars," ACI Journal, No. 12 Proceedings Vol. 64, p. N24, December 1967.

21. Perry, E. S. and J. N. Thompson, "Bond Stress Distribution on Reinforcing Steel in Beams and Pullout Specimens," ACI Journal, Proceedings Vol. 63, No. 8, pp. 865-874, August 1966.

22. Bryson, J. O., and Mathey, R. G., "Surface Condition Effect on Bond Strength of Steel Beams Embedded in Concrete," ACI Journal, Proceedings Vol. 59, No. 3, pp. 397-405, March 1962.

23. A Manual for the Control of Concrete Construction, U.S. Department of the Interior, Bureau of Reclamation, 6th Edition, Revised Reprint, 1956.

24. Abe, K., "Effect of Surface Conditions of Wire on Bond," Proceedings of the Symposium on Prestressed Concrete and Composite Beams, pp. 22-26, November 1955.

25. Janney, J. R., "Nature of Bond in Pre-Tensioned Prestressed Concrete," ACI Journal, Vol. 25, No. 9, pp. 725-736, May 1954.

26. Shermer, C. L. "Corroded Reinforcement Destroys Concrete Beams," Civil Engineering, Vol. 26, No. 12, pp. 56-57, December 1956.

27. Evans, U. R., An Introduction to Metallic Corrosion, Edward Arnold Publishers Ltd., London, 1948.

28. Ferguson, P. M., Reinforced Concrete Fundamentals, John Willy & Sons Inc., New York, 1965.

29. Portland Cement Association, "Design and Control of Concrete Mixtures," PCA Publication, 10th Edition, 1952.

30. Untrauer, R. E. and Henry, R. L., "Influence of Normal Pressure on Bond Strength," ACI Journal, Vol. 62, No. 5, pp. 577-586, May 1965.

31. Neville, A. M., "A General Relation for Strengths of Concrete Specimens of Different Shapes and Sizes," ACI Journal, Vol. 63, No. 10, pp. 1095-1109, October 1966.

32. Lutz, L. A., "The Mechanics of Bond and Slip of Deformed Reinforcing Bars in Concrete," Report No. 324, Cornell University, School of Civil Engineering, August 1966.

33. Ferguson, P. M., and Thompson, J. N., "Development Length for Large High Strength Reinforcing Bars," ACI Journal, Vol. 62, p. 71, January 1965.

34. ASTM Specifications for Steel Bars for Concrete Reinforcement," American Society for Testing and Materials, October 1964.

35. ACI Building Code 1963, American Concrete Institute, Detroit, Mich., 1963.

APPENDIX A

COMPUTER PROGRAM FOR MULTIPLE LINEAR REGRESSION ANALYSIS

MAIN IBM-360

```
      DIMENSION XBAR(40),STD(40),D(40),RY(40),ISAVE(40),B(40),SB(40),T(40),W(40)
      DIMENSION RX(1600),RX(820),ANS(10)
    1 FORMAT (A4,A2,I5,2I2)
    2 FORMAT (25H1MULTIPLE REGRESSION.....A4,A2//6X,14HSELECTION.....I2//.)
    3 FORMAT (9HOVARIABLE,5X,4HMEAN,6X,8HSTANDARD,6X,11HCORRELATION,4X,
     110HREGRESSION,4X,10HSTD. ERROR,5X,8HCOMPUTED/6H   NO.,18X,9HDEVIATION,
     27X,6HX VS Y,7X,11HCOEFFICIENT,3X,12HOF REG.COEF.,3X,7HT VALUE)
    4 FORMAT (1H, I4,6F14.5)
    5 FORMAT (10H DEPENDENT)
    6 FORMAT (1HO/10H INTERCEPT,13X,E13.6//23H MULTIPLE CORRELATION   ,F13.5//
     123H STD. ERROR OF ESTIMATE, F13.5//)
    7 FORMAT (1HO,21X,39HANALYSIS OF VARIANCE FOR THE REGRESSION//5X,19HS
     1OURCE OF VARIATION,7X,7HDEGREES,7X,6HSUM OF, 10X,4HMEAN,12X,7HF VAL
     2UE/30X,10HOF FREEDOM,4X,7HSQUARES,9X,7HSQUARES)
    8 FORMAT (30H ATTRIBUTABLE TO REGRESSION    ,I6,3E16.7/30H DEVIATION F
     1ROM REGRESSION     ,I6,2E16.7)
    9 FORMAT (1H ,5X,5HTOTAL,19X,I6,E16.7)
   10 FORMAT (36I2)
   11 FORMAT (1H, 15X,18HTABLE OF RESIDUALS//9H CASE NO.,5X,7HY VALUE,5X,
     110HY ESTIMATE,6X,8HRESIDUAL)
   12 FORMAT (1H ,I6,F15.5,2F14.5)
   13 FORMAT (53H1NUMBER OF SELECTIONS NOT SPECIFIED.  JOB TERMINATED.)
   14 FORMAT (52H)THE MATRIX IS SINGULAR.  THIS SELECTION IS SKIPPED.)
  100 READ (1,1) PR,PR1,N,M,NS
      REWIND 8
      IO=0
      X=0.0
      CALL CORRE (N,M,IO,X,XBAR,STD,RX,R,D,B,T)
      REWIND 8
      IF (NS) 108,108,109
  108 WRITE (3,13)
      GO TO 300
  109 DO 200 I=1,NS
      WRITE (3,2) PR,PR1,I
      READ (1,10) NREST,NDEP,K,(ISAVE(J),J=1,K)
      CALL ORDER (M,R,NDEP,K,ISAVE,RX,RY)
      IF(DET) 112,110, 112
  110 WRITE (3,14)
      GO TO 200
  112 CALL MULTR (N,K,XBAR,STD,D,RX,RY,ISAVE,B,SB,T,ANS)
      MM=K+1
      WRITE (3,3)
      L=ISAVE(MM)
```

```
      WRITE (3,4) L.XBAR(L),STD(L)
      WRITE (3,6) ANS(I),ANS(2),ANS(3)
      WRITE (3,7)
      L=ANS(8)
      WRITE (3,8) K,ANS(4),ANS(6),ANS(10),L,ANS(7),ANS(9)
      L=N-1
      SUM=ANS(4)+ANS(7)
      WRITE (3,9) L,SUM
      IF(NRESI) 200, 200, 120
  120 WRITE (3,2) PR,PR1,I
      WRITE (3,11)
      MM=ISAVE(K+1)
      DO 140 II=1,N
      READ (8) (W (J), J=1,M)
      SUM=ANS(1)
      DO 130 J=1,K
      L=ISAVE(J)
  130 SUM=SUM+W(L)*B(J)
      RESI=W(MM) - SUM
  140 WRITE (3,12) II,W(MM),SUM,RESI
      REWIND 8
  200 CONTINUE
      GO TO 100
  300 CONTINUE
      END

      SUBROUTINE DATA (M,D)
      DIMENSION D(1)
      N=11
      READ (1,100) (D(I), I=1,N)
  100 FORMAT (6F10.0/5F10.0)
      D(12)=(D(6)*D(1))/D(9)
      D(13)=(D(12))**0.5962
      WRITE (8) (D(I), I=1,M)
      RETURN
      END
```

TABLE A.1

DIMENSIONS AND WEIGHT OF NO.4 AS-ROLLED BARS

Bar No.	Length (in.)	Lug Spacing (in.)	Lug Height (in.)	Weight W_i (gr.)	Bar No.	Length (in.)	Lug Spacing (in.)	Lug Height (in.)	Weight W_i (gr.)
1	6.025	0.225	0.024	156.30	44	5.990	0.222	0.029	156.15
2	6.062	.225	.026	155.50	45	5.995	.222	.024	155.60
3	6.010	.225	.028	155.60	46	6.077	.222	.026	155.70
4	5.893	.225	.021	152.07	47	6.090	.222	.027	158.69
5	6.030	.225	.024	155.21	48	5.754	Avg.	Avg.	144.80
6	5.915	.225	.022	154.21	49	5.982	0.223	0.026	155.32
7	5.795	.225	.021	150.70	50	6.021			156.09
8	5.843	.225	.021	147.09	51	5.963			152.20
9	5.920	.225	.026	151.22	52	5.963			155.90
10	5.925	.225	.029	154.20	53	5.762			150.70
11	6.035	.225	.027	156.60	54	5.732			146.92
12	5.995	.225	.026	154.18	55	5.856			150.48
13	6.130	.225	.027	158.43	56	5.778			151.81
14	6.147	.225	.028	156.63	57	5.775			151.62
15	6.108	.225	.027	160.62	58	5.897			151.13
16	6.125	.225	.028	155.43	59	6.104			158.51
17	5.810	.225	.026	152.24	60	5.763	Avg.		150.43
18	5.995	.225	.025	154.70	61	6.003	Length=6.001"		157.26
19	6.131	.225	.028	157.11	62	5.987			152.73
20	6.077	.225	.028	155.50	63	5.983			150.43
21	5.880	.225	.027	148.90	64	5.888			153.50
22	5.958	.225	.025	150.03	65	5.952	Avg.		155.58
23	5.836	.225	.027	149.61	66	6.031	Weight=154.60		158.10
24	6.050	.225	.025	157.35	67	5.962	(gr.)		154.35
25	6.010	.225	.025	156.35	68	5.963			149.80
26	6.215	.225	.027	162.92	69	5.867			150.25
27	6.100	.225	.025	153.41	70	5.878			152.55
28	5.850	.222	.026	150.34	71	5.905			152.21
29	6.110	.222	.027	157.23	72	5.880			151.90
30	6.212	.222	.028	160.96	73	6.078			156.75
31	5.997	.222	.027	155.12	74	5.810			152.39
32	6.155	.222	.024	157.20	75	6.216			161.30
33	6.185	.222	.029	159.07	76	5.955			155.05
34	6.022	.222	.027	156.03	77	6.028			159.90
35	6.040	.222	.027	158.71	78	6.100			158.20
36	6.000	.222	.024	150.78	79	5.942			153.20
37	5.965	.222	.026	150.35	80	6.012			156.69
38	5.995	.222	.031	154.31	81	6.155			158.21
39	6.000	.222	.026	156.07	82	6.075			153.61
40	5.987	.222	.028	153.50	83	6.050			159.36
41	5.960	.222	.024	150.18	84	6.205			161.48
42	5.982	.222	.026	154.76	85	6.015			158.11
43	6.172	.222	.024	161.88	86	6.070			156.03

TABLE A.1 (con't)

DIMENSIONS AND WEIGHT OF NO. 6 AS-ROLLED BARS

Bar No.	Length (in.)	Lug Spacing (in.)	Lug Height (in.)	Weight W_i (gr.)	Bar No.	Length (in.)	Lug Spacing (in.)	Lug Height (in.)	Weight W_i (gr.)
1	5.917	0.310	0.050	328.18	44	5.943	0.310	0.047	337.09
2	5.850	.310	.043	334.65	45	5.998	.310	.050	339.58
3	5.925	.310	.046	332.61	46	5.938	.310	.048	337.11
4	6.042	.310	.050	342.83	47	5.940	.310	.048	335.30
5	5.9.9	.310	.050	334.28	48	5.955	Avg.	Avg.	338.40
6	5.987	.310	.048	338.45	49	5.915	0.310	0.048	332.59
7	6.008	.310	.050	343.00	50	5.980			334.62
8	5.925	.310	.049	332.55	51	5.915			333.10
9	6.025	.310	.048	341.20	52	6.015			339.50
10	6.028	.310	.045	338.53	53	5.990			337.59
11	5.946	.310	.049	333.00	54	5.947			337.10
12	5.915	.310	.052	335.46	55	5.968			337.70
13	5.982	.310	.048	339.50	56	5.948			335.63
14	5.988	.310	.049	338.12	57	6.000			339.68
15	6.046	.310	.050	341.55	58	6.040			340.42
16	5.880	.310	.049	332.21	59	5.928			333.80
17	6.000	.310	.050	339.70	60	5.900			331.05
18	5.900	.310	.047	329.44	61	5.997	Avg.		339.61
19	6.062	.310	.052	341.43	62	6.000	Length= 5.961"		339.30
20	6.045	.310	.046	339.32	63	5.940			334.98
21	5.0877	.310	.053	332.70	64	6.078			340.85
22	6.100	.310	.050	341.95	65	5.840			327.72
23	5.945	.310	.051	332.96	66	5.935			335.07
24	5.955	.310	.053	340.05	67	6.017	Avg.		341.09
25	5.958	.310	.046	335.29	68	5.892	Weight= 336.31		329.42
26	6.033	.310	.050	339.05	69	5.972	(gr.)		340.73
27	5.961	.310	.050	336.31	70	5.940			336.80
28	6.025	.310	.047	340.17	71	5.970			336.52
29	6.035	.310	.045	341.28	72	6.020			339.23
30	5.975	.310	.047	339.81	73	6.000			338.65
31	5.872	.310	.043	325.56	74	6.040			341.40
32	6.047	.310	.042	341.47	75	6.012			337.78
33	5.950	.310	.048	335.98	76	6.010			337.65
34	5.928	.310	.052	336.73	77	5.855			332.97
35	5.838	.310	.050	331.27	78	5.975			335.40
36	5.965	.310	.046	331.23	79	5.950			336.73
37	5.950	.310	.042	333.49	80	6.028			341.21
38	5.900	.310	.051	332.40	81	5.890			330.25
39	6.170	.310	.050	341.39	82	6.056			340.58
40	5.975	.310	.053	335.40	83	5.980			340.78
41	6.102	.310	.048	346.35	84	6.020			338.13
42	5.925	.310	.043	334.32	85	5.916			333.35
43	6.135	.310	.045	345.89					

TABLE A.1 (con't)

DIMENSIONS AND WEIGHT OF NO. 8 AS-ROLLED BARS

Bar No.	Length (in.)	Lug Spacing (in.)	Lug Height (in.)	Weight W_i (gr.)	Bar No.	Length (in.)	Lug Spacing (in.)	Lug Height (in.)	Weight W_i (gr.)
1	5.940	0.410	0.062	598.15	44	5.900	0.417	0.075	585.65
2	6.100	.410	.062	603.55	45	5.948	.417	.063	598.31
3	6.058	.410	.072	616.50	46	5.940	.417	.066	588.56
4	5.967	.410	.059	593.75	47	6.050	.417	.072	602.48
5	5.908	.410	.065	594.65	48	5.930	Avg.	Avg.	598.89
6	5.871	.410	.060	584.50	49	5.930	0.413	0.067	596.00
7	5.950	.410	.058	600.88	50	5.896			604.33
8	5.853	.410	.067	580.95	51	5.983			599.52
9	5.972	.410	.070	590.60	52	5.900			583.26
10	5.908	.410	.070	598.70	53	5.950			596.75
11	5.988	.410	.068	588.29	54	5.945			594.80
12	5.961	.410	.066	595.17	55	6.112			612.70
13	5.883	.410	.067	590.40	56	6.026			602.25
14	5.833	.410	.070	590.45	57	5.917			602.30
15	6.035	.410	.067	598.29	58	5.927			595.19
16	5.945	.410	.070	599.52	59	5.890			594.30
17	5.937	.410	.070	599.40	60	5.997			593.42
18	5.984	.410	.069	593.78	61	6.060	Avg.		610.95
19	6.022	.410	.070	600.62	62	5.950	Length= 5.958"		602.52
20	6.000	.410	.068	600.85	63	6.050			600.78
21	5.920	.410	.068	595.82	64	5.947			586.83
22	5.980	.410	.070	600.98	65	6.000			605.70
23	5.972	.410	.065	593.81	66	5.960			606.48
24	5.912	.410	.072	599.95	67	5.960	Avg.		597.73
25	5.995	.410	.066	590.93	68	5.885	Weight= 597.29		595.13
26	5.925	.410	.071	598.22	69	5.850	(gr.)		589.71
27	6.026	.410	.070	600.50	70	5.923			589.80
28	5.894	.417	.070	593.21	71	5.810			581.80
28	5.873	.417	.071	588.39	72	5.997			595.74
30	5.900	.417	.062	586.28	73	5.992			600.44
31	5.961	.417	.068	602.38	74	6.065			607.65
32	6.038	.417	.065	600.42	75	5.918			586.73
33	5.970	.417	.065	600.80	76	5.949			594.51
34	5.960	.417	.072	607.26	77	5.880			591.50
35	5.937	.417	.061	589.76	78	6.032			611.68
36	5.922	.417	.060	595.92	79	5.990			599.39
37	5.946	.417	.070	595.18	80	5.912			597.86
38	5.935	.417	.060	595.45	81	6.000			605.56
39	6.040	.417	.061	602.36	82	5.927			597.37
40	6.062	.417	.066	602.78	83	5.935			601.43
41	6.080	.417	.063	600.55	84	5.860			594.62
42	6.000	.417	.075	607.78	85	5.897			586.61
43	5.970	.417	.076	609.36	86	5.977			605.97

TABLE A.1 (con't)

DIMENSIONS AND WEIGHT OF NO. 10 AS-ROLLED BARS

Bar No.	Length (in.)	Lug Spacing (in.)	Lug Height (in.)	Weight W_i (gr.)	Bar No.	Length (in.)	Lug Spacing (in.)	Lug Height (in.)	Weight W_i (gr.)
1	5.940	0.500	0.092	943.83	44	6.247	0.500	0.086	985.35
2	5.920	.500	.084	935.73	45	5.880	.500	.089	935.65
3	6.100	.500	.088	966.00	46	5.968	.500	.086	931.61
4	5.918	.500	.094	934.05	47	6.047	.500	.087	947.41
5	5.988	.500	.082	942.40	48	5.868	Avg.	Avg.	933.42
6	6.148	.500	.071	982.35	49	5.915	0.500	0.087	934.41
7	5.988	.500	.093	946.58	50	5.913			936.80
8	5.989	.500	.094	943.10	51	5.962			948.29
9	5.958	.500	.094	943.10	52	6.040			952.70
10	5.855	.500	.085	910.39	53	5.970			942.83
12	5.997	.500	.093	950.67	54	5.875			928.08
13	6.016	.500	.093	945.76	55	5.970			943.12
14	6.049	.500	.088	948.48	56	6.030			957.95
15	5.950	.500	.093	943.84	57	5.833			919.23
16	5.881	.500	.083	920.68	58	5.995			948.40
17	6.016	.500	.090	947.83	59	5.825			929.60
18	5.877	.500	.090	926.70	60	6.060	Avg.		967.34
19	5.916	.500	.087	932.49	61	5.860	Length= 5.954"		930.88
20	5.932	.500	.084	933.96	62	6.020			942.33
21	5.804	.500	.092	919.55	63	5.938			942.00
22	5.802	.500	.084	902.72	64	5.895			946.30
23	5.925	.500	.093	938.46	65	5.875			929.55
24	5.964	.500	.083	936.97	66	5.992	Avg.		950.32
25	5.922	.500	.083	919.30	67	6.130	Weight= 939.87		965.55
26	5.945	.500	.081	917.80	68	5.865	(gr.)		916.83
27	5.830	.500	.092	923.79	69	5.990			945.85
28	6.064	.500	.095	962.38	70	5.832			927.80
29	5.963	.500	.090	944.10	71	5.977			940.00
30	5.823	.500	.085	921.72	72	5.947			938.53
31	5.978	.500	.085	944.05	73	5.910			939.58
32	6.035	.500	.086	951.23	74	6.047			964.63
34	6.030	.500	.086	951.04	75	5.920			935.85
35	5.956	.500	.091	946.19	76	5.790			902.41
36	5.893	.500	.087	931.55	77	5.965			943.76
37	5.928	.500	.089	933.00	78	5.948			942.55
38	5.888	.500	.087	928.85	79	5.868			929.12
39	5.968	.500	.085	946.18	80	5.890			935.10
40	5.962	.500	.086	938.64	81	5.950			936.20
41	5.903	.500	.087	931.02	82	5.980			941.21
42	5.855	.500	.084	919.86	83	5.990			926.18
43	6.078	.500	.089	966.04	84	6.130			969.65
44	6.040	.500	.087	954.85	85	5.930			937.90
45	5.940	.500	.084	926.51					

TABLE A.1 (con't)

DIMENSIONS AND WEIGHT OF NO. 11 AS-ROLLED BARS

Bar No.	Length (in.)	Lug Spacing (in.)	Lug Height (in.)	Weight W_i (gr.)	Bar No.	Length (in.)	Lug Spacing (in.)	Lug Height (in.)	Weight W_i (gr.)
1	6.143	0.540	0.076	1189.14	44	5.949	0.530	0.066	1179.71
2	5.947	.540	.074	1178.48	45	6.037	.530	.068	1192.51
3	5.955	.540	.072	1181.46	46	6.075	.530	.070	1189.90
4	5.858	.540	.080	1154.47	47	6.133	.530	.067	1203.56
5	6.137	.540	.075	1212.75	48	5.900	Avg.	Avg.	1185.63
6	6.017	.540	.077	1185.69	49	5.940	0.536	0.072	1180.48
7	5.927	.540	.077	1173.75	50	6.005			1192.50
8	6.043	.540	.076	1184.15	51	5.832			1148.80
9	5.853	.540	.076	1156.08	52	5.945			1183.78
10	5 825	.540	.080	1168.40	53	5.865			1168.24
11	5.913	.540	.079	1181.60	54	6.030			1198.18
12	5.005	.540	.068	1187.42	55	5.980			1157.13
13	6.859	.540	.075	1174.01	56	5.950			1177.15
14	5.861	.540	.075	1162.83	57	6.022			1196.06
15	5 989	.540	.077	1193.79	58	5.862			1160.61
16	5.906	.540	.078	1167.72	59	5.808			1144.72
17	5.997	.540	.075	1192.62	60	5.757			1149.39
18	6.050	.540	.075	1197.33	61	6.008	Avg.		1186.10
19	5.931	.540	.065	1168.40	62	5.930	length= 5.946"		1176.10
20	5.872	.540	.065	1152.00	63	5.965			1181.89
21	6.050	.540	.064	1189.02	64	5.982			1197.52
22	6.085	.540	.065	1192.25	65	5.900			1179.80
23	6.023	.540	.068	1192.58	66	5.845			1171.05
24	5.978	.540	.064	1178.63	67	5.827	Avg.		1162.21
25	5.925	.540	.069	1168.29	68	6.038	Weight=1178.72		1191.30
26	5.917	.540	.069	1170.00	69	6.022	(gr.)		1211.32
27	6.012	.540	.066	1176.78	70	6.045			1208.29
28	6.020	.530	.073	1199.18	71	6.012			1204.70
29	5.965	.530	.067	1178.91	72	6.070			1202.20
30	5.840	.530	.079	1165.10	73	6.160			1212.55
31	5.753	.530	.074	1148.00	74	5.927			1183.13
32	5.932	.530	.074	1186.09	75	5.875			1165.80
33	6.005	.530	.065	1175.80	76	6.078			1207.29
34	5.916	.530	.077	1168.24	77	5.808			1161.58
35	6.075	.530	.079	1206.72	78	5.883			1162.50
36	5.795	.530	.067	1143.22	79	5.910			1164.26
37	5.903	.530	.070	1161.16	80	5.840			1171.81
38	5.978	.530	.069	1191.56	81	5.940			1187.99
39	5.895	.530	.075	1164.61	82	6.012			1184.50
40	5.915	.530	.070	1169.19	83	6.037			1202.58
41	5.940	.530	.076	1184.41	84	5.945			1192.90
42	5.800	.530	.073	1160.34	85	5.995			1180.58
43	5.641	.530	.075	1122.20	86	5.808			1159.12

TABLE A.2

THE AMOUNT OF MILL SCALE ON REINFORCING BARS

Bar No.	L (in)	D_1 (in.)	H_1 (in.)	W_i (gr.)	W_b (gr.)	W_1 (gr.)	D_2 (in.)	H_2 (in.)	W_2 (gr.)	W_4 (gr.)	W_2-W_4	$W_m = W_1-W_4$
						No. 4 Bars						
1	5.850	.475	.026	150.34	150.16	149.56	.475	.027	149.42	149.36	.06	0.98
2	6.110	.483	.027	157.23	157.06	156.45	.483	.027	156.30	156.22	.08	1.01
3	6.212	.485	.028	160.96	160.82	160.10	.482	.027	159.91	159.85	.06	1.11
4	5.997	.486	.027	155.12	155.02	154.27	.485	.028	154.09	154.01	.08	1.11
5	6.155	.488	.024	157.20	157.01	156.44	.484	.026	156.30	156.24	.06	0.96
6	6.185	.481	.029	159.07	158.90	158.20	.483	.028	158.02	157.98	.04	1.09
7	6.022	.485	.027	156.03	155.90	155.20	.483	.027	155.00	155.00	.00	1.03
8	6.040	.488	.027	158.71	158.58	157.81	.485	.026	157.65	157.59	.06	1.12
9	6.000	.487	.024	150.78	150.46	150.02	.483	.023	149.90	149.86	.04	0.92
10	5.965	.487	.026	150.35	150.04	149.64	.480	.026	149.52	149.51	.01	0.84
11	5.995	.483	.031	154.31	154.11	153.56	.478	.029	153.40	153.33	.07	0.98
12	6.000	.485	.026	156.07	155.90	155.20	.478	.027	155.04	154.93	.11	1.14
13	5.987	.483	.028	153.50	153.33	152.73	.483	.027	152.59	152.52	.07	0.98
14	5.960	.487	.024	150.18	149.85	149.42	.485	.025	149.30	149.28	.02	0.90
15	5.982	.487	.026	154.76	154.54	153.98	.483	.026	153.80	153.79	.01	0.97
16	6.172	.493	.024	161.88	161.72	160.96	.485	.026	160.70	160.66	.04	1.22
17	5.990	.487	.029	156.15	155.99	155.31	.483	.027	155.11	155.09	.02	1.06
18	5.995	.493	.024	155.60	155.39	154.85	.488	.025	154.60	154.57	.03	1.03
19	6.077	.487	.026	155.70	155.51	154.90	.484	.025	154.74	154.71	.03	0.99
20	6.090	.485	.027	158.69	158.30	157.86	.482	.027	157.67	157.63	.04	1.06
						Averages						
	6.039	.486	.026	155.63	155.44	154.82	.483	.026	154.65	154.60	.04	1.02
						No. 6 Bars						
1	6.025	.708	.047	340.17	339.72	338.95	.709	.045	338.62	338.52	.10	1.65
2	6.035	.712	.045	341.28	340.95	339.98	.709	.046	339.65	339.50	.15	1.78
3	5.975	.715	.047	339.81	339.59	338.28	.708	.049	338.08	337.94	.14	1.87
4	5.872	.712	.043	325.56	325.25	324.20	.709	.046	324.00	323.87	.13	1.69
5	6.047	.717	.042	341.47	341.21	329.90	.707	.045	339.62	329.56	.06	1.91
6	5.950	.711	.048	335.98	335.61	334.61	.706	.048	334.29	334.20	.09	1.78
7	5.928	.710	.052	336.73	336.61	335.16	.707	.048	334.92	334.79	.13	1.94
8	5.838	.713	.050	331.37	331.00	329.91	.707	.046	329.71	329.65	.06	1.72
9	5.965	.710	.046	331.23	330.95	329.91	.703	.046	329.58	329.49	.09	1.74
10	5.950	.717	.042	333.49	333.31	332.00	.710	.045	331.71	331.60	.11	1.89
11	5.900	.702	.051	332.40	331.96	331.02	.696	.052	330.71	330.63	.08	1.77
12	6.170	.703	.050	341.39	341.02	339.96	.700	.051	339.50	339.41	.09	1.98
13	5.975	.701	.053	335.40	334.88	334.12	.702	.049	333.78	333.70	.08	1.70
14	6.102	.715	.048	346.35	346.16	344.73	.709	.048	344.52	344.41	.11	1.94
15	5.925	.717	.043	334.32	334.12	332.85	.711	.045	332.54	332.43	.11	1.89

TABLE A.2 (con't)

Bar No.	L (in.)	D_1 (in.)	H_1 (in.)	W_1 (gr.)	W_b (gr.)	W_1 (gr.)	D_2 (in.)	H_2 (in.)	W_2 (gr.)	W_4 (gr.)	W_2-W_4	W_m = W_1 - W_4
					No. 6 Bars (con't)							
16	6.135	.710	.045	345.89	345.53	344.44	.701	.047	344.09	344.00	.09	1.89
17	5.943	.706	.047	337.09	336.79	335.60	.700	.048	335.28	335.20	.08	1.89
18	5.998	.710	.050	339.58	339.26	338.10	.710	.046	337.79	337.70	.09	1.88
19	5.938	.715	.048	337.11	336.85	335.72	.708	.048	335.59	335.49	.10	1.62
20	5.940	.704	.058	335.30	334.99	334.09	.702	.048	333.58	333.39	.19	1.91
					Averages							
	5.980	.710	.047	332.09	336.79	335.67	.706	.047	335.37	335.27	.10	1.82
					No, 8 Bars							
1	5.894	.965	.070	593.21	592.58	591.61	.961	.066	590.90	590.67	.23	2.54
2	5.873	.955	.071	588.39	587.82	586.90	.956	.072	586.00	585.79	.21	2.60
3	5.900	.960	.062	588.28	585.49	584.84	.950	.070	584.28	584.15	.13	2.13
4	5.961	.964	.068	602.38	601.79	600.76	.957	.070	599.90	599.70	.20	2.68
5	6.038	.953	.065	600.42	599.79	598.84	.954	.072	598.13	597.98	.15	2.44
6	5.970	.960	.065	600.30	599.73	598.70	.951	.068	597.81	597.70	.21	2.60
7	5.960	.960	.072	607.26	606.88	605.84	.960	.071	604.71	604.49	.22	2.77
8	5.937	.960	.061	589.76	588.91	588.08	.953	.068	587.32	587.11	.21	2.65
9	5.922	.960	.060	595.92	595.31	594.31	.954	.069	593.69	593.31	.38	2.61
10	5.946	.961	.070	595.18	594.72	593.68	.956	.073	592.76	592.58	.18	2.60
11	5.935	.960	.060	595.45	594.87	592.82	.955	.070	593.18	593.03	.15	2.42
12	6.040	.960	.061	602.36	601.65	600.87	.950	.069	600.20	600.07	.13	2.29
13	6.062	.962	.066	602.78	602.06	601.26	.953	.068	600.57	600.40	.17	2.38
14	6.080	.960	.063	600.55	599.75	598.74	.952	.068	598.00	597.81	.19	2.74
15	6.000	.970	.075	607.78	607.30	606.58	.960	.066	605.47	605.15	.32	2.63
16	5.970	.970	.076	609.36	608.99	607.40	.963	.074	606.50	606.26	.24	3.10
17	5.900	.955	.075	585.65	584.92	583.98	.955	.067	582.51	583.35	.16	2.30
18	5.948	.965	.063	598.31	597.75	596.66	.959	.066	596.13	595.80	.33	2.51
19	5.940	.962	.066	588.56	587.74	586.68	.956	.065	586.18	585.93	.25	2.63
20	6.050	.965	.072	602.48	601.76	600.78	.957	.070	600.10	599.78	.32	2.70
					Averages							
	5.966	.963	.067	597.61	598.92	595.99	.956	.069	595.27	595.05	.22	2.57
					No. 10 Bars							
1	5.963	1.19	.090	944.10	942.48	940.68	1.18	.093	939.88	939.81	.07	4.39
2	5.823	1.19	.085	921.72	921.03	919.25	1.19	.089	918.54	918.40	.14	3.32
3	5.978	1.19	.085	944.05	942.90	940.82	1.18	.086	940.02	939.84	.18	4.21
4	6.035	1.19	.086	951.23	950.42	948.14	1.18	.088	947.01	946.85	.16	4.38
5	6.030	1.19	.086	951.04	950.43	947.77	1.19	.091	946.90	946.78	.12	4.26

TABLE A.2 (con't)

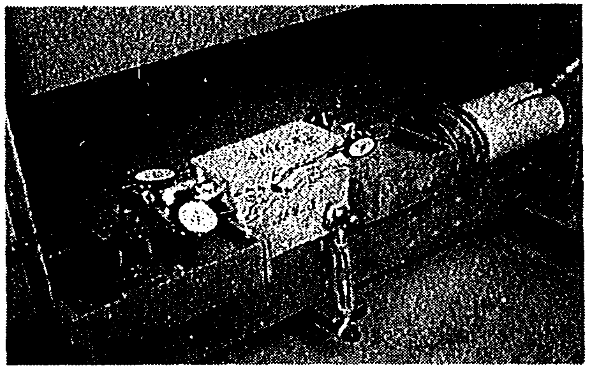
Bar No.	L (in.)	D1 (in.)	H1 (in.)	W_i (gr.)	W_b (gr.)	W_1 (gr.)	D2 (in.)	H2 (in.)	W_2 (gr.)	W_4 (gr.)	$W_2 - W_4$	$W_m = W_1 - W_4$
					No. 10 Bars (con't)							
6	5.956	1.19	.091	946.19	945.62	942.80	1.19	.090	942.01	941.85	.16	4.34
7	5.893	1.19	.087	931.55	931.01	928.05	1.19	.090	927.20	927.02	.18	4.53
8	5.927	1.19	.089	933.00	932.42	929.91	1.18	.089	929.10	928.91	.19	4.09
9	5.888	1.19	.087	928.85	927.96	926.01	1.19	.088	925.10	924.90	.20	3.95
10	5.968	1.19	.085	946.18	945.61	944.30	1.19	.088	942.24	941.92	.32	4.26
11	5.962	1.19	.086	938.64	937.82	936.54	1.19	.086	934.89	934.67	.22	3.97
12	5.903	1.19	.087	931.02	930.36	928.76	1.19	.088	927.25	927.04	.21	3.98
13	5.855	1.19	.084	919.86	918.83	917.63	1.19	.086	916.21	915.96	.25	3.90
14	6.078	1.19	.089	966.04	965.48	963.83	1.19	.090	961.52	961.38	.14	4.66
15	6.040	1.20	.087	954.85	954.18	952.70	1.19	.090	950.78	950.52	.26	4.33
16	5.940	1.19	.084	926.51	924.89	923.65	1.18	.084	922.86	922.70	.16	3.81
17	6.247	1.19	.086	984.61	984.61	982.89	1.18	.085	981.70	981.53	.17	3.82
18	5.880	1.20	.089	935.38	935.38	933.07	1.19	.088	931.45	931.12	.33	4.53
19	5.968	1.19	.086	930.56	930.56	928.59	1.19	.087	928.00	927.76	.24	3.85
20	6.047	1.19	.087	946.02	946.02	944.25	1.18	.090	943.95	943.61	.34	3.84
					Averages							
	5.969	1.19	.087	941.74	940.90	938.98	1.19	.088	937.83	937.63	.20	4.12
					No.11 Bars							
1	6.020	1.37	.073	1199.18		1195.89	1.36	.081	1195.17	1194.70	.47	4.48
2	5.965	1.36	.067	1178.91		1175.57	1.36	.068	1174.71	1174.47	.24	4.44
3	5.840	1.36	.079	1165.10		1161.90	1.36	.081	1160.87	1160.60	.27	4.50
4	5.753	1.37	.074	1148.00		1145.00	1.37	.074	1143.73	1143.43	.30	4.57
5	5.932	1.36	.074	1186.09		1182.50	1.36	.074	1181.45	1181.14	.31	4.95
6	6.005	1.37	.065	1175.80		1172.25	1.36	.069	1171.40	1171.15	.25	4.65
7	5.916	1.35	.077	1168.24		1164.59	1.34	.076	1163.90	1163.72	.18	4.52
8	6.075	1.36	.079	1206.73		1203.45	1.36	.079	1202.23	1201.93	.30	4.79
9	5.795	1.36	.067	1143.22		1140.24	1.36	.068	1139.38	1139.10	.28	4.12
10	5.903	1.36	.070	1161.16		1157.95	1.36	.074	1157.07	1156.77	.30	4.39
11	5.978	1.37	.069	1191.56		1188.21	1.36	.068	1187.18	1186.75	.43	4.81
12	5.895	1.36	.075	1164.61		1161.92	1.36	.073	1161.06	1160.70	.36	3.91
13	5.915	1.37	.070	1169.19		1166.51	1.37	.068	1164.98	1164.53	.45	4.66
14	5.940	1.36	.076	1184.41		1181.91	1.36	.077	1180.40	1180.03	.37	4.38
15	5.800	1.36	.073	1160.34		1157.65	1.36	.075	1156.49	1156.18	.31	4.16
16	5.641	1.36	.075	1122.20		1119.50	1.36	.076	1118.39	1118.13	.26	4.07
17	5.949	1.37	.066	1179.71		1177.31	1.36	.068	1175.70	1175.23	.47	4.48
18	6.037	1.36	.068	1192.51		1190.11	1.36	.072	1188.60	1188.15	.45	4.36
19	6.075	1.36	.070	1189.90		1187.20	1.36	.069	1185.75	1185.40	.31	4.50
20	6.133	1.36	.067	1203.56		1200.30	1.36	.066	1199.03	1198.77	.26	4.79
						Averages						
	5.928	1.36	.072	1174.52		1171.50	1.36	.073	1170.37	1170.04	.33	4.47

TABLE A.3

ULTIMATE LOADED-AND FREE-END SLIP

Specimen No.	Slip in. x 10^{-5} LE	FE	Spec. No.	Slip in. x 10^{-5} LE	FE	Spec. No.	Slip in. x 10^{-5} LE	FE
4A 0-a	238	000	6M 6-c	1150	60	6N12-e	1385	21
b	175	50	-d	1560	35	8M12-a	503	00
c	1300	000	6N 6-c	1132	10	-b	500	21
6A 0-a	360	7	-d	1105	15	-c	557	00
b	575	6	-e	1075	185	-d	1625	00
c	820	105	8M 6-a	180	25	8N12-c	575	10
8A 0-a	605	2	-b	115	00	-d	530	3
b	612	5	-c	570	25	-e	1340	09
c	437	5	-d	420	30	10M12-a	525	24
10A 0-a	325	0	8N 6-c	990	80	-b	--	17
b	110	1	-d	495	225	-c	910	55
c	385	15	-e	1113	55	-d	1600	24
11A 0-a	720	5	10M 6-a	460	20	10N12-c	785	4
b	750	10	-b	1510	35	-d	1225	36
c	612	15	-c	500	20	-e	1445	9
4N 3-c	600	15	-d	530	20	11M12-a	135	4
-d	560	15	10N 6-c	675	10	-b	--	12
-e	1190	45	-d	245	40	-c	1190	6
6N 3-c	1130	10	-e	365	35	-d	1150	6
-d	870	5	11M 6-a	--	05	11N12-c	1600	15
-e	820	230	-b	410	00	-d	1175	26
8N 3-c	565	10	-c	738	05	-e	1610	18
-d	1330	50	-d	1175	45	4S 3-a	900	5
-e	785	10	11N 6-c	555	30	-c	650	5
10N 3-c	510	10	-d	340	45	-d	1760	640
-d	480	25	-e	375	15	6S 3-a	650	30
-e	400	10	4M12-a	--	40	-c	1400	40
11N 3-c	1100	20	-b	990	55	-d	3000	145
-d	175	25	-c	540	26	8S 3-a	420	10
-e	470	15	-d	1062	10	-c	1320	630
4M 6-a	205	5	4N12-c	1260	04	-d	1160	200
-b	250	35	-d	1200	125	10S 3-a	275	60
-c	625	00	-e	575	15	-c	900	220
-d	1430	60	6M12-a	150	11	-d	725	60
4N 6-c	1255	00	-b	113	03	11S 3-a	250	00
-d	1575	25	-c	575	10	-c	760	800
-e	283	20	-d	1575	27	-d	380	60
6M 6-a	375	30	6N12-c	925	00	LE = Loaded end		
-b	1425	105	-d	600	313	FE = Free end		

A



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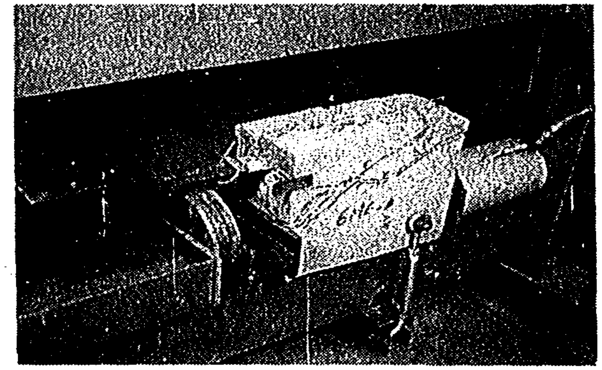

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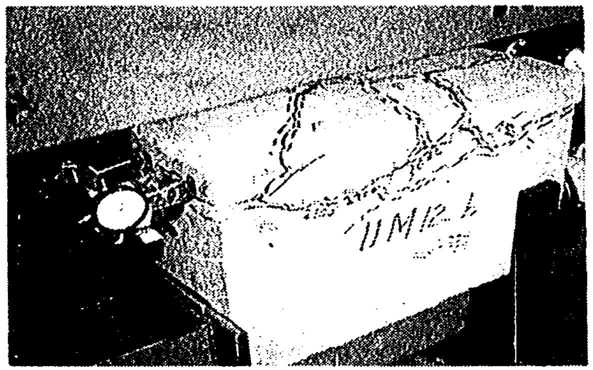
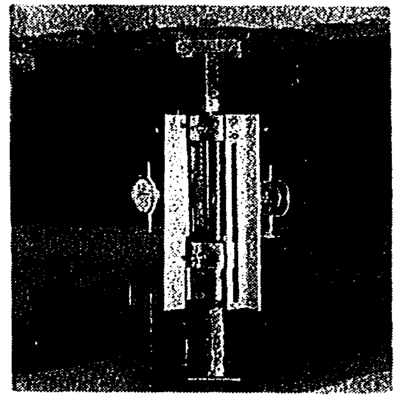
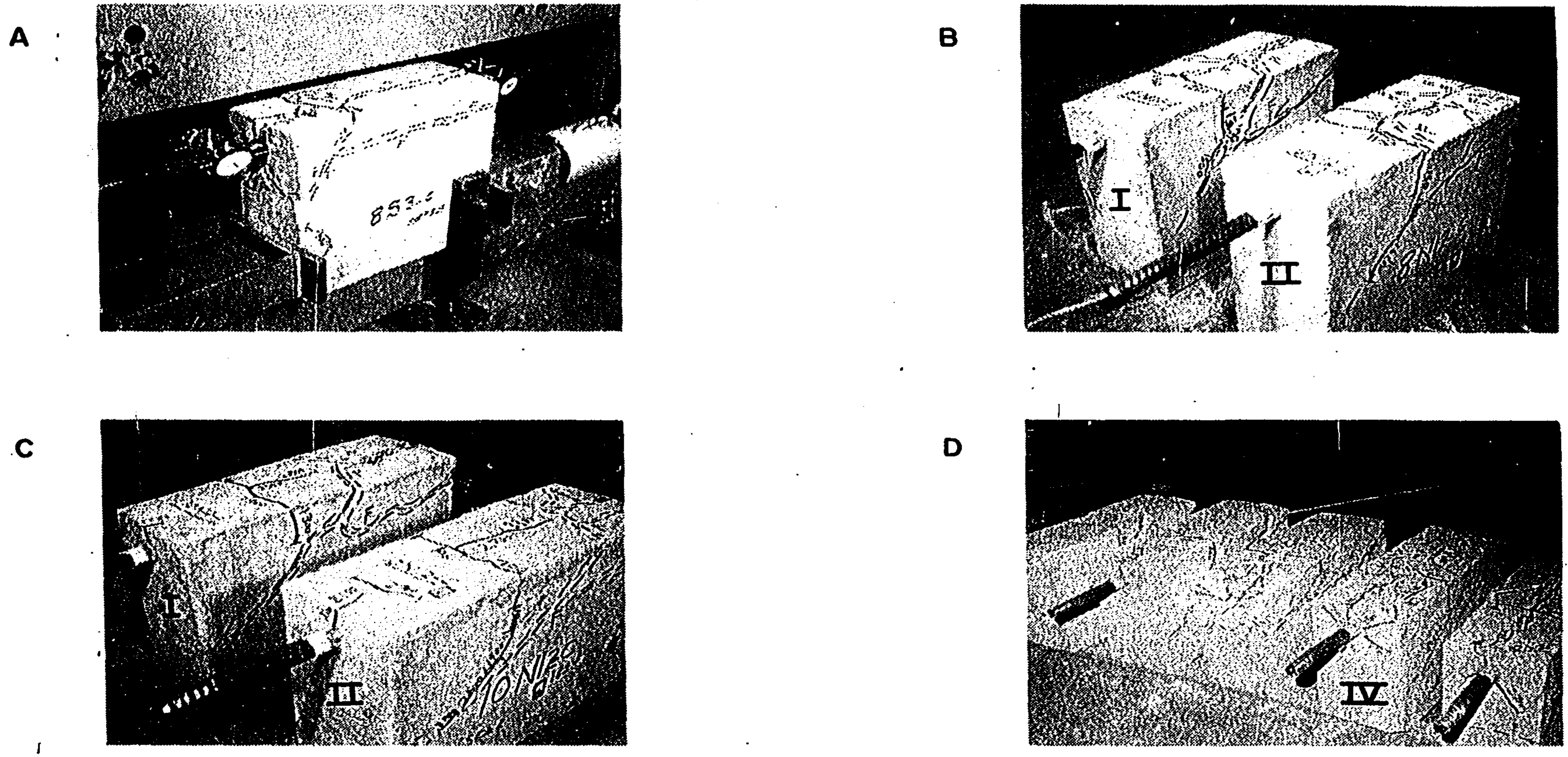


D



A. Double Spaced #4 Bars, 6 Months Indoor B. Double Adjacent #6 Bars, 6 Months Indoor
C. Double Adjacent #11 Bars, 12 Months Indoor D. Tensile Test Rig.

Fig. A.1

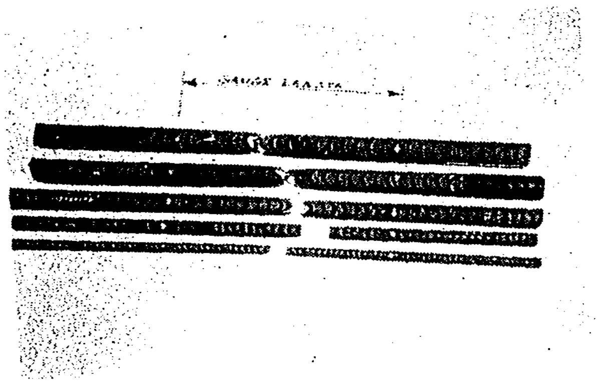


A. # 8, 3 Months Sea Water
C. # 10, 12 Months Outdoor
(I. No Stirrups; II. With Stirrups)

B. # 8, 12 Months Outdoor (I. No Stirrups; II. With Stirrups)
D. # 11,12 Months Outdoor (IV. With Stirrups)

Fig. A.2 Typical cracking patterns of single bar specimens

A



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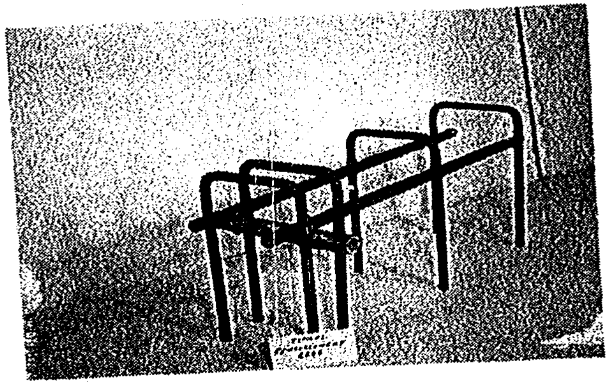
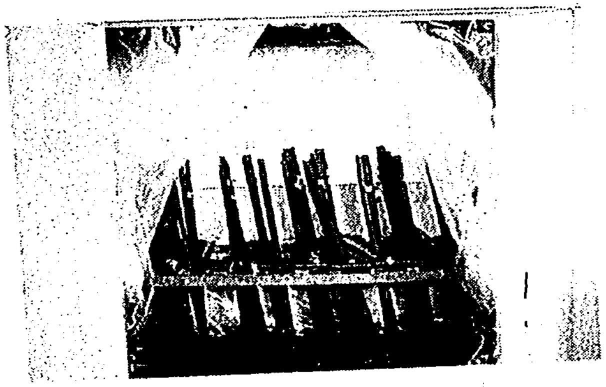
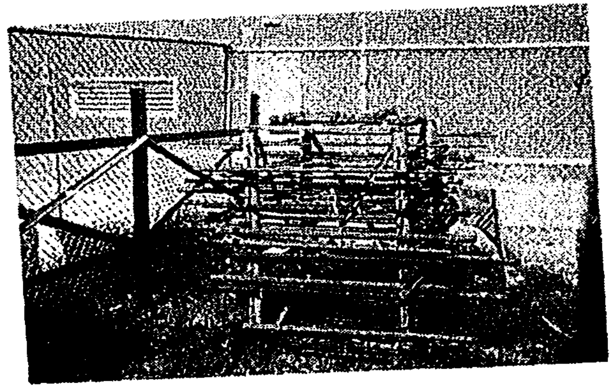


C

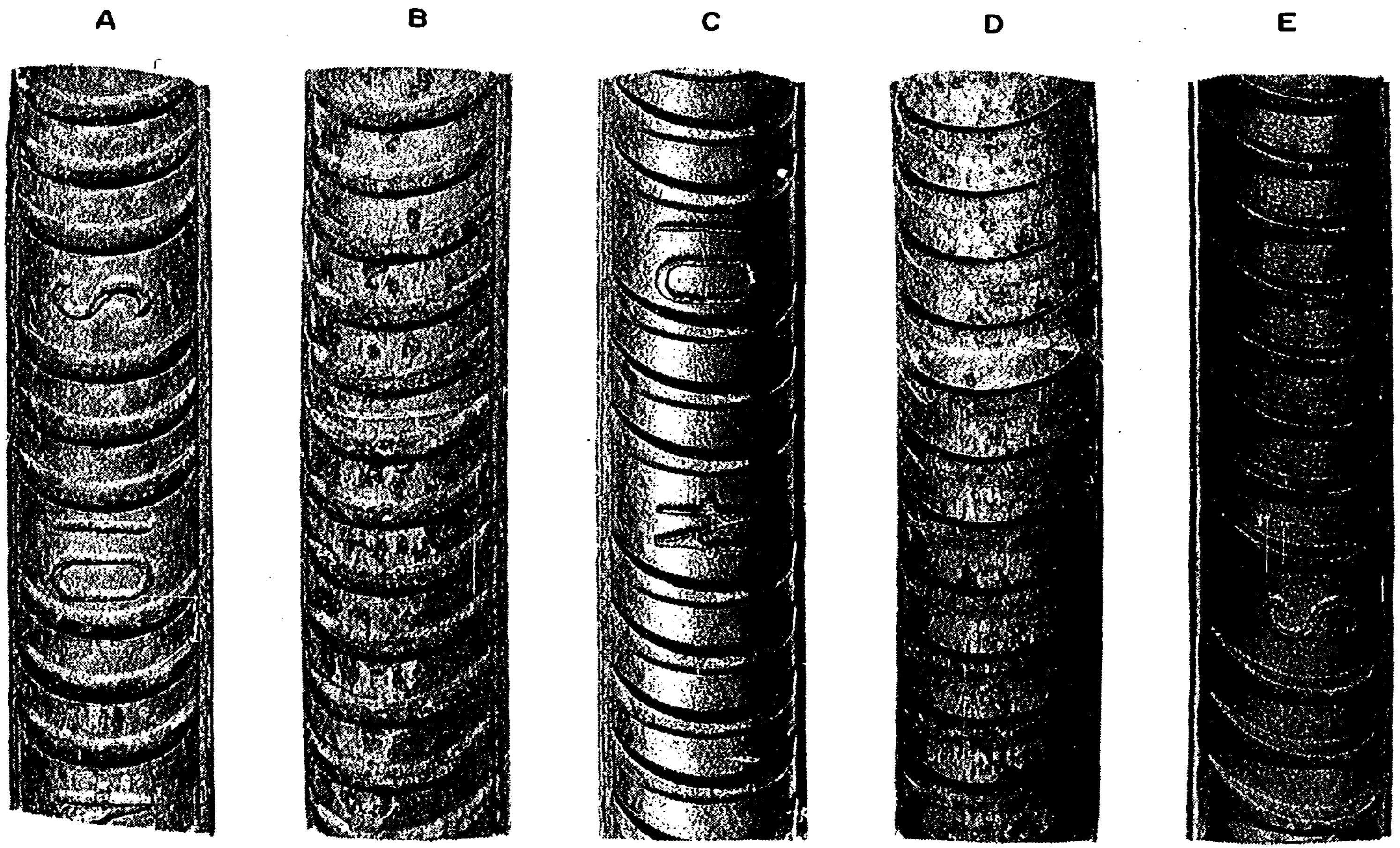


D



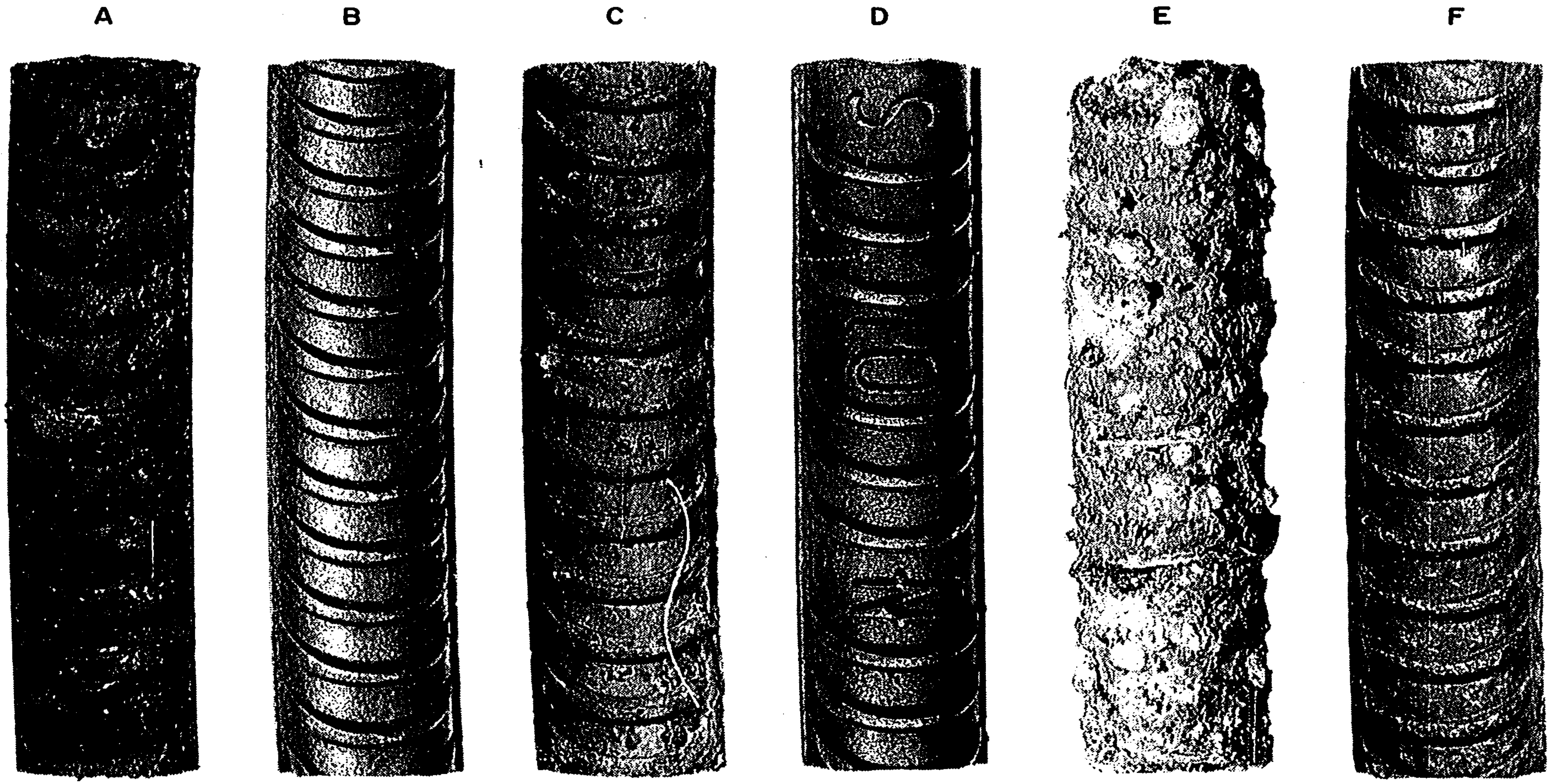
A. Typical tensile failure of bars
B. Typical reinforcing cage
C. Moist cabinet
D. Outdoor rack

Fig. A.3



A. As-Rolled B. 3 months rust, outdoor C. 3 months outdoor, cleaned D. 12 months rust,outdoor E. 12 months outdoor, cleaned

Fig. A.4 #10 as-rolled, corroded and chemically cleaned and wire brushed bars



A. 6 months rust, indoor B. 6 months indoor, cleaned C. 12 months rust, indoor
D. 12 months indoor, cleaned E. 3 months rust, sea water F. 3 months sea water, cleaned

Fig. A.5 #10 corroded and chemically cleaned and wire brushed bars

VITA

Mehdi Ghaffarzadeh was born in Isfahan, Iran (Persia) the son of Esmael Ghaffarzadeh and Baygom Ghaffarzadeh. Before completing his last year of study at Saadi High School, Isfahan, Iran, he was granted a full scholarship to Abadan Institute of Technology, Abadan, Iran where he received the degree of Bachelor of Science in General Engineering in June 1962. Upon graduation, he was employed by National Iranian Oil Company and was sent to Tehran, Iran, to take a semester of special course work in Industrial Management Engineering. He worked for a short time in the Organization, Methods, and Systems Division of the National Iranian Oil Company. In January 1963 he enrolled in the Graduate College of the University of Oklahoma at Norman, Oklahoma where he was awarded the degree of Master of Science in June 1965.

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