PB93-214658

日米

U.S. - JAPAN COORDINATED PROGRAM

FOR

MASONRY BUILDING RESEARCH

REPORT NO. 6.2-1

BOND OF REINFORCEMENT IN GROUTED HOLLOW-UNIT MASONRY: A STATE- OF-THE-ART

by

JOHN C. SCRIVENER

JULY 1986

supported by:

NATIONAL SCIENCE FOUNDATION

GRANT NO. ECE-8421234

Atkinson-Noland & Associates

This report presents the results of a research project which was part of the U.S. Coordinated Program for Masonry Building Research. The program constitutes the United States part of the United States - Japan Coordinated Masonry Research Program conducted under the auspices of the Panel on Wind and Seismic Effects of the U.S.-Japan Natural Resources Development Program (UJNR).

This material is based on work supported by the National Science Foundation under the direction of Program Director, Dr. S.C. Liu.

BOND OF REINFORCEMENT IN GROUTED HOLLOW-UNIT MASONRY: A STATE-OF-THE-ART

conducted by:

ATKINSON-NOLAND & ASSOCIATES, INC.
2619 Spruce Street
Boulder, Colorado 80302
(303) 444-3620

Principal Investigator: John C. Scrivener

July 1986

PREFACE

This report was prepared in support of research Category 6, Task 6.2 of the U.S. Coordinated Program for Masonry Building Research. The subject area of Task 6.2 is reinforcement bond and splices in grouted masonry. The report documents a review of relevant literature in the subject area and includes work in reinforced concrete as well as reinforced masonry.

Dr. John C. Scrivener prepared the report while on sabbatical leave from the University of Melbourne, Australia. The work was supported by NSF Grant ECE-8421234.

J. L. Noland
July 1986

INTRODUCTION

In the U.S. Research Plan for the U.S.-Japan Coordinated Program for Masonry Building Research, August 1984, the Title and Purpose of Task 6.2 was:

"Title - "Reinforcement, Bond and Splices in Grouted Hollow Unit Masonry"

Purpose - To develop data and behavioral models on the bond strength and slip characteristics of deformed bars in grouted hollow unit masonry; to develop data and behavioral models on the bond strength and slip characteristics of deformed bar lap splices in grouted hollow unit masonry as needed for building modelling."

The bond and slip characteristics are required not only in the static load situation but also under conditions when the reinforced masonry component is subjected to static cyclic loading containing reversals of small and large amplitude simulating an earthquake (Task 9.4).

Early research in reinforced masonry using static, static cyclic and dynamic loading, found that reinforced masonry behaves broadly in the same manner as reinforced concrete with, of course, minor differences due to geometrical restrictions etc. As the study of bond, slip and anchorage lengths in reinforced concrete is much more advanced in time and quantity than equivalent research in reinforced masonry, and as it is expected that reinforced masonry will behave broadly in the same way as reinforced concrete, this report will commence with a survey of reinforced concrete findings.

In summary the report will deal with research findings in the bond, slip and anchorage field in:

(a) reinforced concrete monotonically loaded;

(b) reinforced concrete subjected to static cyclic, dynamic and simulated earthquake loading;

(c) reinforced masonry.

BOND IN REINFORCED CONCRETE MONOTONICALLY LOADED

The bond resistance of plain bars at low stresses consists of a chemical adhesion between the cement paste and the bars. Once this adhesion is broken and slips occur further bond may be developed by friction. With the advent of deformed bars, and the interlocking effect of the ribs on the surrounding concrete, much increased bond capacity was possible. This led to a virtual takeover by deformed bars and accordingly, this report will not cover bond in plain bars.

The bond behaviour of deformed bars, the introduction of higher strength steel in reinforcing and the use of larger diameter bars precipitated a fresh look at bond considerations by ACI Committee 408 in 1966 and their state-of-the-art report (1) is a useful starting point.

Bond Stress

Bond stress, u, is usually defined as a shear force per unit area of bar surface,

$$u = \frac{q}{\Sigma_0} = \frac{\Delta f_s A_b}{\Sigma_0} = \frac{\Delta f_s d_b}{4} \qquad (1)$$

where q = change of bar force per unit length

Σ_0 = nominal surface area of bar of unit length

d_b = nominal diameter of bar

Δf_s = change in steel stress over unit length

A_b = area of bar

Bond stresses arise from two different circumstances - from the anchorage of bars and from the fluctuation in bar force caused by a change in the bending moment.

Anchorage or Development Bond

Anchorage length is the length necessary to fully transfer a given force in a bar to the surrounding concrete while development length is the length necessary to fully transfer a given force in the concrete into the surrounded bar. Whether the bond should be called anchorage bond or development bond is simply a matter of viewpoint as they are identical concepts.

If the bond stress, u, is assumed to be uniformly distributed over the development or anchorage length, l_d, then

$$T = A_b f_s = u \ \Sigma_0 \ l_d \tag{2}$$

where T = bar space

f_s = bar stress

This leads to

$$l_d = \frac{d_v f_s}{4u} \tag{3}$$

The alternative now is to define a safe value for the bond stress, u, or, as the ACI Reinforced Concrete Code (2), to prescribe minimum development lengths, l_d, for various situations.

Flexural Bond

By consideration of the moment equilibrium on a short length, Δx, of a beam subjected to a bending moment, ΔM, changing along the length of the beam, the increment of tensile force in the bar, ΔT, is

$$\Delta T = \frac{\Delta M}{jd} = \frac{V \Delta x}{jd} = u \Sigma_0 \Delta x \qquad (4)$$

where V = shear force

Hence, flexural bond stress, u, is

$$u = \frac{V}{\Sigma_0 jd} \qquad (5)$$

From equation (5) it can be seen that if V is high (i.e., the rate of change of bending moment is high) then the flexural bond stress may be high. As this does not take into account the likely cracks, equation (5) oversimplifies the situation. Even when the shear force is zero, at a constant moment region, bond stress will be developed.

However, it appears from tests that failure originating from flexural bond stress does not occur provided the bars are given sufficient anchorage length. Thus, anchorage lengths should be checked in regions where the bending moment is zero e.g., at simple supports and at points of contraflexure. This will be discussed later.

Bond Stress in a Cracked Flexural Member

Even where shear is zero and bending moment is constant, large local bond stresses exist adjacent to each flexural crack (1). At the crack most of the tension is carried by the bars and the steel stress is maximum. Between cracks the concrete carries tension and the steel stress drops off in a compensating manner. Thus bond takes stress out of the steel adjacent to a crack and puts it back just before the next crack is reached. The rate of change of the steel stress is a function of the area of the bar relative to the area of the concrete.

Now consider the situation when the bending moment is changing from crack to crack requiring that the (average) steel stress must change. This implies a bond stress over and above the "out-and-in" stresses of the previous paragraph. It is not known exactly how this occurs. Further since ultimate bond stresses already exist near the crack even in constant moment conditions an increase of stress there is not possible. Either a greater length of bar must carry the high stress as in Fig. 1(a), or near the adjacent crack the reverse kind of bond must be reduced as in Fig. 1(b). If some splitting starts, perhaps the stress near the right hand crack decreases even more than shown by the alternate in Fig. 1(b). ACI Committee 408 (1) believes that it is reasonable that considerable "out-and-in" bond remains, which is obviously not very efficient and tension cracking must lower this efficiency.

When a diagonal tension crack opens the stresses must be redistributed leading to high steel stress and more bond stress on the side of the crack nearer the point of inflection or the simple beam support. In addition the dowel action of the bar puts tension into the concrete parallel to the bar which adds to the splitting caused by high local bond stress (1).

Bond in Brackets, Short Cantilevers and Deep Beams

In brackets, short cantilevers and deep beams there may not be sufficient development length available from the position of maximum moment (at the support for brackets and cantilevers) to the free end of the bracket or cantilever. It can be shown (1) from deep beam theory, using an elastic uncracked section and cracked beam behaviour, that where the moment approaches zero, the steel stress remains quite large. Accordingly the reinforcing must be anchored beyond this point to develop the stress.

Tests (1) have shown that an extension beyond the load in a cantilever or a cross bar welded to the main steel can provide the necessary anchorage.

Straight Anchorages of Deformed Bars

The required anchorage length of deformed bars in tension to develop the stress of the bar depends upon a number of factors.

The resistance against splitting is crucial. Partly crushed concrete forms a wedge in front of the ribs on a deformed bar with increase of bar tension and hence bond stress. The movement of this wedge will cause the concrete to split. The concrete surrounding the bar cannot sustain the circumferential tensile stresses. Fig. 2 illustrates these splitting cracks.

Thickening the cover will delay the splitting provided that the bars are not too closely spaced laterally (see fig. 2(c)). However, it is usually not economical to increase bond stress by increasing cover.

ACI Committee 408 (1) writes

> "Splitting seems to start at flexure cracks, being most evident where steel stress is largest. Thus splitting is a progressive phenomenon, working its way gradually along the length of embedment. Splitting may not be continuous from flexure crack to flexure crack. A splitting crack often stops short of the next flexure crack in a bond test beam; in fact the opening of a new flexural crack usually occurs beyond the end of a splitting crack, and added splitting then develops from the new flexure crack. Normally the splitting will eventually close the gap. Splitting can develop over 60 to 75 percent of the bar length without loss of average bond strength. Apparently splitting is

one means by which some of the unevenness in bond stress distribution may be smoothed out. However, the final failure is sudden (in the absence of stirrups) as the split suddenly runs through to the end of the bar."

As splitting is a tension failure, the ultimate bond stress varies approximately as $\sqrt{f'_c}$, as does the modulus of rupture (f'_c is the specified compressive strength of the concrete.) The ACI code (2) gives the basic development of a #11 bar or smaller as

$$0.04\ A_b f_y \sqrt{f'_c}$$

Park and Paulay (3) consider the particularly severe situation in the shear span of beams and contend that splitting can be induced along the flexural reinforcement by the combination of three events:

(i) Circumferential tensile stresses generated in the vicinity of each flexural crack.

(ii) Circumferential or transverse tensile stresses induced by wedging action of the deformations and by the compressed concrete at the ribs where large bond forces need to be transferred.

(iii) Transverse tensile stresses resulting from dowel action of the flexural reinforcement.

Splitting resistance is reduced where the bars are top cast and when lightweight concrete is used. However, greater splitting resistance is obtained when the side cover is large, the lateral spacing of the bars is large and the concrete is confined by spirals. Stirrups tend to slow the propagation of splitting cracks but much more than adequate stirrups for the expected shear is needed to increase the ultimate bond strength.

With bars in compression there is less tendency for splitting than in the tension case. Part of the compression force can be transferred to the concrete by end bearing. Accordingly development lengths in compression need not be as large as those required in tension.

Anchorage Length of Hooked Bars

In the late 70's a major departure in the thinking about the behaviour and design of hooked bar anchorages occurred in the U.S.A. Failures in tests (4) indicated that splitting of the cover parallel to the plane of the hook is the primary cause of failure and that the splitting originates at the inside of the hook where the local stress concentrations are very high. Thus hook development is a direct function of the bar diameter, d_b, which governs the magnitude of compressive stresses on the inside of the hook. This new understanding led to the uncoupling of hooked bar anchorages from straight bar development provisions and to the ACI code (2) giving the basic development length for a hooked bar of yield stress 60,000 psi as

$$1200 \, d_b \sqrt{f'_c}$$

Some pull-out tests by Muller in 1968, reported in (3), were conducted to obtain the strength of hooked anchorages. For deformed bars the strain distribution in the steel along the hook revealed that the bar force is transferred rapidly into the concrete (which is equivalent to the U.S. findings above) and the straight portion following the hook is generally ineffective. However, for plain bars the tensile stresses reduce more slowly along the hook so extra anchorage may be obtained by extending the straight portion beyond the hook which, of course, was the earlier understanding.

This Muller paper also showed that 10 - 30% more tensile stress can be developed for the same amount of slip when a bar is bent around a transverse bar. But this benefit can only be obtained if direct contact exists between the hook and the transverse bar partially because some deterioration in the quality of the concrete in the vicinity can be expected.

Another German paper, by Rehm in 1969, again reported in (3), showed that larger diameter hooks will transmit larger loads for given slips e.g. an $8d_b$ diameter hook, as used in the ACI code (2) for #3 through #8 bars, transmits some 50% more load than a $2.5d_b$ diameter hook at the same slip. Rehm also found that hooked anchorages with bends less than 180° do not necessarily provide anchorage superior to a straight bar of the same length.

Lap Splices

Where bars are lapped, at any point in the splice forces are being transferred by bond from one bar to the concrete surrounding it and then by bond from the concrete to the other bar. These forces in the concrete can generate high shear stresses as well as splitting forces.

Because each end of a splice introduces stress concentrations tending to precipitate early splitting cracks, splices are generally made longer than ordinary anchorage lengths.

Stirrups over splices increase the splice strength between 20 and 50% and the failure is less violent.

When the splice is between bars in compression, the bearing against the end of the bar strengthens the splice and so required lap lengths are lower than for tension splices and are approximately the development length in compression.

Anchorage Requirements for Flexural Bond

Where the external bending moment is very small (e.g. at simply supported beam supports and near points of contraflexure) the area of flexural reinforcement may be quite small. But the shear force may be high, so from equation (5) the flexural bond stress requirements may be critical.

From equation (2), the bond stress, u, may be written

$$u = \frac{A_s f_y}{\Sigma_0 l_d} \tag{6}$$

To restrict u to an acceptable maximum value a minimum anchorage length l_d is prescribed.

In the flexural bond situation, equation (5) gives

$$u_f = \frac{V}{\Sigma_0 jd}$$

In order that the flexural bond, u_f, be less than u

$$\frac{V}{\Sigma_0 jd} < \frac{A_s f_y}{\Sigma_0 l_d}$$

$$\text{i.e, } l_d < \frac{A_s f_y jd}{V}$$

$$\text{i.e, } l_d < \frac{M_n}{V} \tag{7}$$

where $M_n = A_s f_s jd$ = nominal flexural capacity of the section.

l_d is now the required anchorage length for the particular bar size used. If an embedment length, l_a, beyond the support (or point of contraflexure) is provided then equation (7) may be modified to

$$l_d - l_a < \frac{M_n}{V} \tag{8}$$

As stated by Park & Paulay (3) if equation (8) is not satisfied then one of the three following actions is required:

(i) increase the total steel area, A_s, at the section thus increasing M_n;

(ii) increase the anchorage length, l_a, beyond the section by hooking or bending up bars;

(iii) reduce the required development length, l_d, by using smaller diameter bars.

Test Methods for Bond Strength

In the pull-out test, the bar is embedded in the concrete specimen and it is pulled from one end while the concrete at the same end is subject to a reaction force. As the bar is in tension and the concrete is in compression, differential strains create a relative slip, even at low steel stresses, and the slip represents a local loss of adhesion. As a very high bond stress can be resisted over a short length of bar, the slip at small loads is localized near the loaded end. Beyond the slip zone, bond exists but most of the bar is unstressed by the tension pull. As the tension pull is increased, the length of slip is increased and the length of bar unstressed is correspondingly reduced. With deformed bars, the loaded end slip gradually brings the lugs into bearing which raises the average bond stress. Finally, in a structural element, this stress is restricted by the splitting which will occur. As the concrete is in compression in the pull-out test, transverse tension cracking is restricted or eliminated and therein lies the weakness of the test. However, it is a useful comparative test and it does appear to give a reasonable idea of the necessary anchorage length of a bar embedded in a mass of concrete. It cannot simulate bond behaviour in a beam or flexural situation where shear forces and diagonal cracks, splitting cracks initiated by dowel action and cover all affect bond performance. Fig. 3 illustrates various types of pull-out tests, (a) to (e), and other test arrangements, (f) and (g), which overcome, to a greater or lesser extent, the objections of the pull-out test. ACI Committee 408 (5) gives a detailed guide for the determination of bond stress in beam specimens.

The problem remains of how to determine the safe bond strength of structural members from the ultimate bond strength developed in a particular pull-out test. Usually the bond strength is expressed in terms of the average bond stress developed along and around the embedded surface of the bar, knowing that peak bond stress values are well in excess of this average. Mathey and Watstein (6) in addressing this problem produced two criteria to define "critical" bond stresses: a loaded-end slip of 0.01 in. and a free end slip of 0.002 in. whichever occurs first in the pull-out test. Bond stresses corresponding to these values of slip are sufficiently low to ensure that under-reinforced beams designed on the basis of these criteria will fail by yielding of reinforcement and not in bond. Their tests gave no assurance that the loaded-end slip alone would indicate impending bond failure in a beam.

Some other tests (7) showed that long specimens developed high steel and bond stresses even though the loaded end slip was large. The loaded end slip varied approximately as the bar diameter but the length of embedment had little influence on the steel stress developed at a loaded end slip of 0.01 in. Unloaded end slip occurred in top bars, at relatively small loads even before splitting started in some cases.

Further methods for the testing of bond are described in later sections.

Bond Stress-Slip Relationships by Tests

While the early studies concentrated on average bond stresses, primarily to determine anchorage lengths for design codes, later research has been more concerned with the distribution of bond stresses along the reinforcement and with the bond-slip relationship. The interest in the latter arose with finite element analyses and with concern for the behaviour under cyclic and seismic loading.

Lahnert (8) summarizes the various techniques that have been used to measure slip. Wahla (9) used LVDTs with the core in the concrete and the body on the bar; Tanner (10) and Dorr (11) measured steel and concrete strains, with strain gauges, between a point of known slip and the point of interest; Mirza and Houde (12) calculated slips from measured end slips and an assumed concrete strain distribution; Jiang et al (13) using a specimen in which a reinforcing bar is split and the two halves embedded in opposite sides of the concrete cross-section, measured slips optically with a microscope; using very short embedment lengths, end slips only were needed by Rehm (14) and Edwards and Yannapoulos (15).

According to Lahnert (8), the first to develop a local force-displacement relation was Rehm (14). He was followed by Nilson (16), Duarte (17) and Jiang et al. (13) who all showed that the local bond stress-slip relationship is dependent on the distance from the face of the specimen.

The experimental technique adopted by Lahnert (13) to measure local slip directly was originally developed by Nies (18). Coils within the reinforcing bar produced a magnetic field which was sensed by a target embedded in the concrete at 0.4 in. from the bar surface. Bond stresses were derived from strains obtained from strain gauges mounted in grooves milled on opposite sides of the reinforcing bar. It was difficult to get accurate bond stresses, even though the strain gauges were closely spaced, as they are very sensitive to the distribution of the bar forces. Lahnert plotted the bond stress-slip relationships of several investigators (Fig. 4) and his own results (Fig. 5) from both tension and pull-out tests for external zones (at specimen ends) and internal zones (away from specimen ends). He concluded,

> "...it appears that the differences between test types, barring scatter, depend primarily on the amount of confinement provided, where confinement means the radial restraint on the bar whether provided by a large concrete cover, secondary reinforcement, or applied radial pressure. For similarly dimensioned specimens, pullout tests are more confined than tension tests, and short bond length pullout tests are the most confined of all, a trend that is reflected in Figs. 4 and 5."

Analytical Bond Stress-Slip Relationships

Jiang (19) studied analytically the bond and slip relationship between a bar and the concrete within a segment bounded by two primary cracks in a reinforced concrete tension or flexural member. He pointed out that having written equilibrium and compatibility equations and stress-strain relationships for steel and concrete, only five equations are available to solve for six unknowns. There are two alternatives for the required additional equation. Either a local bond stress-slip relation must be assumed (or derived experimentally) or a bond stress distribution must be assigned.

Different solutions (20,21,22) have been proposed in which the relationship between bond stress and slip have been assumed. If a linear and unique relationship at every point is assumed, then a second order differential equation evolves which is relatively simply solved. If, however, the bond slip relationship is assumed to be non-linear no general analytical solution is available. Tassios et al. (22) solved the problem numerically.

The alternative approach of assuming a bond stress distribution has the advantage that a differential equation is not necessary and the solution may be obtained by integration. Jiang (19) tried various shapes of bond stress distribution and found that the results varied little one from another. Accordingly, he chose a simple expression, a second degree parabola satisfying the boundary conditions. He stated that results from this compared well with experimental results from the University of Illinois but unfortunately the crucial figure referred to was not printed in the paper! Attempts have been made to obtain it without success.

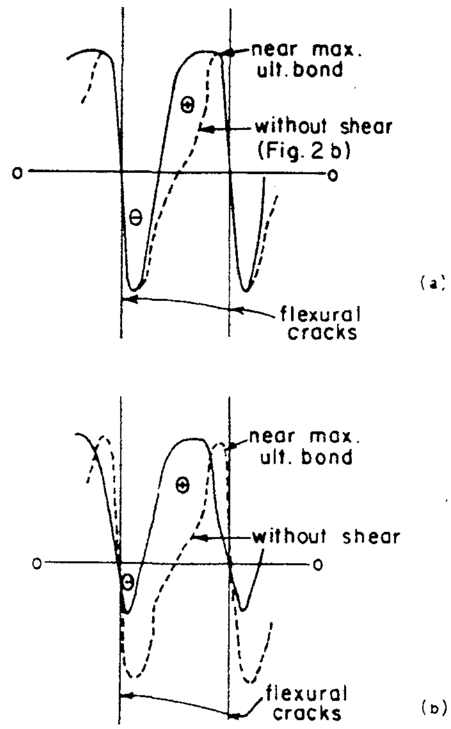
Linear elastic finite element analyses have been used by Ngo and Scordelis (23), Lutz and Gergely (24) and Bresler and Bertero (25) to predict stress and strain distributions in bond test specimens. Using a fracture mechanics approach, Gerstle et al. (26) devised a new finite element which lumps all the bond-slip behaviour at a location where a crack crosses a reinforcing bar. Other models, taking into account radial pressure (27) and the influence of the loading rate (28), employ a bond-slip layer finite element.

Other Research in Monotonically Loaded Reinforced Concrete

Since the 1966 ACI Committee 408 state-of-the-art report (1) there have been many other papers addressing bond behaviour in monotonically loaded reinforced concrete. Some of these papers are recorded here for completeness, and they are classified into topic areas:

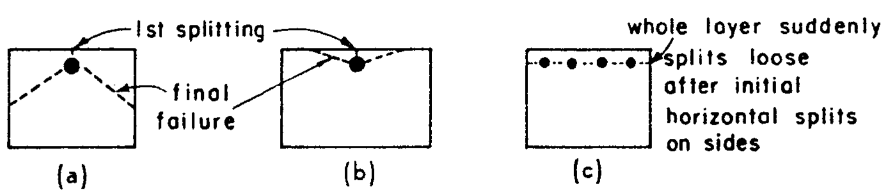
Development length and lap splices:	Thompson et al. (29), Orangun et al. (30), Untrauer and Warren (31)
Splitting:	Morita and Tetsuzo (32), Losberg and Olsson (33), Ferguson (34)
Effect of ribs:	Soretz and Hölzenbein (35)
Tests and design implications:	Kemp (36)
Dowel effect:	Kemp and Wilhelm (37), Johnston and Zia (38), Acharya and Kemp (39), Jiminez et al. (40)
Influence of normal pressure:	Untrauer and Henry (41)

To the above must be added the papers of the International Conference on Bond in Concrete held in Scotland in June 1982 (see ref. 26). The papers covered slip measurement by holographic interferometry and by speckle interferometry using a fringe pattern for measurement, the effect of loading rate and of elevated temperatures on bond, time dependence of bond and cyclic bond testing which is dealt with later in this report.



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

Fig. 1 Bond Stress in Beam Carrying a Positive Shear



Splitting cracks and ultimate failure. (a) Typical. (b) Very wide beam. (c) With closely spaced bars

Fig. 2 Splitting Cracks and Ultimate Failure

(Figs. 1 & 2 have been reproduced from reference (1))

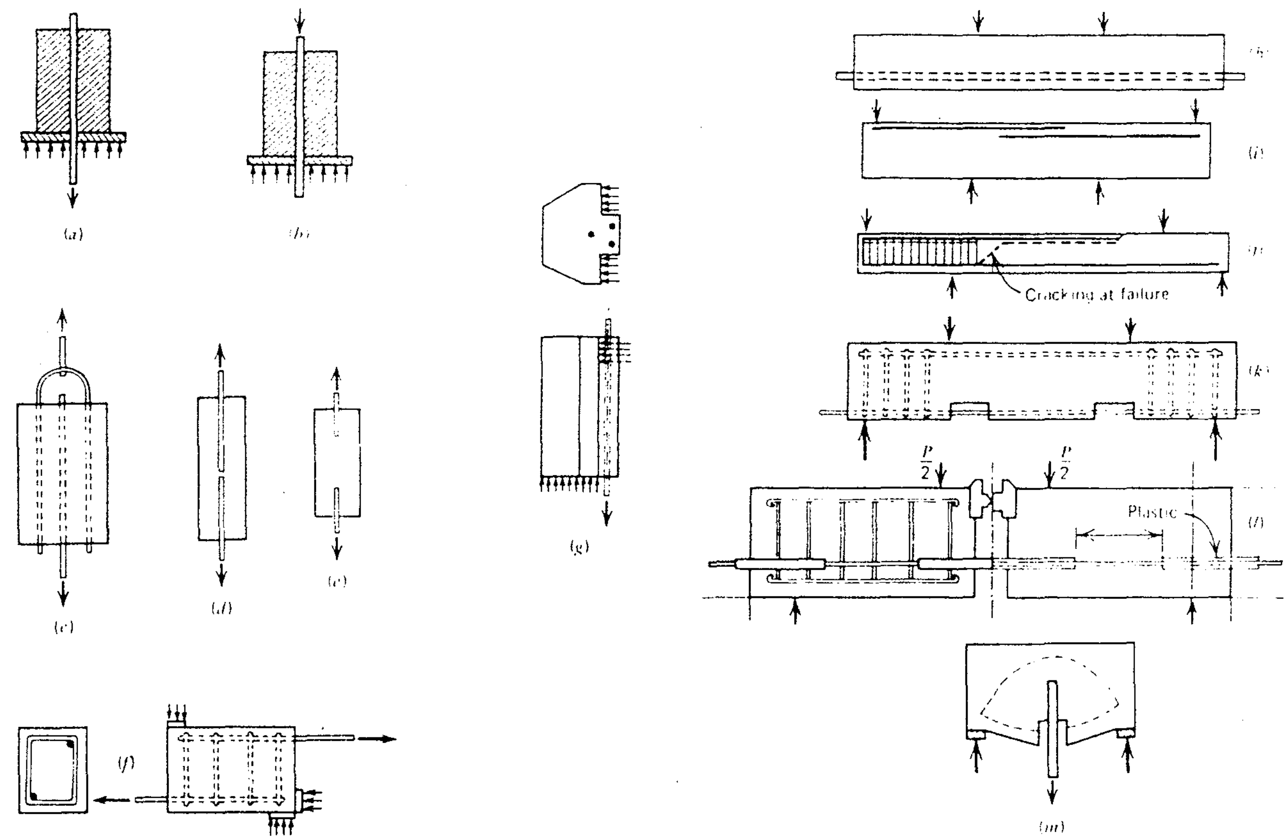
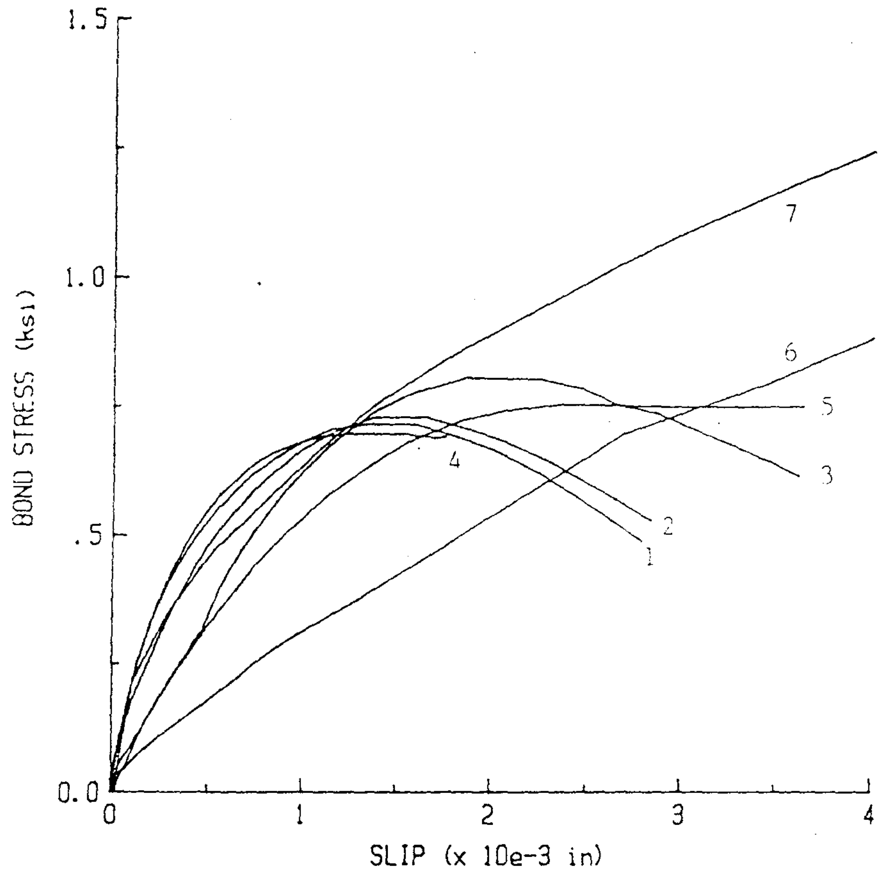


Fig. 3 VARIOUS BOND TEST METHODS

(reproduced from reference (3))



NO.	REF.	TEST TYPE
1	Nilson	18" Tension
2	Wahla	18" Tension
3	Tanner	18" Tension
4	Houde	16" Tension
5	Dorr	18" Tension
6	Duarte	Short Pullout
7	Edwards	Short Pullout

Fig 4 Bond Stress-Slip Relationships-Monotonic Loading

(reproduced from reference (8))

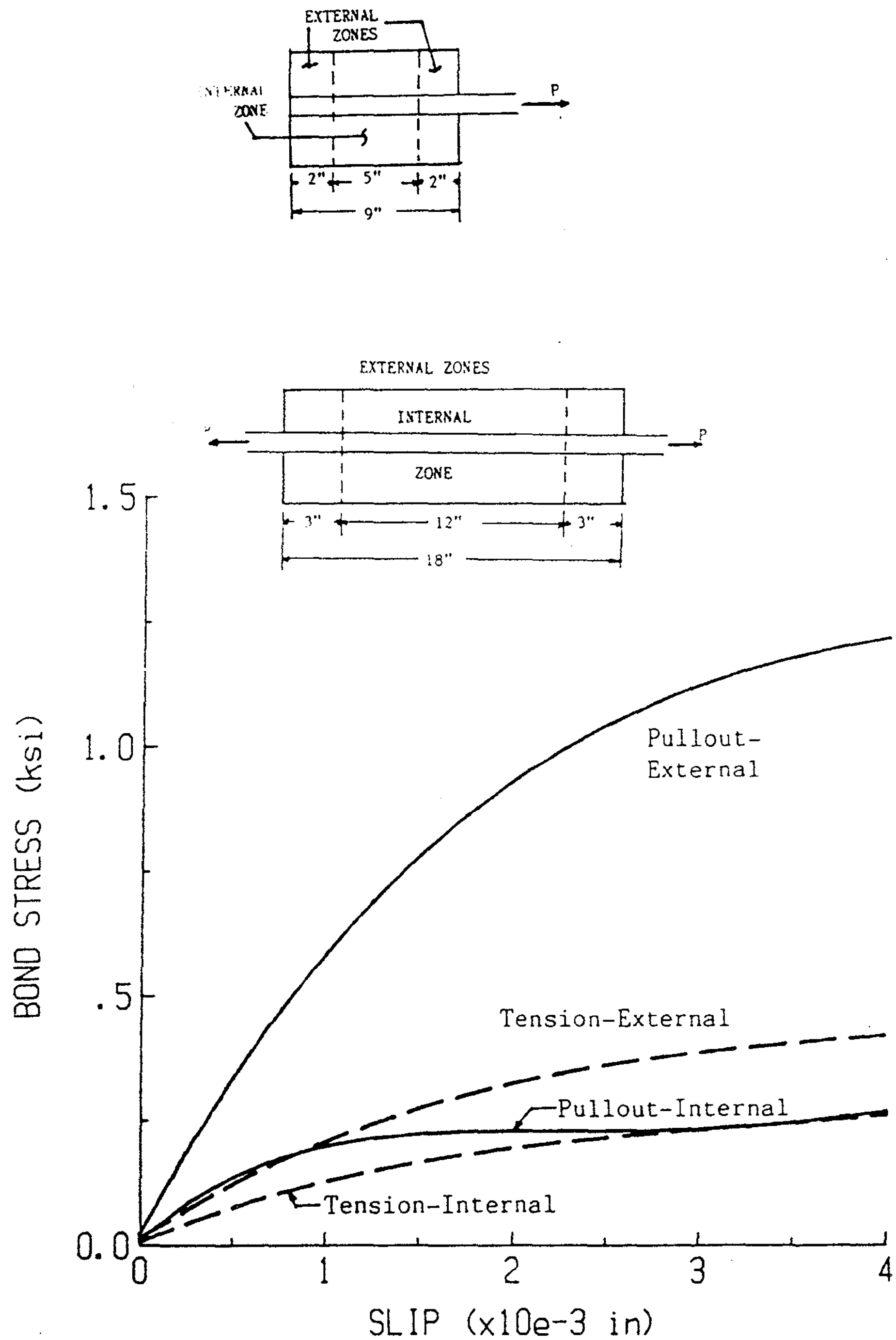


Fig 5 Bond Stress-Slip Relationships of Lahnert-Monotonic Loading

(reproduced from reference (8))

BOND IN REINFORCED CONCRETE SUBJECTED TO REPEATED, CYCLIC OR SIMULATED SEISMIC LOADS

Tests and Analytical Studies at the University of California, Berkeley

In 1968 Bresler and Bertero (25) investigated bond behaviour of concrete cylinders reinforced axially with the bar extending outside the concrete at both ends. The bar was repeatedly loaded in tension. They used the technique of Mains (42) in which the reinforcing bar is instrumented by cutting it longitudinally into two halves and a 1/2 in. wide by 1/8 in. groove milled in each of the two parts. Electrical resistance wire strain gauges are secured in the groove and waterproofed and the two halves of the bar are tack welded together to form a bar which has an exterior almost identical to a standard solid deformed bar.

Bresler and Bertero (25) showed that stress transfer from steel to concrete at any given stress level is influenced by the previous stress history.

They hypothesize that

> "The basic mechanism of bond deterioration is a failure in the concrete "boundary layer" adjacent to the steel-concrete interface. This failure occurs when the high local stresses reach critical values, and inelastic deformations and/or local fracture take place. Due to the nature of the failure surface and to the interlocking of the deformed bar lugs with the surrounding concrete, shearing stresses below the critical value may be transmitted by friction and by wedging action. Some slippage of the steel relative to concrete takes place and is accompanied by inelastic deformation and local crushing at the steel-concrete interface. Also the concrete undergoes some inelastic extensional deformation resulting from cracking, release of shrinkage, and local crushing. Upon slight unloading, the reverse motion between steel and concrete is resisted primarily by the wedging action of the rugged surfaces in the boundary layer. This generates a resistance to slip initially greater than that during the preceding loading stage, but with further unloading this wedging action is overcome and the resistance to slip, primarily due to

friction, is about the same as that during loading. With all of the external tension load removed, the full recovery of steel elongation is prevented by the shear resistance at interface between concrete and steel. This results in a residual state of tension in reinforcing steel and of over-all net compression in concrete. Locally cracks in the boundary layer as well as those extending fully through the section do not close completely after unloading as a result of irrecoverable deformation.

The over-all lag in slip recovery during unloading leads to the development of a hysteresis loop in the slip-load curve. It is believed that a major part of the residual slip is due to wedging action of the interface (boundary) layer.

With repeated cycles of loading and unloading, provided the tension in the steel does not exceed the previous maximum, there may be some further disruption in the boundary layer. Based on the limited evidence obtained the tests reported herein, and for the low stress levels and a number of cycles, the rate of disruption with cycling is low. With a few cycles the process appears to stabilize almost to a conservative system.

The effectiveness of bond between concrete and steel depends on the given level of tension stress in the steel and on the magnitude of the previous maximum tension. The greater the magnitude of the previous maximum tension, the greater is the disruption of the boundary layer, and the lesser is the effectiveness of the bond at lower levels of tension. When the previous maximum tension is considerably greater than the working stress level, even a small number of cycles of this high tension reduces the bond effectiveness at the working stress level. However, within the levels of stresses and limited number of cycles investigated in this study, the bond effectiveness at the level of maximum tension is not greatly affected by repetitions of maximum load."

The consequences of the deterioration of bond in a constant moment area are not serious for they affect stiffness and crack widths only. The behaviour of the anchorage zone under repeated loads is much more important because it may affect strength.

Experiments and an analytical approach to bond and anchorage under reversed cyclic loading is discussed by Popov (43). The main thrust of the paper is directed at the behaviour of bars in well confined concrete, particularly the case of beam bars cyclically pulling through columns at interior joints. Popov comments,

> "in the inelastic range, and particularly under cyclic loading, there is a progressive loss of bond between the reinforcing bars and the concrete. Moreover, the bars stretch significantly at yield. These effects give rise to the development of beam cracks. Such cracks, together with the major one at the beam-column interface, can cause a large fixed-end rotation relative to a column axis."

He gives the test results of a short cantilever where this fixed end rotation, at a ductility ratio of 4.6, accounted for 25% of the cantilever tip deflection.

The bond force to be disposed of by the beam bars embedded in the column is the net force in the bar at the column faces. Hence, with an internal beam-column joint, subjected to cyclic loading, the bond force can reach twice the yield force of the bar as there can be tension yield on one side of the column and compression yield on the other. With this particularly high bond stress within the joint, the anchorage of the bar can be destroyed and the loosened bar can move back and forth within the column core with ease.

Popov (43) reports bond tests (44) on long bars cast within a typical column stub but remarks that it is very difficult to generalize the results to make them applicable to members of any size. An attempt was then made to assign a local bond stress - slip law for various points on a long bar so that any member with any set of boundary conditions could be studied analytically. As bond stress is determined experimentally by taking the difference of axial strains at adjoining points along the bar, the results are very sensitive to the obtained strain readings which tend to be erratic. While smoothing using a least squares method assisted it was an extremely time-consuming process.

The solution adopted by Eligehausen et al. (45) was to investigate bond of short bars, limited to five bar diameters. The authors assumed that as the bar is short "the calculated average bond stress may be considered as representative of a local bond stress." Further as the specimen was relatively large in comparison with the critical splitting area of the concrete, a well confined environment may be assumed.

In monotonic load tests, the bond stress-slip relationship for tension and for compression were almost identical. The descending branch of the bond stress-slip curve (see Fig. 6) levelled off at a slip approximately equal to the clear distance between protruding lugs of the bar. Failure was caused by pull-out well below the bar yield stress. The bond strength was increased by restraining reinforcement, increase of the transverse applied pressure and increase of the clear distance between bars.

During cyclic loading, the degradation of bond strength and bond stiffness depends primarily on the maximum value of peak slip in either direction reached previously. The number of cycles and the difference between the peak values of the slip between which the bar is cyclically loaded are also significant parameters. For instance, up to 10 load cycles between slip values corresponding to bond stresses less than 80% of the maximum resistance attained under monotonically increasing slip only slightly reduces the bond resistance at the peak slip value as the number of cycles increase. It does not affect the bond-slip behaviour at larger slip values. But pronounced deterioration of the bond stiffness (see Fig. 6) occurs when cycling is between slip limits larger than that corresponding to the "80% bond stress", at slip values smaller than the peak slip value. Furthermore, the bond stress-slip behaviour at larger slip values is distinctly affected. If the deterioration is related to the pertinent monotonic envelope, then the behaviour of bond during cyclic loading is not significantly affected by the parameters mentioned in the previous paragraph.

The test results were used to deduce an analytical model (45,46,47) for the local bond stress-slip relationship valid for confined concrete under generalized excitations (see Fig. 7). The authors write,

> "The bond stress-slip curve follows the "monotonic envelope" valid for monotonically increasing slip (paths OABCD or $OA_1B_1C_1D_1$). Imposing a slip reversal at an arbitrary slip value, a stiff "unloading branch" and the "friction branch" with $\tau = \tau_f$ are followed successively up to the intersection with the curve OA'_1 (path EFGHI). Then the "reduced envelope" (curve $OA'_1B'_1C'_1D'_1$) which is similar to the virgin monotonic curve but with reduced values of τ is followed (path IA'_1J). When reversing the slip again at J, unloading branch, frictional branch and "reloading branch" (same stiffness as the unloading branch) are followed successively up to the intersection with the reduced envelope O A'B'C'D' (path JLNE'), which is followed thereafter (path E'B'C'). If instead of increasing the slip beyond point N more cycles between the slip values corresponding to the points N and K are imposed, the bond stress slip relationship is like that of a rigid plastic model, but with decreasing frictional bond resistance as the number of cycles increases."

Comparisons of local bond-slip relationships were made between experimental results and predictions of the model. The agreement was satisfactory except for the re-loading curves near values of the peak slip between which the specimen was cycled.

Ciampi et al. (46) contend that the model can easily be extended to cover bond of bars with different bar diameters, lug patterns, concrete strengths, degrees of confinement and transverse pressures provided that the pertinent experimental data necessary for computing the different parameters, in particular the monotonic envelope, is obtained.

A similar experimental and analytical program was conducted on deformed hooked bars by Eligehausen et al. (48). The 25mm diameter bars were bent through a right angle and embedded $1d_b$ before and $5d_b$ after the bend respectively. They found that the hook improved the bond behaviour and that under monotonic loading the resistance of hooks after reaching the maximum value is almost constant over a large slip range. Cyclic loading produces a significant deterioration of strength and stiffness of the anchored hook at slip values smaller than the peak slip values between which the hook is cyclically loaded. However there is not much influence on the force-slip behaviour at slip values larger than the peak values during previous cycles. Using this experimental information, a modified analytical bond stress-slip law for hooked bars was derived and it was found to give satisfactory agreement with experimental results under various slip histories.

The next phase was to generalize the analytical model for straight bars, to cover bond conditions found in joints of reinforced concrete frames subjected to severe earthquake loadings. Ciampi et al. (46) attempted this using the bond stress-slip model for confined concrete and found that the quantitative agreement between observed and calculated response needed improvement. In later work (47,49) they took into account the different bond conditions along the embedment length in an interior joint recognizing that at the ends the concrete is not confined. This affects the monotonic envelope as does the slips at each end, one being positive and the other negative. Different models were derived for these end conditions. They used a bi-linear stress-strain relationship for the bar as they found this to be computationally more economical than a non-linear model while not sacrificing accuracy. The actual behaviour of the bar was idealized as a one-dimensional problem and modelled using an ordinary first order non-linear differential equation. With boundary values specified at the bar ends, it becomes a non-linear two point boundary value problem which was solved using a "shooting technique". The comparison between the analytical predictions and the experimental results was "sufficient for practical purposes". In particular, the force-slip relationships for the pulled bar end were predicted satisfactorily (for example, Fig. 8). The distribution of steel strain, normal force, slip and bond force along the embedment length agree less well (see Fig. 9).

The tests also showed that superior behaviour (i.e., less damage) of anchorages was obtained with steel of lower yield stress and/or lower strain hardening ratio. This is due to the lower steel stresses at given peak slips requiring smaller bond stresses for force transfer. It was also found that bond damage increases with decreasing anchorage length.

Filippou et al. (50) slightly modified the original bond stress-slip model for a better account of the behaviour during reloading. They used the model to describe the hysteretic behaviour of reinforced concrete beam-column joints. Using new models for concrete and steel, taking into account the Bauschinger effect, they solved a system of non-linear equations by a modified Gauss-Seidel non-linear iteration scheme and found good agreement between analysis and experiment. The authors stated

> "that the basic features of the presented method, being of a general nature, can be used in many other situations where cyclic bond-slip behaviour of reinforcing bars has to be taken into account."

Other Tests and Analytical Studies

Perry and Jundi (51) subjected #6 reinforcing bars, in eccentric pull-out tests, to static and dynamic loading. The peak bond stresses tended to shift from the loaded end of the specimen to the unloaded end as the number of cycles of loading and unloading increased but the redistribution tended to become stabilized after several hundred cycles. A small apparent reduction in the average bond stress at ultimate was caused by the repeated loadings.

The behaviour of anchored bars supporting a cantilever beam was studied by Ismail and Jirsa (52) in a simulation of the anchorage conditions of an exterior beam-column joint. The beam was subjected to monotonic, repeated or reversed load cycles to failure. Constant normal pressure was applied on the end block to simulate column loads. While stress distributions and depth of yield penetration (14 to 18 bar diameters) did not exhibit large changes with increasing number of cycles, the bar elongation, measured in the cantilever just beyond the end block, gradually increased with increasing end deflection and varied considerably for the various loading cases. In the monotonic and repeated loading cases, elongations of the anchored bars contributed 30-45% of the total end deflection while under reversed load the contribution was up to 60%.

In an earthquake it is quite possible that the end anchorage may not be subjected to such a favorable compression and the effect of penetration of yielding into the anchorage zone of diminished effective development length available to absorb the yield strength of the bar may be crucial.

Bond-slip relationships for very small relative displacements, under 0.1 mm, were found by Giuriani (53) using pull-out tests with a bar having two ribs embedded in the concrete. Cyclic loading with and without sign reversal was used.

Hawkins et al. (54) conducted tension-tension bond tests on short specimens using both monotonic and cyclic loading. They found that with two lugs embedded greater maximum stresses and more consistent results were obtained than with either one or four lugs embedded. From the experimental results they developed tri-linear monotonic and modified cyclic response models suitable for integration. The authors stated that Lin (55)

> "demonstrated experimentally and analytically that the load-slip response for an inelastically and reversed cyclically-loaded bar can be modelled by integration of the local bond-slip relationship of the bar, the stress-strain relationship for the bar and conditions of continuity of forces in, and displacements along, the bar."

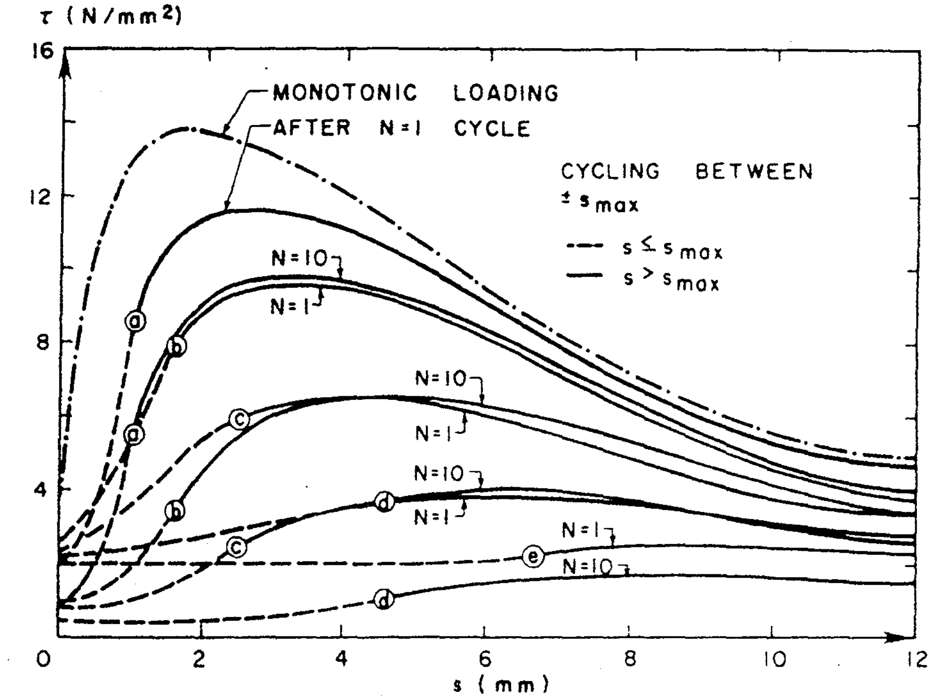


Fig. 6 EFFECTS OF NUMBER OF CYCLES AND OF THE PEAK VALUES OF SLIP s_{max} AT WHICH THE CYCLING IS PERFORMED ON THE ENSUING BOND STRESS-SLIP RELATIONSHIP FOR $s > s_{max}$

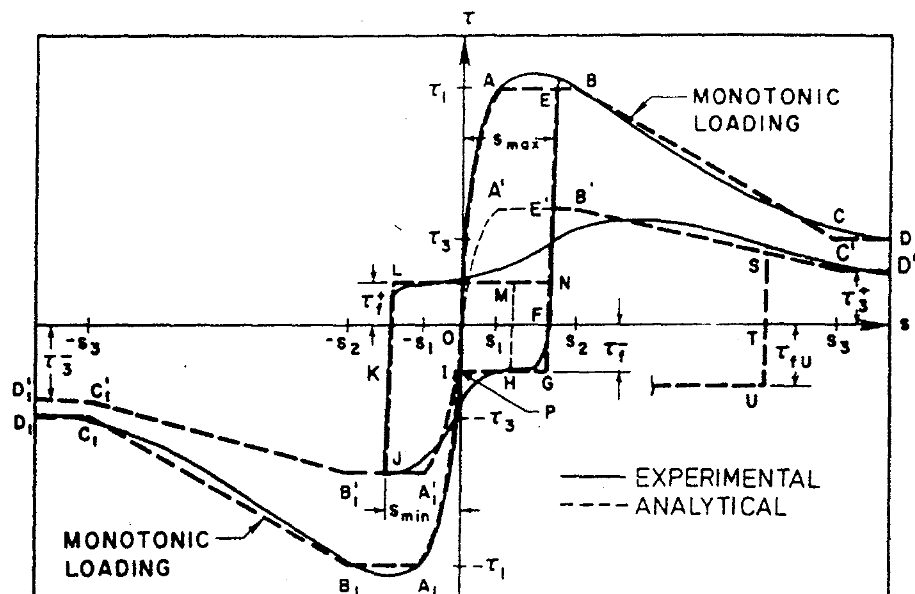
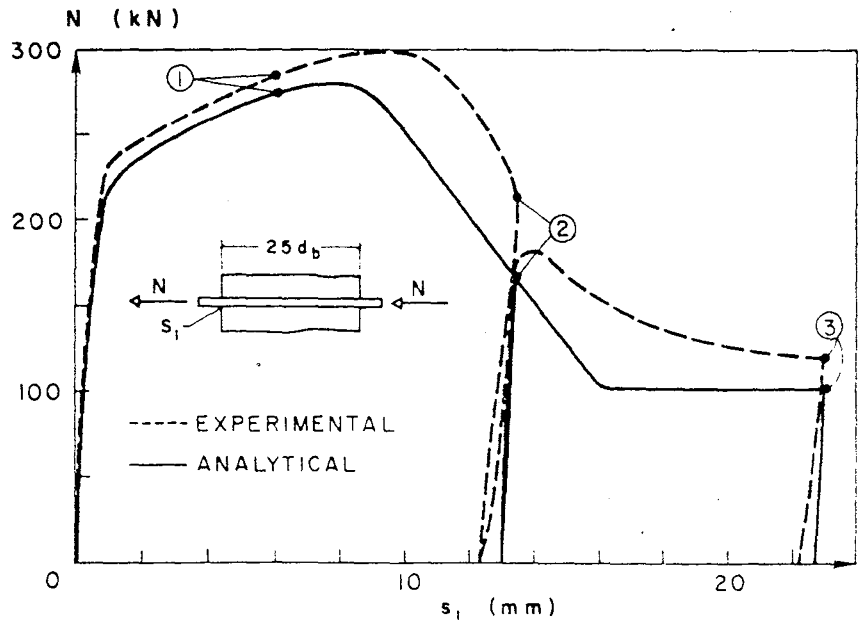
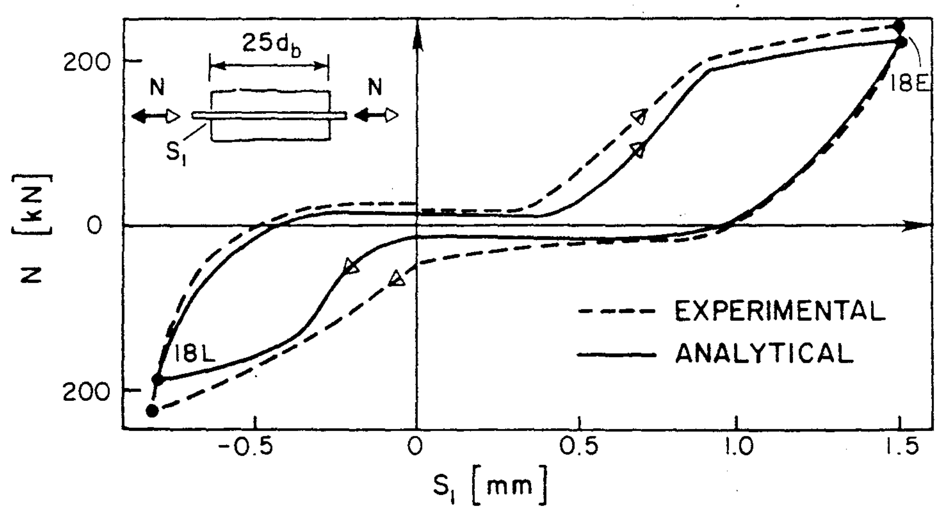


Fig. 7 PROPOSED ANALYTICAL MODEL FOR LOCAL BOND STRESS-SLIP RELATIONSHIP

(Figs 6 & 7 reproduced from reference (47))



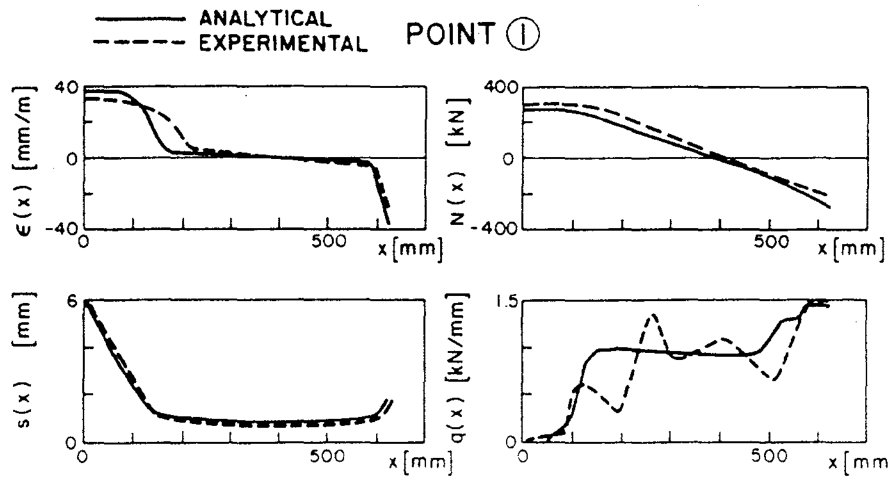
(a)



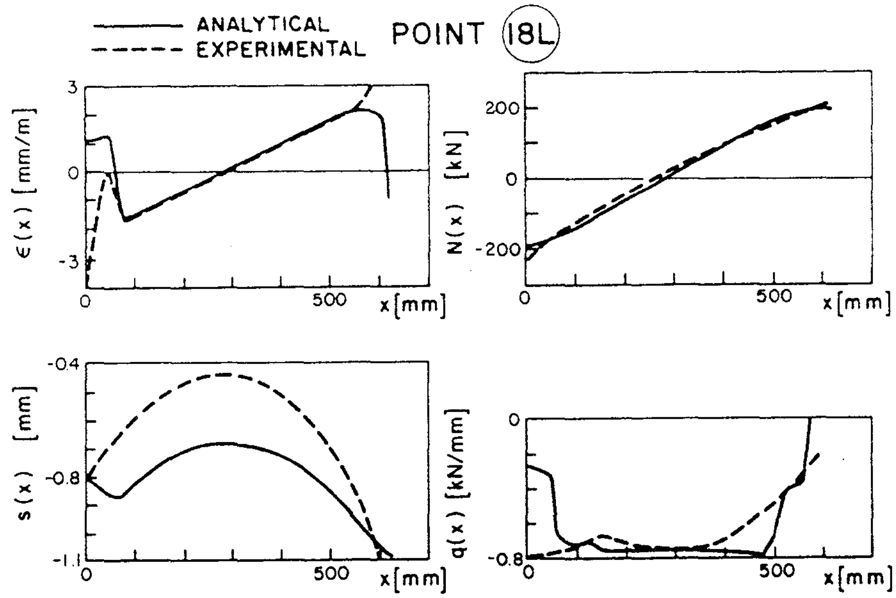
(b) CYCLE 18

Fig. 8 Force-Slip Relationships for Pulled Bar End in Reinforced Concrete-Comparison Between Results from Analysis and Experiment (Bond Resistance Increased 10% Compared to Average Behavior)

(reproduced from reference (47))



(a) Point 1 as in Fig. 8(a)



(b) Point 18L as in Fig. 8(b)

Fig. 9 Reinforced Concrete Bond - Distribution of Steel Strain $\varepsilon(x)$, Slip $S(x)$, Normal Force $N(x)$ and Bond Force $g(x)$ along the Anchorage Length - Analytical and Experimental Results for Characteristic Load Stages.

(reproduced from reference (47))

BOND IN REINFORCED MASONRY

Failure modes, tensile capacity and bond-slip characteristics of monotonically loaded grade 60 deformed reinforcement anchored in grouted concrete masonry were investigated by Cheema and Klingner (56). Their test set-up consisted of a single wythe 8 in. wall with a vertical bar. The length of the wall enabled the reactions of the loading ram to be placed far away from the reinforced block by means of a steel beam. The span of the beam was chosen so that the vertical compressive stresses in the masonry near the bar would not exceed 100 psi at yield of the bar and so would not influence significantly the anchorage behaviour. As this arrangement creates flexural tensile stresses in the horizontal direction, a second hydraulic actuator applied an eccentric horizontal compression to the wall specimen to give small resultant compression stresses at the bar. The wall height was adjusted to give desired embedment lengths to the bar. Slips were measured using slip wires (57) connected to linear potentiometers.

Cheema and Klingner (56) found that failure of the anchorage occurred in one of three modes or in a combination of two of them. The first two, pull-out and splitting, reflect reinforced concrete behaviour but the third, block uplift is unique to reinforced masonry. Block uplift involves the lifting off of masonry courses from a bed joint and across head joints adjacent to the bar.

Pull-out failures, which are the breakdown of the interlock between the grout and the bar, were observed only with #4 bars. Splitting failures, involving longitudinal cracks on the surfaces of the specimen and numerous cracks in the grouted core extending radially from the bar, were observed for some #8 and #11 bars. Block uplift predominated for #11 bars but occurred for some #8 bars.

The authors observed that the load-slip behaviour varied with the length of embedment. In the short embedment cases (e.g., 20 in. embedment for a #8 bar), at 40% of the failure load, the leading third and the middle third of the embedment transfer most of the load equally (see Fig. 10). Increasing the bar tensile force causes the contribution of the leading third of the embedment to gradually reduce to zero (at failure) as the slip propagates toward the tail end. The average nominal bond strengths for #4, #8 and #11 bars were 0.29, 0.16 and 0.15 respectively of the compressive strength of the grout as measured from specimens poured in porous moulds. The bars did not reach yield at failure.

For intermediate length embedment (e.g. 28 in. embedment for a #8 bar) yield did occur with some tail end slip. At 30% of the failure load, the leading third of the embedment was transferring approximately one third of the load and the bond stress level in that portion of the bar remained reasonably constant to failure. Bond stress levels in the middle and tail thirds gradually increased to failure but the tail third stress had not yet peaked (see Fig. 11).

Bars with long embedment length (e.g. 43 in. embedment for a #8 bar) experience a different stress distribution (see Fig. 12) and exhibit a significant reserve capacity. In the leading quarter of the embedment, as the tensile load increases the bond stress peaks and then falls to zero at a higher load. At failure the second and third quarters of the embedment have peaked while the tail quarter carries very low bond stresses.

Cheema and Klinger followed up their experimental paper with one on failure criteria (58) in which simplified bond stress distributions at failure were adopted in a linear elastic analysis using a finite element solution. Capacities and failure modes agreed favourably with their test results. A further paper (59) extended their work into design recommendations.

Watanabe (60) using standard pull-out tests investigated bond and anchorage of deformed bars in grouted clay bricks and concrete blocks and in control specimens of concrete. He reported:

(i) Bars in concrete masonry were anchored as effectively as in concrete;

(ii) Clay brick masonry bars did not anchor as effectively as bars in concrete. Watanabe considered this to be due to the very high absorbtivity of the clay bricks which had deleterious effects on the grout;

(iii) For concrete block masonry where the embedment length exceeded 14 bar diameters, the ultimate anchorage load exceeded the yield load. The failure was in bond slip for bar diameters less than 19 mm (3/4 in.) while for bars of diameter 22 mm (7/8 in.) splitting failures dominated;

(iv) Variation in the cover of the grout from 10 to 60 mm (0.39 to 2.36 in) had an almost negligible effect on the ultimate anchorage load obtained.

Tests on lap splices in concrete masonry in which 19 mm (3/4 in.) bars are cast within stack bonded piers and tensioned from both ends are reported by Kubota (61). Various cases were tested viz. bars continuous from one end to the other, bars lapped, and both of the former with either joint reinforcement or spiral reinforcement or both. Elongations were measured on two surfaces of the blocks at the mortar joints and at the specimen ends. With two specimens in each case, one was loaded monotonically and the other carried one-way repeated loading.

Kubota found that specimens without lap splices deformed almost identically up to yield as the reinforcement on its own and the distribution of the elongations were approximately uniform along the length of the specimens. Lap spliced specimens, however, exhibited "soft spring behaviour", and elongations (slips?) were much greater at the two ends than they were in the middle of the specimen. The author in commenting on this difference in behaviour of lapped and continuously reinforced specimens states that

> "In walls subjected to flexural tension the crack width of the case of the lap joint will be little more concentrated at the bottom than that of the case without lap joint. But the crack widths on the lap joint and the total elongation which may affect the plastic rotation capacity of the wall in case with lap joints are smaller than that in case without lap joints."

No difference was observed between the behaviour of specimens with and without joint or spiral reinforcement or both although the author states," the splitting speed was faster for specimens with no surrounding reinforcement."

Some preliminary tests on the cyclic behaviour of bed joint reinforcement in masonry have been reported by Modena and Cecchinato (62) and the authors observe that the cyclic behaviour was very similar to that obtained with reinforced concrete. They believe that their work eventually will lead to a model such as the model of Ciampi et al. (46) for reinforced concrete. Their references give four papers (63,64,65,66) on monotonic tests of bed-joint reinforcement bond.

Suter and Fenton (67) tested 13 full-scale concrete masonry walls with the reinforcement spliced at wall mid-height with various lap lengths. Monotonic loads were applied laterally to the walls at their third points so the splices were in a constant moment and zero shear region. The typical test behaviour involved the yielding of the steel with resultant large wall deflections in excess of span/24. No bond failures occurred even though the splice lengths were significantly lower than those recommended by ACI for reinforced concrete design. The authors found that the masonry compressive strength f_m, (the masonry equivalent of the concrete compressive strength, f_c) is not a reliable indicator of ultimate bond strength and that a better indicator was the compressive strength of the fill.

The paper (67) refers to work by Baynit (68) which indicated a significant reduction in ultimate bond stress with increasing number of bars in a layer within hollow reinforced concrete masonry. When mortar was used instead of grout as the fill material a 10% reduction of bond stress resulted (unfortunately it was not recorded whether or not this was due to reduced compressive strength of the fill).

In a further paper, Suter and Fenton (69) describe a flexural test program on reinforced masonry at Carleton University, Ottawa. The thrust of the program was on flexural capacity of reinforced beams, monotonically loaded, and not on bond although one bond failure occurred. Their references contain several other papers on reinforced masonry in flexure but from the titles it does not seem as if any have a prime interest in bond.

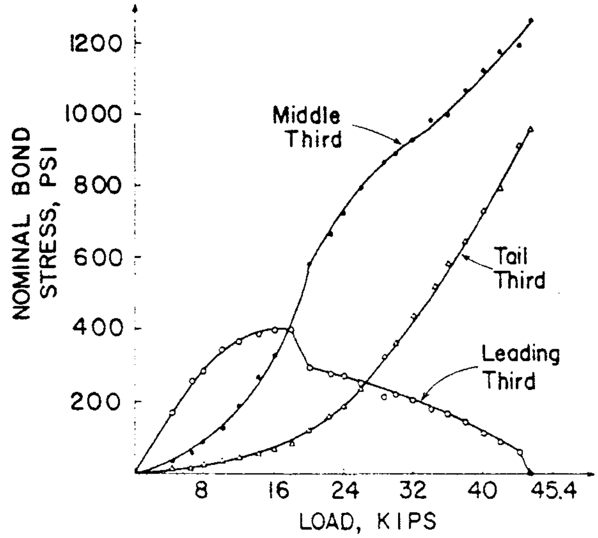


Fig. 10 *Nominal bond stress variations for No. 8 bar with short embedment (Specimen #8-20-2-II)*

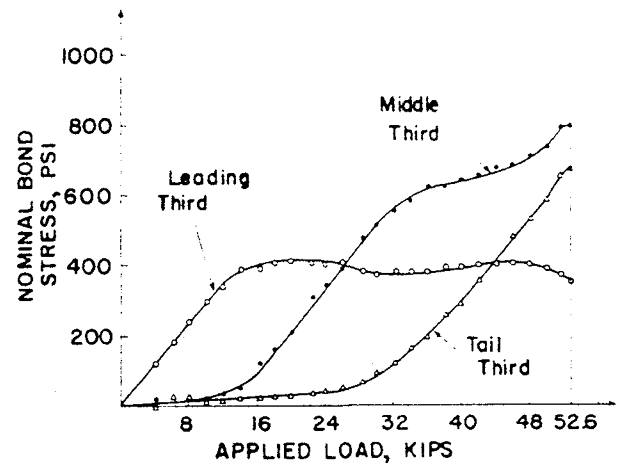


Fig. 11 *Nominal bond stress variation for No. 8 bar with intermediate embedment (Specimen #8-28-2-II)*

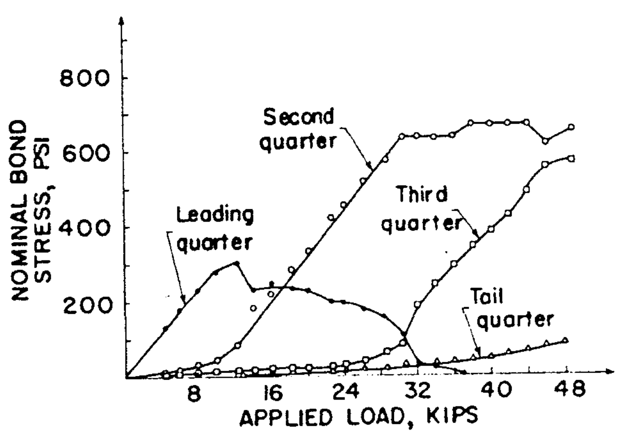


Fig. 12 *Nominal bond stress variation for No. 8 bar with long embedment (Specimen #8-43-2-II)*

Figs. 10, 11, 12 Bond Stresses in Monotonically Loaded Concrete Masonry (reproduced from reference (56))

DISCUSSION

Inevitably this report has mainly considered bond in reinforced concrete as the research there has been so much greater than research in bond on reinforced masonry. However, so many papers on reinforced masonry, including some discussing bond, have stated that it behaves in the same general way as reinforced concrete and the authors have all been prepared to use reinforced concrete reasoning and formulae, with suitable changes to masonry terms, to describe reinforced masonry behaviour. The major exception is in the paper of Cheema and Klingner (56) who in modified pull-out tests found that a further failure type occurred with masonry specimens in addition to the well-known reinforced concrete specimen failures of pull-out and splitting. Block uplift, or progressive "unzipping" of the concrete masonry units from the specimen commencing from the upper and loaded end, dominated for the larger bars, #11, and occurred for some #8 bars. Always keeping this uniquely masonry failure in mind, it would seem that reinforced masonry can borrow very heavily from knowledge of bond behaviour in reinforced concrete.

Bond in Reinforced Concrete

The current most pressing problem is the determination of a bond stress-slip relationship which can be applied to particular structural elements including connections. It is obvious from the tests that a unique law for all situations is not possible as bond depends crucially on the particular stress situation in each direction and so the relationship will change according to the situation. This has the important consequence that tests must be conducted on the actual structural element or connection, or on an accurate simulation of it, in order to determine the true bond action. The pull-out test is a prime example of a test which seldom simulates the actual conditions and hence the results of pull-out tests must be considered with extreme caution.

Again as bond is so dependent on the actual circumstances applying, it is unlikely that any single local bond stress-slip law can apply along the length of a structural element or connection. Ciampi et al. (47,49), Lahnert (13) and others have realised this and indicated that the bond stress-slip law must vary, for instance, according to the confinement of the reinforcing. Lahnert usefully defines confinement to mean the radial restraint on the bar whether provided by large concrete cover, secondary reinforcement or applied pressure.

This has led to two different approaches:

(i) the determination of local bond stress-slip laws at various points along the structural member, by embedding only a short length of the bar at the point of interest. The overall effect is the summation of all local effects.

This technique suffers from the problem of knowing where on the structural member it is necessary to transfer from one local bond stress-slip law to another, without requiring a multitude of local laws. Further, it is possible that confined conditions occur on the short embedded length tested and this may not echo the real conditions in the structural element.

(ii) the determination of bond stress and of slip at various points along the reinforcement which is all fully embedded in the concrete as it is in the structure.

It has been shown (13,43) that it is very difficult to determine bond stress accurately as it is usually obtained from strain measurements leading to bar forces and hence bond stresses. The bond stress depends so much on the exact distribution of strain requiring very closely spaced strain measurement. Slip is relatively easier to measure with accuracy, even internally, and there are many techniques now available for accurate measurement.

Various analytical approaches to find a bond stress-slip law for reinforced concrete have been attempted including finite element solutions. A simple and different approach of promise is that of Jiang (19). He showed that equilibrium, compatibility and stress-strain relationships of a cracked segment are one equation insufficient to enable a solution. The usual approach is to find or assume a local bond stress-slip law and the problem eventually requires the solution of a differential equation which is difficult to solve particularly if the bond law is non-linear. Jiang's alternative is to assume a bond stress distribution, and he showed that various shapes affect the solution little, so he took a simple one in a second degree parabola. Now the solution can be obtained by integration. Apparently the results agreed well with experimental ones.

Although general considerations in bond behaviour follow through from the monotonically loaded to the reverse cyclically loaded situation, the bond stress-slip law is much more complicated in the latter case. Account needs to be taken of the deterioration of bond which occurs when the specimen is cycled at high slips and when there are many cycles. The effects of reduction in bond stress and bond stiffness must be considered. The model (45-50) of the University of California, Berkeley, has given satisfactory results, as compared with experiment for a reinforced concrete beam/column joint i.e. in a situation where most of the structural element is confined. It is a complex model and requires much computation.

Perhaps the simpler model of Hawkins et al. (54) using a tri-linear monotonic response modified for cyclic response and suitable for integration may be able to be developed further for use in reinforced masonry bond.

Tests (48) on hooked bars under reverse cyclic loading not only showed superior bonding over straight bars but also that hooked bars maintain their resistance over a large slip range whereas straight bars give a reducing bond resistance with slip after a maximum is reached.

An important point emerged from the Berkeley tests (47) in that less damage occurred in cyclically loaded specimens with steel of lower yield stress and/or lower strain hardening ratio. This is due to lower steel stress at a given peak slip requiring a smaller bond stress for force transfer.

Bond in Reinforced Masonry

Reinforced concrete bond research, certainly in the seismic case, has concentrated on the confined reinforcement situation. In a reinforced masonry wall neither the vertical nor the horizontal steel is in a confined situation. The geometry is such that secondary reinforcement cannot be present and there is little cover (assuming that the thickness of the masonry unit is not as effective as concrete in restraint). There may, however, be some transverse pressure from the vertical loads on the wall, but this will be lower in general than in a reinforced concrete column.

The stress situation in a masonry wall is also somewhat different from a reinforced concrete beam. For the vertical steel, particularly that on the periphery of a wall, the flexural situation is not dissimilar to a beam but the shear distribution from seismic horizontal loads is considerably different. The horizontal steel in the wall has no directly comparable counterpart in reinforced concrete, except of course in a wall.

The bond situation at splices in reinforced masonry needs further consideration. Suter (67) has tested some splices under monotonic loading and Kubota (61) has conducted a few tests under cyclic loading.

It is encouraging to read in the paper of Ciampi et al. (46) that the authors believe that the Berkeley model can easily be extended to cover bond of bars of different bar diameters, lug patterns, concrete strengths, degrees of confinement and transverse pressures provided that the pertinent experimental data necessary for computing the different parameters, in particular the monotonic envelope, is obtained. In their proviso should be the prompting for further tests on reinforced masonry bond.

ACKNOWLEDGEMENTS

The report was prepared in Boulder, Colorado while the author was on Special Studies Programme from the Department of Architecture and Building, University of Melbourne, Australia between January and July 1986. The support of the University of Melbourne, the University of Colorado, Boulder and Atkinson/Noland and Associates, Consulting Engineers, Boulder is much appreciated.

REFERENCES

1. ACI Committee 408., "Bond Stress - The State of the Art", ACI Journal, Proceedings, V. 63, No. 11, Nov. 1966, pp. 1161-1190.

2. ACI Committee 318., "Building Code Requirements for Reinforced Concrete (ACI-83)", and "Commentary on Building Code Requirements for Reinforced Concrete (ACI-83)", ACI, Michigan, 2nd printing, Sept. 1984.

3. Park, R., and Paulay, T., "Reinforced Concrete Structures", Wiley, 1975.

4. Jirsa, J.O., Lutz, L.A. and Gergely, P., "Rationale for Suggested Development, Splice and Standard Hook Provisions for Deformed Bars in Tension", Concrete International, July 1979, pp. 47-61.

5. ACI Committee 408., "A Guide for the Determination of Bond Strength in Beam Specimens", ACI Journal, Proceedings, V. 61, No. 2, Feb. 1964, pp. 129-135.

6. Mathey, R.G. and Watstein, D., "Investigation of Bond in Beam and Pull-out Specimens with High-Yield-Strength Deformed Bars", ACI Journal, Proceedings, V. 57, No. 9, Mar. 1961, pp. 1071-1090.

7. Ferguson, P.M., Breen, J.E. and Thompson, J.N., "Pullout Tests on High Strength Reinforcing Bars", ACI Journal, Proceedings, V. 62, No. 8, Aug. 1965, pp. 933-950.

8. Lahnert, B.J., "Direct Measurement of Slip between Steel and Concrete", M.S. Thesis, University of Colorado, Boulder, 1984.

9. Wahla, M.I., "Direct Measurement of Bond-Slip in Reinforced Concrete", Ph.D. Thesis, Cornell University, Ithaca, New York, 1970 (not viewed by author).

10. Tanner, J.A., "An Experimental Investigation of Bond Slip in Reinforced Concrete", M.S. Thesis, Cornell University, Nov. 1971 (not viewed by author).

11. Dörr, K., "Bond Behavior of Ribbed Reinforcement Under Transversal Pressure", Proc. IASS Symposium, Darmstadt, Germany, July 1978, pp. 13-24 (not viewed by author).

12. Mirza, S.M. and Houde, J., "Study of Bond Stress-Slip Relationships in Reinforced Concrete", ACI Journal, Proceedings V. 76, No. 1, 1979, pp. 19-46.

13. Jiang, D.H., Shah, S.P. and Andonian, A.T., "Study of the Transfer of Tensile Forces by Bond", ACI Journal, Proceedings V. 81, No. 3, May-June, 1984, pp. 251-259.

14. Rehm, G., "The Fundamental Law of Bond", RILEM Symposium on Bond and Crack Formation in Reinforced Concrete, Stockholm, Sweden, 1957, V. 1, pp. 491-498 (not viewed by author).

15. Edwards, A.D., and Yannopoulos, P.J., "Local Bond Stress to Slip Relation for Hot Rolled Deformed Bars and Mild Steel Plain Bars", ACI Journal, Proceedings V. 76, No. 3, Mar. 1979, pp. 405-420.

16. Nilson, A.H., "Bond Stress-Slip Relation in Reinforced Concrete", Report No. 345, School of Civil and Environmental Engineering, Cornell University, Ithaca, New York, Dec. 1971, (not viewed by author).

17. Duarte, I.C., d'A., "Bond Stress-Bond Slip Relations for Short-Bonded Concrete Pullout Specimens", M.S. Thesis, University of Colorado, 1983, 235 pp. (not viewed by author).

18. Nies, W., "A New Method for the Measurement of Local Slip Between Steel and Concrete", Ph.D. Dissertation, Technical University, Darmstadt, Germany, 1979 (not viewed by author).

19. Jiang, D., "Bond and Slip between Steel and Concrete", Journal of Tongji University, V. 1, 1984, pp. 17-30.

20. Venkateswarlu, B. and Gesund, H., "Cracking and Bond Slip in Concrete Beams", Proceedings ASCE, V. 98, No. ST12, Dec. 1972, pp. 2663-2686.

21. Edwards, A.D. and Picard, A., "Theory of Cracking in Concrete Members", op. cit., pp. 2687-2700.

22. Tassios, T.P. and Yannopoulos, P.J., "Analytical Studies on Reinforced Concrete Members under Cyclic Loading Based on Bond-Slip Relationships", ACI Journal, Proceedings V. 78, No. 3, May-June 1981, pp. 206-216.

23. Ngo, D., and Scordelis, A.C., "Finite Element Analysis of Reinforced Concrete Members", ACI Journal, Proceedings V. 64, No. 3, Mar. 1967, pp. 152-163.

24. Lutz, L.A. and Gergely, P., "Mechanics of Bond and Slip of Deformed Bars in Concrete", ACI Journal, Proceedings, V. 64, No. 11, Nov. 1967, pp. 711-721.

25. Bresler, B. and Bertero, V., "Behavior of Reinforced Concrete under Repeated Load", Proceedings ASCE, V. 94, No. ST6, June 1968, pp. 1567-1590.

26. Gerstle, W., Ingraffea, A.R., and Gergely, P., "Tension Stiffening: A Fracture Mechanics Approach", Proceedings, International Conference on Bond in Concrete, Scotland, June 1982, published in "Bond in Concrete ", P. Bartos, Ed., Applied Science Publishers, 1982, pp. 97-106.

27. deGroot, A.K., Kusters, G.M.A., and Monnier, Th., "Numerical Modeling of Bond-Slip Behavior ", Heron, V. 26, No. 1B, 1981, 90 pp. (not viewed by author).

28. Vos, E., "Influence of Loading Rate and Radial Pressure on Bond in Reinforced Concrete ", Delft University Press, Oct. 1983, 235 pp. (not viewed by author).

29. Thompson, M.A., Jirsa, J.O., Breen, J.E. and Meinheit, D.F., "Behavior of Multiple Lap Splices in Wide Sections", ACI Journal, Proceedings, V. 76, No. 2, Feb. 1979, pp. 227-248.

30. Orangun, C.O., Jirsa, J.O. and Breen, J.E., "A Re-evaluation of Test Data on Development Length and Splices", ACI Journal, Proceedings V. 74, No. 3, Mar. 1977, pp. 114-122.

31. Untrauer, R.E. and Warren, G.E., "Stress Development of Tension Steel in Beams", ACI Journal, Proceedings V. 74, No. 8, Aug. 1977, pp. 368-372.

32. Morita, S. and Tetsuzo, K., "Splitting Bond Failures of Large Deformed Bars", ACI Journal, Proceedings V. 76, No. 1, Jan. 1979, pp. 93-110.

33. Losberg, A. and Olsson, P., "Bond Failure of Deformed Reinforcing Bars Based on the Longitudinal Splitting Effect of the Bars", ACI Journal, Proceedings V. 76, No. 1, Jan. 1979, pp. 5-17.

34. Ferguson, P.M., "Small Bar Spacing or Cover - A Bond Problem for the Designer", ACI Journal, Proceedings V. 74, No. 9, Sept. 1977, pp. 435-439.

35. Soretz, S. and Hölzenbein, H., "Influence of Rib Dimensions of Reinforcing Bars on Bond and Bendability", ACI Journal, Proceedings V. 76, No. 1, Jan. 1979, p. 111-125.

36. Kemp, E.L., "Bond in Reinforced Concrete: Behavior and Design Criteria", ACI Journal, Proceedings V. 83, No. 1, Jan.-Feb. 1986, pp. 50-57.

37. Kemp, E.L. and Wilhelm, W.J., "Investigation of the Parameters Influencing Bond Cracking", ACI Journal, Proceedings V.76, No. 1, Jan. 1979, pp. 47-71.

38. Johnston, D.W. and Zia, P., "Analysis of Dowel Action", Proceedings ASCE, V. 97, No. ST5, May 1971, pp. 1611-1630.

39. Acharya, D.N. and Kemp, K.O., "Significance of Dowel Forces on the Shear Failure of Rectangular Reinforced Concrete Beams without Web Reinforcement", ACI Journal, Proceedings V. 62, No. 10, Oct. 1965, pp. 1265-1278.

40. Jiminez, R., White, R.N. and Gergely, P., "Bond and Dowel Capacities of Reinforced Concrete", ACI Journal, Proceedings V. 76, No. 1, Jan. 1979, pp. 73-91.

41. Untrauer, R. and Henry, R., "Influence of Normal Pressure on Bond Strength", ACI Journal Proceedings, V. 62, No. 5, May 1965, pp. 577-586.

42. Mains, R.M., "Measurement of the Distribution of Tensile and Bond Stresses along Reinforcing Bars", ACI Journal, Proceedings V. 48, No. 10, Nov. 1951, pp. 225-252.

43. Popov, E.P., "Bond and Anchorage of Reinforcing Bars under Cyclic Loading", ACI Journal, Proceedings V. 81, No. 4, July-Aug. 1984, pp. 340-349.

44. Viawathanatepa, S., Popov, E.P. and Bertero, V.V., "Effects of Generalized Loadings on Bond of Reinforcing Bars Embedded in Confined Concrete Blocks", EERC 79/22, University of California, Berkeley, Aug. 1979, 304 p. (not viewed by author).

45. Eligehausen, R., Popov, E.P. and Bertero, V.V., "Local Bond Stress-Slip Relationships of Deformed Bars under Generalized Excitations", Proceedings, 7th European Conference on Earthquake Engineering, Athens, Sept. 1982, Session 4, pp. 69-80.

46. Ciampi, V., Eligehausen, R., Bertero, V. and Popov, E., "Analytical Model for Deformed Bar Bond under Generalized Excitations", IABSE Colloquium, Advanced Mechanics of Reinforced Concrete, Delft, 1981, V.-Band 34, pp. 53-67.

47. Ciampi, V., Eligehausen, R., Bertero, V., and Popov, E., "Analytical Model for Concrete Anchorages of Reinforcement Bars under Generalized Excitations", EERC 82/23, University of California, Berkeley, 1982.

48. Eligehausen, R., Bertero, V.V., and Popov, E.P., "Hysteretic Behavior of Reinforcing Deformed Hooked Bars in R/C Joints", Proceedings, 7th European Conference on Earthquake Engineering, Athens, Sept. 1982, Session 4, pp. 171-8.

49. Ciampi, V., Eligehausen, R., Bertero, V.V. and Popov, E.P., "Hysteretic Behavior of Deformed Reinforcing Bars under Seismic Excitations", Proceedings, 7th European Conference on Earthquake Engineering, Athens, Sept. 1982, Session 4, pp. 179-187.

50. Filippou, F.C., Popov, E.P. and Bertero, V.V., "Modeling of R/C Joints under Cyclic Excitations", Proceedings ASCE, V. 109, No. ST11, Nov. 1983, pp. 2666-2684.

51. Perry, E.S. and Jundi, N., "Pullout Bond Stress Distribution Under Static and Dynamic Repeated Loadings", ACI Journal, Proceedings V. 66, No. 5, May 1969, pp. 377-380.

52. Ismail, M.A.F. and Jirsa, J.O., "Behavior of Anchored Bars under Low Cycle Overloads Producing Inelastic Strains", ACI Journal, Proceedings V. 69, No. 7, July 1972, pp. 433-438.

53. Giuriani, E., "Experimental Investigation on the Bond-Slip Law of Deformed Bars in Concrete", IABSE Colloquium, Advanced Mechanics of Reinforced Concrete, Delft, 1981, Final Report, V.-Band 34, pp. 121-142.

54. Hawkins, N.M., Lin, I.J., and Jeang, F.L., "Local Bond Strength of Concrete for Cyclic Reversed Loading", Proceedings, International Conference on Bond in Concrete, Scotland, June 1982, published in "Bond in Concrete", P. Bartos, Ed., Applied Science Publishers, 1982, pp. 151-161.

55. Lin, I.J., "Anchorage Characteristics for Reinforcing Bars Subjected to Reversed Cyclic Loading", Ph.D. Thesis, University of Washington, March 1981 (not viewed by author.)

56. Cheema, T.S. and Klingner, R.E., "Tensile Anchorage Behavior of Deformed Reinforcement Anchored in Grouted Concrete Masonry", ACI Journal, Proceedings V. 82, No. 3, May-June 1985, pp. 372-380.

57. Cheema, T.S., "Anchorage Behavior and Prism Strength of Grouted Concrete Masonry", Ph.D. Thesis, University of Texas at Austin, May 1981, (not viewed by author.)

58. Cheema, T.S. and Klingner, R.E., "Failure Criteria for Deformed Reinforcement in Grouted Concrete Masonry", ACI Journal, Proceedings V. 82, No. 4, July-Aug. 1985, pp. 434-442.

59. Cheema, T.S. and Klingner, R.E., "Design Recommendations for Tensile Anchorages of Deformed Reinforcement in Grouted Concrete Masonry", ACI Journal, Proceedings V. 82, No. 5, Sept.-Oct. 1985, pp. 616-621.

60. Watanabe, M., "Bond and Anchorage of Reinforcement in Masonry", First Joint Technical Co-ordinating Committee on Masonry Research, U.S.-Japan Co-ordinated Earthquake Research Program, Tokyo, Japan, Aug. 1985, 10 p.

61. Kubota, T., "On the Lap Joint of Flexural Reinforcement in Concrete Masonry", First Joint Technical Co-ordinating Committee on Masonry Research, U.S.-Japan Co-ordinated Earthquake Research Program, Tokyo, Japan, Aug. 1985, 8 p.

62. Modena, C. and Cecchinato, P., "Bond of Plain and Deformed Bars in Block Masonry Mortar Joints", Proceedings, 4th Canadian Masonry Symposium, Fredericton, Canada, June 1986, V.2, pp. 800-814.

63. Manalastas, J.C. and MacLeod, I.A., "Bond Strength of Bed-Joint Reinforcement in Brickwork", International Symposium on Reinforced and Prestressed Masonry, Edinburgh, Scotland, Aug. 1984 (not viewed by author.)

64. Southcombe, C. and Dasht, A.S.L., "The Bond Strength of Ladder-Type Bed Joint Reinforcement", Proceedings, 3rd Canadian Masonry Symposium, Edmonton, Canada, June 1983 (not viewed by author.)

65. Tassios, T.P., Vintzeleou, E., and Trohanis, A., "Interaction Diagrams for Reinforced and Unenforced Masonry Walls", 3rd International Symposium on Wall Structures, CIB, Warsaw, June 1984 (not viewed by author).

66. Barlet and Zelger, "Test on Bond between Steel and Mortar", British Ceramic Abstract, Apr. 1983 (not viewed by author).

67. Suter, G.T. and Fenton, G.A., "Splice Length Tests on Reinforced Concrete Masonry Walls", Proceedings, 3rd North American Masonry Conference, Arlington, Texas, June 1985, Paper No. 68, 14 p.

68. Baynit, R.A., "Bond and Development Length in Reinforced Concrete Block Masonry", M. Eng. Thesis, Carleton University, Ottawa, Canada, 1980 (not viewed by author).

69. Suter, G.T. and Fenton, G.A., "Flexural Capacity of Reinforced Masonry Members", ACI Journal, Proceedings V. 83, No. 1, Jan.-Feb. 1986, pp. 127-136.