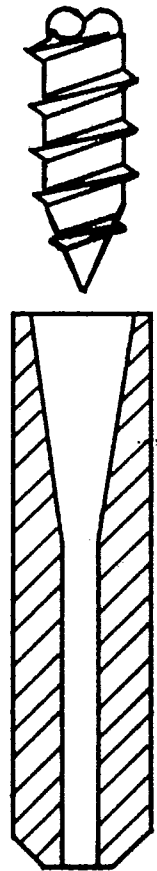
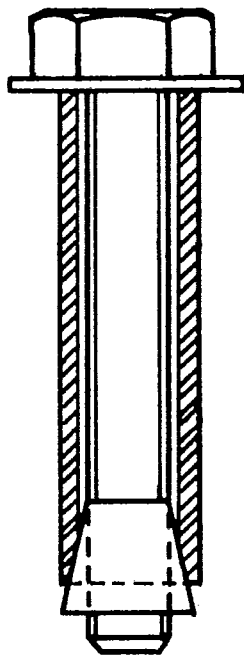
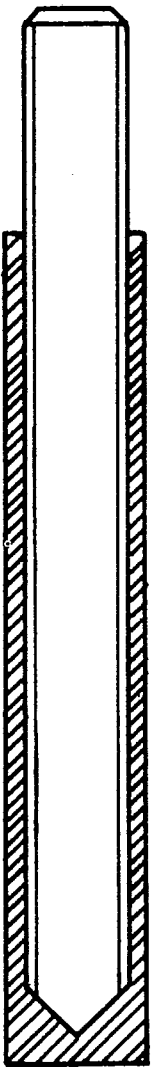


Anchorage to Concrete

Gerard B. Hasselwander
Editor



SP-103

American Concrete Institute, Detroit



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PREFACE

The fifteen papers in this volume are based on two symposia sponsored by ACI Committee 355, Anchorage to Concrete. The symposia were held during the American Concrete Institute Conventions in Atlanta (1982) and in Los Angeles (1983). This volume covers a range of topics on anchorage to concrete.

The publication of this volume was made possible by the work of many individuals. The efforts and patience of the authors is gratefully acknowledged. Each paper was reviewed according to ACI Publication Policy and the efforts of the reviewers are much appreciated. The committee's editorial group, Gerard B. Hasselwander, Robert M. Eshbach, Donald F. Meinheit, Teresa Perry, and Charles Poparad worked with the authors in the preparation of their manuscripts so that the comments of the reviewers were appropriately addressed.

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Dowels for Anchoring New Concrete Facing to Existing Lock Walls

by T. Liu and T. Holland

Synopsis: This paper presents the results of laboratory and field dowel pullout tests and shear transfer tests. These tests were conducted to evaluate the pullout performance of dowels and the influence of dowel spacing on the load-carrying capacity of the replacement concrete cast during rehabilitation of vertical walls in navigation locks. The results of these tests and a review of the existing literature on the interface shear transfer and the dowel action mechanisms were the bases for the development of rational design criteria for dowels for anchoring replacement concrete to vertical lock walls. The design criteria include surface preparation, minimum dowel size, dowel spacing, and anchorage requirements. An example of designing dowels for a typical lock wall rehabilitation project is included.

Keywords: anchors (fasteners); concrete construction; dowels; joints (junctions); load transfer; locks (waterways); pullout tests; renovating; shear tests; structural design

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ACI Member Terence C. Holland is a Research Civil Engineer in the Concrete Technology Division, Structures Laboratory, U. S. Army Engineer Waterways Experiment Station. Dr. Holland is primarily involved with methods and materials for repair and rehabilitation of concrete structures. He is a member of ACI Committee 304, Measuring, Mixing, Transporting, and Placing Concrete.

SIGNIFICANCE

The research described in this paper is believed to be significant because a rational method for designing the dowels used in the rehabilitation of lock walls was developed to replace the existing "rule of thumb" approach. The new method is simple to apply and is expected to lead to significant cost savings as it is used on future projects.

BACKGROUND

The U. S. Army Corps of Engineers operates and maintains over 200 concrete navigation locks in the navigable rivers of the United States. Of these lock structures, more than half are over 50 years old. With structures of such age, major repair or rehabilitation is often required to assure the safety and continuation of operation of the structures.

A significant portion of most navigation lock rehabilitation projects is work on the lock walls. Typical damage found on lock walls is shown in Figure 1. The deteriorated concrete seen in this figure will be removed, and the lock walls will be restored to original dimensions by the placement of new concrete. A typical construction drawing showing resurfacing details is given in Figure 2.

The first step in the repair technique is to remove the surface concrete to a depth of 12 to 24 in. (300 to 600 mm) by means such as drilling and blasting. The next step is to install dowels in the walls. The dowels serve to position vertical and horizontal reinforcing steel in the replacement concrete and to anchor the replacement concrete to the existing wall elements. The dowels are usually reinforcing bars with a 90-degree bend. They may be anchored into the old concrete by the use of hydraulic-cement grout, epoxy resin, or polyester resin with a 15- to 18-in. (380- to

450-mm) embedment being typical. Figure 3 shows a section of wall in which the installation of dowels has been completed.

Once the dowels are in place, reinforcing steel is placed, exterior forms are positioned, and concrete is placed to restore the walls to original dimensions.

The steps most open to question in the rehabilitation process have been the design and installation of the dowels. No engineering data were found upon which to base dowel size and spacing. As a result, a large number of dowels on close centers (usually 2 ft (600 mm) center-to-center) have been specified. Since installation of the dowels is very labor intensive, it turns out to be a very costly segment of the project.

Some of those who have observed current practice questioned whether the typical size and spacing of dowels are too conservative. A reduction in the number of dowels required for a project could lead to significant savings. The study described in this paper was undertaken at the U. S. Army Engineer Waterways Experiment Station (WES) to resolve this question. This study consisted of five parts, and the results have been reported (1). The parts were:

- a. Laboratory pullout tests of dowels anchored with polyester-resin cartridges.
- b. Field pullout tests of contractor-installed dowels (the same type of polyester-resin cartridges was used).
- c. Literature survey of load-transfer mechanism across the concrete joints.
- d. Laboratory tests of load-carrying capacity across such joints that contain dowels representing various percentages of steel.
- e. Development of a design method and design guidance for dowels anchoring replacement concrete to vertical lock walls.

LABORATORY PULLOUT TESTS

Laboratory pullout tests were conducted to evaluate the effects of embedment lengths on the pullout resistance of No. 6 (19.0-mm) reinforcing bars. A large concrete block was used as a test bed. Holes for the embedment of reinforcing bars were drilled into the test block. Commercially available polyester-resin cartridges were used as the bonding agent. A total of eight pullout tests were conducted in the laboratory.

The test block in which the reinforcing bars were embedded was cast in 1972 as part of a mass concrete slipform construction program conducted at the WES (2). The block measured 3 by 6 by

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10 ft (0.9 by 1.8 by 3.0 m) and contained 6-in. (150-mm) nominal maximum size limestone aggregate. Five 6-in.- (150-mm) diameter cores were drilled from the test block using a diamond bit. These cores were taken from the center and the four corners of the test block to obtain representative samples. Five 6- by 12-in. (152- by 305-mm) specimens were prepared from the cores and were tested for compressive strength and tensile splitting strength. The average compressive strength and the tensile splitting strength were 2510 and 440 psi (17.3 and 3.0 MPa), respectively.

Eight 1-1/8-in.- (29-mm) diameter holes were drilled using a pneumatic rotary-percussive drill. These holes were drilled deep enough to embed No. 6 (19.0-mm) reinforcing bars at embedment lengths equal to 6.0, 7.5, 11.25, and 15.0 in. (150, 190, 285, and 380 mm) (embedment length to nominal bar diameter ratios, $L/D = 8, 10, 15,$ and 20 , respectively). To simulate the field conditions, all holes had a 10-degree inclination from the horizontal. The slight inclination was used to prevent loss of the bonding agent. These drilled holes were spaced to allow for a possible 45-degree conical failure of the concrete.

Standard No. 6 (19.0-mm) deformed reinforcing bars were used as dowels. The polyester-resin cartridges (1-in. (25-mm) diameter by 12 in. (600 mm) long) used as the bonding agent consisted of a pack containing a polyester-resin component with a catalyst. The components are isolated from each other by a physical-chemical barrier that prevents reaction between the components until required.

The procedures for embedding the reinforcing bars in the drilled holes were as follows:

- a. Air flush the drilled holes to remove all debris and dust.
- b. Insert a polyester-resin cartridge.
- c. Force the reinforcing bar into the hole breaking the cartridge.
- d. Couple a pneumatic drill to the reinforcing bar and rotate the bar into the hole at 200 to 450 rpm.
- e. Stop inward movement when the reinforcing bar reached the desired embedment length, and continue rotating the bar for 15 to 20 sec to mix the resin system thoroughly.
- f. Stop rotation and uncouple the drill from the reinforcing bar. (The bar was firmly bonded when the resin set in a few minutes.)

The test apparatus used to conduct the pullout tests is shown in Figure 4. The reaction frame was constructed with 9-in. (230-mm) channel sections. The clear span of the reaction frame

was 2 ft 2 in. (660 mm). A 60-ton (530-kN), hollow plunger jack in combination with an electric pump was used to apply the axial load to the reinforcing bar. To account for the 10-degree inclination of the reinforcing bar, a special steel shim plate was inserted between the jack and reaction frame. The jacking system was calibrated, and as the load was applied to the bar, a 10,000-psi (70-MPa) pressure gage was monitored. From the pressure gage readings, the jack load was determined for any increment of pressure.

After the polyester resin had cured for approximately 30 minutes, the bar was tested for pullout resistance. A particular test was terminated when no additional load could be applied to the reinforcing bar because of either slippage or excessive elongation.

The results of the laboratory pullout tests are presented in Table 1. All reinforcing bars yielded when the embedment length was greater than 11.25 in. (285 mm) ($L/D = 15$). The yielding was evident when the rust on the bar began flaking off and when the bar would not support additional load. The yielding was also evident when the axial load reached 20.8 kips (93 kN), which was the yield strength of the bars.

Slippage along the concrete-resin interface generally caused failure when the embedment length was less than 11.25 in. (285 mm). A typical bar after pullout is shown in Figure 5. No concrete failures were observed for the embedment lengths investigated.

FIELD PULLOUT TESTS

Field pullout tests were conducted to determine the pullout performance of dowels installed under field conditions by a contractor working on a major rehabilitation project. At the site selected,* the dowels were being installed by the use of the polyester-resin cartridge system and the basic procedures that had been used for the laboratory pullout tests.

The original test plan envisioned the use of the laboratory pullout test apparatus. This approach was determined to have two problems:

- a. Use of the laboratory pullout test apparatus would require installation of special dowels long enough to be gripped by the jack; day-to-day installations could not be tested. This caused concern that the bars to be tested might receive special attention during placement and would not be representative.
- b. Site inspection indicated that the test apparatus probably could not be supported in the proper position for testing. The top of the lock walls did not

* Locks and Dam No. 3, Monongahela River, near Pittsburgh, Pennsylvania.

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provide enough room for a crane to operate. All of the test equipment would therefore have to be capable of being placed in and operated from a small boat.

The test plan was revised to allow a random selection of contractor-placed dowels for testing. This modification was achieved by the following revisions to the test equipment (the revised test apparatus is shown in Figure 6):

- a. The reaction frame was eliminated, which allowed the reaction force to be transmitted directly to the wall surrounding the dowel. This was determined to be a reasonable approach since none of the bars tested in the laboratory had shown a conical type of concrete failure.
- b. The hollow core jack was replaced by two flat jacks mounted on a steel plate. This significantly reduced the length of bar required for testing.
- c. The inclined plate and the prestressing chuck used in the laboratory tests were retained for the field tests.
- d. A manually operated hydraulic pump was used to power the two flat jacks. The jacks, pump, and gage were calibrated using a laboratory test machine.

The procedure used to perform the pullout tests at the site was as follows:

- a. Random dowels were selected from those inside the river chamber and those on the riverside of the middle wall downstream of the river chamber.
- b. The bends in the ends of the dowels were straightened using a come-along and a cheater pipe technique. This was necessary to provide enough dowel to grip even with the flat jack apparatus.
- c. Surface irregularities in the vicinity of the dowel to be tested were reduced by hand to provide a bearing surface as flat as possible.
- d. The pullout apparatus was mounted on the bar. Loading was done in gage pressure increments of 500 psi (3.4 MPa). Each increment was held for approximately 30 sec before the load level was increased.
- e. Bars were normally loaded twice in each test. The first loading would exhaust jack travel at a relatively low load because of localized crushing of protruding concrete beneath the inclined wedge. The

initial load was then released, and a new grip was obtained on the bar. Loading was then again accomplished in increments as described above.

- f. Measurements of actual dowel displacement were not made. The dowels were loaded until a given load could no longer be held, indicated by loss of pressure on the gage, or until the approximate yield range of the steel was reached.

Field pullout test data as shown in Table 2 indicated that none of the bars failed to hold at least the design stress of 40,000 psi (275 MPa).

LITERATURE SURVEY

An understanding of the shear-transfer mechanism across an interface between new and old concrete is of critical importance in dowel design. The shear-transfer mechanism, which acts by combined action of aggregate interlock and friction and dowel action, has been studied by many investigators (3-13), mainly through tests simulating connection and construction joints in precast and cast-in-place concrete. The existing literature was reviewed in this study to evaluate the performance of the combined action of the interface shear-transfer and the dowel-action mechanisms.

Based on the literature review the shear-transfer mechanism across an interface between new and old concrete may be hypothesized as follows.

- a. The shear forces initially are transferred through the uncracked interface by bond. When a crack forms at the interface, the shear forces are transferred by the combined action of aggregate interlock and friction and by dowel action.
- b. When shear acts along a crack, slip of one crack face occurs with respect to the other. If the crack faces are rough and irregular, this slip is accompanied by separation of the crack faces. This separation will stress the dowel crossing the crack, which in turn will provide a clamping force across the crack face. The applied shear is then resisted by friction between the crack faces and by dowel action of the reinforcement crossing the crack.

The shear-friction theory proposed by Birkeland and Birkeland (3) provides a lower bound to the experimental data available on shear test specimens and was selected as the basis for the dowel design technique. The shear-friction method of calculation assumes that all shear resistance is due to friction between the crack faces. Therefore, a reasonable value of the coefficient of

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friction for the shear-friction equation must be developed so that the calculated shear strength will be in reasonably close agreement with test results.

LABORATORY SHEAR TRANSFER TESTS

The objectives of the laboratory shear transfer tests were to develop the value of the coefficient of friction to be used in the shear-friction equation and to examine the influence of dowel spacing on the load-carrying capacity of the replacement concrete in a lock wall renovation. These tests were accomplished using laboratory-cast blocks to represent the old and new concretes. Dowels of various diameters were used with the blocks to represent the spacing variations in prototype structures.

The same concrete mixture was used for both old and new portions of the test blocks. The nominal 4500-psi (31-MPa) mixture contained 3/4-in. (19.0-mm) nominal maximum size aggregate. The test blocks were designed to provide a contact area of 576 sq in. (0.4 m²) between the old and new concretes. The dimensions of the old and new portions of each block were identical. Each portion (old or new) contained approximately 5.7 cu ft (0.2 m³) of concrete and weighed approximately 850 lb (385 kg).

The old portion of the blocks was cast in a position so that the contact surface between the old and new concretes would be available for finishing. After the concrete had stopped bleeding, a retarder was applied to the contact surface. The forms were removed approximately 24 hr after the blocks were cast, and the contact surface was cut with a low-pressure water jet to remove paste and to provide a roughened surface. The blocks were then stored at 100 percent relative humidity for a minimum of 28 days.

The old concrete blocks were removed from the fog room after at least 28 days. Forms and reinforcing steel were then placed for the casting of the new concrete. A compressible material was used to cast a void between the blocks to allow for movement during testing. After the forms were removed the block assemblies were again stored for 28 days at 100 percent relative humidity.

The compressible material between the blocks was removed prior to the testing. Testing was accomplished using a 440,000-lb (1.96-MN) testing machine that applied loading at a rate of 25,000 lb/min (110 kN/m). Figure 7 shows a block in place in the testing machine. The blocks were loaded to failure (i.e., separation of the new concrete from the old concrete).

Data from the shear-transfer tests are presented in Table 3. The specimens that did not contain dowels showed an average shear strength of 196 psi (1.35 MPa). The average ultimate shear strengths were 203, 234, 242, and 233 psi (1.40, 1.61, 1.67, 1.60 MPa), respectively, for specimens containing No. 3, 4, 5, and 6 (9.5-, 12.7-, 15.9-, and 19.0-mm) dowels.

The effect of the dowels within the range of the percentage of steel tested did not appear to increase significantly the load-carrying capacity of the blocks. Instead, the bond of the new to the old concrete appeared to have been much more significant than the amount of steel present.

Figure 8 plots the average ultimate shear stresses against the values of ρf_y .^{*} There appears to be a slight upward trend in the data that shows a small but increasing contribution from the dowels as dowel size increases.

Based on the test data, the relationship between average ultimate shear stress, V' , and the value of ρf_y was derived using the least-squares fit technique:

$$V' = 200 + 1.35 \rho f_y \quad (f_y \text{ in psi}) \quad (1)$$

$$V' = 1.38 + 1.35 \rho f_y \quad (f_y \text{ in MPa}) \quad (1a)$$

The value of the coefficient of friction obtained from the test data, 1.35, is consistent with values reported in the literature.

The specimens instrumented during loading experienced essentially no differential movement between the old and new concretes prior to failure with failure defined as the maximum load the block would carry. Two distinct modes of behavior for the replacement concrete were noted at failure. In the specimens without dowels, failure resulted when the top block (new concrete) dropped completely down onto the bottom block as a result of a brittle fracture. In the specimens with dowels (regardless of dowel size), the failure was more ductile; the dowel was able to carry the dead load of the new concrete, thus preventing the new concrete from dropping completely onto the old concrete. This could be of importance in a prototype structure.

Examination of the failure surface of several of the specimens showed that failure did occur on the plane between the old and new concretes. These surfaces showed bond failures with some plucking of aggregate particles from the old and new concretes. A very small percentage of aggregate particles was broken.

DERIVATION OF DESIGN EQUATION

Laboratory test results indicate that well-bonded concrete is relatively strong in shear transfer; however, there is always the possibility that a crack will form at the interface because of

* ρ = reinforcement ratio

$$= \frac{\text{Area of dowel reinforcement}}{\text{Area of shear plane}}$$

f_y = yield strength of the dowel

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shrinkage, thermal stresses, or other reasons. Therefore, in the design of dowels, a crack should be assumed to be present along the interface, with relative displacement along the assumed crack resisted by friction maintained by dowels across the assumed crack.

According to the American Concrete Institute (ACI) Building Code (14), the design of cross sections subject to shear should be based on

$$V_u \leq \phi V_n \quad (2)$$

where

V_u = factored shear force at section considered, lb (N)

ϕ = strength reduction factor = 0.85 for shear

V_n = nominal shear strength, lb (N)

Based on the shear-friction concept,

$$V_n = A_d f_y \mu \quad (3)$$

where

A_d = cross-sectional area of dowel, sq in. (m^2)

f_y = specified yield strength of dowel, psi (Pa)

μ = coefficient of friction

Substituting Equation 3 into Equation 2 and solving for A_d

$$A_d = \frac{V_u}{\phi f_y \mu} \quad (4)$$

The shear-friction concept is valid for conditions in which failure is attained by yielding of the reinforcement crossing the crack. Thus, dowels must be anchored in both sides of the concrete by embedment or hooks to develop yield in the steel.

For the polyester-resin cartridges tested in the laboratory, an embedment length of not less than 15 times the nominal diameter of the dowel was found to be satisfactory. For cement grouts, Stowe (15) also recommended an embedment length of 15 times the nominal dowel diameter. For the epoxies he tested, Stowe recommended a somewhat shorter embedment length of 10 times the nominal bar diameter. Overall, for portland-cement, epoxy-resin, or polyester-resin grouts, an embedment length of at least 15 times the nominal diameter of the dowel should be satisfactory for concretes with a compressive strength of 3000 psi (21 MPa) or greater.

Tests of laboratory specimens indicated the average coefficient of friction between old and new concretes was 1.35. To account for the variations expected in the field construction, a more conservative value of 1.00 should be used. This reduced value of 1.00 is also in agreement with ACI 318-77 (14) recommendations for concrete placed against hardened concrete. To ensure that the coefficient of friction of 1.00 is attainable, all unsound, damaged, dirty, porous, or otherwise undesirable old concrete should be removed, and the old concrete surface should be clean, free of laitance, and intentionally roughened to a relief of at least 1/4 in. (6 mm).



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The factored shear force V_u for lock walls at the interface should be based on the following (16):

$$V_u = 1.5V_D + 1.9V_L \quad (5)$$

where

V_D = shear force due to dead load, lb (N)

V_L = shear force due to all live loads, lb (N)

The vertical shear force at the interface in a typical lock wall is due primarily to the mass of the new concrete, and the vertical shear force due to live loads is usually small.

If the dowels are equally spaced at distance S both vertically and horizontally, Equation 5 can be rewritten as

$$V_u = (1.5Wt + 1.9v_L)S^2 \quad (6)$$

where

V_u = the factored shear force to be resisted by one dowel, lb (N)

W = unit weight of concrete, lb/cu ft (kg/m^3)

t = thickness of the replacement concrete, ft (m)

v_L = unit vertical shear stress due to all live loads, lb/ft² (Pa)

S = dowel spacing, ft (m)

Substituting Equation 6 into Equation 4 and rearranging,

$$S = \sqrt{\frac{A_d \phi f_y \mu}{1.5Wt + 1.9v_L}} \quad (7)$$

If the vertical shear force due to live loads is negligible, Equation 7 can be reduced to

$$S = \sqrt{\frac{A_d \phi f_y \mu}{1.5Wt}} \quad (8)$$

Substituting $\phi = 0.85$ and $\mu = 1.00$ into Equation 11 can provide a general equation for designing dowel spacing

$$S = \sqrt{\frac{0.85 A_d f_y}{1.5Wt + 1.9v_L}} \quad (9)$$

In most cases, the dowel spacing equation (Equation 9) will lead to spacings larger than those currently being used in practice, particularly if live loads are small. It is believed that a practical upper limit on dowel spacing should be imposed based upon considerations of the possible failure mode and of constructibility. The objective of the failure consideration is to eliminate the possibility of brittle failure, as was seen in the laboratory shear test specimens that contained no dowels. The smallest percentage of steel evaluated in the laboratory shear tests was 0.019 percent (equivalent to No. 6 (19.0-mm) dowels at a 48-in. (1.2-m) spacing). These specimens exhibited ductile failure.

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Specimens containing a smaller percentage of steel than 0.019 percent were not evaluated. From a constructibility standpoint, an upper limit seems appropriate to ensure proper positioning and support for the horizontal and vertical reinforcement in the replacement concrete. Based on these considerations (and minimum recommended bar size as discussed below), a 48-in. (1.2-m) maximum spacing is recommended. This admittedly arbitrary recommendation is conservative; however, a significant reduction in the required number of dowels can be realized.

It can be shown that relatively small bars can be used to satisfy the requirements of the design equation (Equation 9), again assuming no live loads. As with dowel spacings, it is believed that there is a practical lower limit on bar size which must be imposed. The theory in this case is based on considerations of damage after bars are installed but before concrete is placed and on live loads possibly imposed on the dowels by construction workers during the setting or tying of outside steel. An additional consideration is the relatively low cost of the steel compared with the cost of labor involved in dowel placement. Based on these considerations, a minimum dowel size of a No. 6 (19.0-mm) bar is recommended.

If tensile stresses are present across the interface where cracks are assumed, reinforcement for the tension must be provided in addition to that provided for shear friction. However, the tensile stresses due to temperature and shrinkage need not be considered because these stresses are self-limiting in a cracked interface.

DESIGN EXAMPLE

In a navigation lock wall rehabilitation project, 12 in. (300 mm) of deteriorated surface concrete will be removed and new replacement concrete will be placed to return the wall to original dimension. The following design information is given:

- a. Thickness of the replacement concrete = 1.0 ft (300 mm).
- b. Specified concrete compressive strength = 3000 psi (21 MPa).
- c. Dowel size = No. 6 (19.0-mm) deformed bars.
- d. Design yield strength for dowels = 40,000 psi (275 MPa).
- e. Vertical shear stresses due to all live loads are negligible.
- f. Concrete cover = 4 in. (100 mm).

- g. Dowels will be anchored into the old concrete with a commercially available polyester-resin cartridge system.
- h. Existing concrete compressive strength = 3000 psi (21 MPa).
- i. Unit weight of concrete = 150 lb/ft³ (2400 kg/m³).

The dowel spacing can be determined from Equation 9:

$$S = \sqrt{\frac{0.85 A_d f_y}{1.5 W t + 1.9 v_L}} \quad (9)$$

where

$$\begin{aligned} A_d &= 0.44 \\ f_y &= 40,000 \\ W &= 150 \\ t &= 1.0 \\ v_L &= 0 \end{aligned}$$

Thus,

$$\begin{aligned} S &= \sqrt{\frac{0.85 \times 0.44 \times 40,000}{1.5 \times 150 \times 1.0}} \\ &= 8.15 \text{ ft} > 4 \text{ ft} \quad (2.5 \text{ m} > 1.2 \text{ m}) \end{aligned}$$

The maximum dowel spacing of 4 ft (1.2 m) should be used.

In accordance with ACI 318-77 (14), the development length ℓ_d for deformed bars in tension should be computed by

$$\ell_d = 0.04 A_b f_y / \sqrt{f'_c} \quad (10)$$

but not less than

$$0.0004 d_b f_y \quad (11)$$

where

$$\begin{aligned} A_b &= \text{area of individual bar, sq in. (m}^2\text{)} \\ f'_c &= \text{specified compressive strength of concrete, psi (Pa)} \\ d_b &= \text{nominal diameter of bar, in. (m)} \end{aligned}$$

Solving Equation 10 gives

$$\begin{aligned} \ell_d &= 0.04 \times 0.44 \times 40,000 / \sqrt{3000} \\ &= 12.85 \text{ in. (326 mm)} \end{aligned}$$

whereas Equation 11 gives

$$\begin{aligned} \ell_d &= 0.0004 \times 0.75 \times 40,000 \\ &= 12.0 \text{ in. (305 mm)} \end{aligned}$$

Thus, the required development length is 12.85 in. (326 mm), which is greater than the thickness of the replacement concrete minus

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concrete cover; therefore, hooked dowels should be used. The equivalent embedment length ℓ_e of a standard hook may be computed in accordance with ACI 318-77 (14):

$$\ell_e = 0.04 A_b f_h / \sqrt{f'_c} \quad (12)$$

where

$$f_h = 360 \sqrt{f'_c}$$

Thus,

$$\begin{aligned} \ell_e &= 0.04 \times 0.44 \times 360 \sqrt{3000} / \sqrt{3000} \\ &= 6.33 \text{ in. (161 mm)} \end{aligned}$$

The combined development length = $(12 - 4) + 6.33 = 14.33 \text{ in.} > 12.85 \text{ in. (364 mm} > 326 \text{ mm)}$. Therefore, the standard hook may be used.

Assuming that the polyester-resin cartridges in this example are the same type as those tested in the laboratory, an embedment length of not less than 15 times the nominal diameter of the dowel will be satisfactory. Therefore, an embedment length of 11.25 in. (285 mm) is required.

CONCLUSIONS

Based on the results of this study, the following conclusions may be drawn:

- (1) The results of laboratory pullout tests indicated that for embedment lengths less than 15D, where D is the nominal bar diameter, the failure was generally caused by slippage along the concrete-resin interface. All reinforcing bars yielded when the embedment length was greater than 15D.
- (2) The polyester-resin cartridges evaluated were an effective bonding agent for anchoring dowels in concrete, provided that adequate embedment lengths have been obtained and that the cartridge manufacturer's installation recommendations have been followed.
- (3) The performance of dowels installed under field conditions using the polyester-resin cartridges was acceptable. No modifications to the general installation procedure seem appropriate.
- (4) Within the range of the percentage of steel used in the laboratory shear transfer tests, the effect of the dowels did not appear to increase significantly the load-carrying capacity of the test specimens. The bond of the new to the old concrete appeared to have been much more significant than the amount of steel present.
- (5) A value of 1.35 for the coefficient of friction between new and old concretes was developed from the laboratory shear test data. This value is consistent with values reported in the literature. A more conservative value of 1.00 was included in the recommended design equation.

(6) Two distinct failure modes were observed in the laboratory shear tests. In the specimens without dowels, the failure mode was brittle with a sudden dropping of the top block (new concrete) onto the bottom block. In the specimens with dowels (regardless of the percentage of steel in the range tested), the failure was ductile, with the dowel being able to carry the dead load of the concrete, thus preventing the complete separation of the two elements. The minimum reinforcement ratio tested (equivalent to No. 6 (19.0-mm) dowels at 48-in. (1.2-m) spacing) is apparently adequate to force the ductile failure mode to prevail.

(7) The dowel design criteria presented are based on the shear-friction concept and are rational and simple to use. These criteria are a significant improvement over current design techniques.

RECOMMENDATIONS

(1) Every effort should be taken to ensure adequate surface preparation of the old concrete in order to develop the shear-friction mechanism upon which the design criteria are based. These efforts will also help to ensure good bond between the old and new concretes, which will extend the life of the rehabilitation.

(2) Extra care should be taken in the field to avoid or minimize impact damage or bending of dowels by work barges or other causes prior to the placement of concrete.

(3) Although the dowel design criteria are based on the assumption of no bond, the importance of a good bond between the old and new concretes must not be overlooked.

(4) The long-term bond behavior of the replacement concrete under extended weathering and under freezing and thawing conditions should be investigated.

ACKNOWLEDGMENT

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Table 1

Laboratory Pullout Test Results

Specimen No.	Embedment Length		L/D	Pullout Force		Type of Failure
	in.	mm		lb	kN	
1	6.0	152	8	16,680	74	Slippage
2	6.0	152	8	13,900	62	Slippage
3	7.5	190	10	16,680	74	Slippage
4	7.5	190	10	19,460	87	Slippage
5	11.25	285	15	19,460	87	Slippage
6	11.25	285	15	22,240	99	Yielding
7	15.0	380	20	22,240	99	Yielding
8	15.0	380	20	22,240	99	Yielding

Table 2

Results of Field Pullout Tests

Dowel No.	Maximum Gage Pressure		Maximum Stress in Dowel	
	psi	MPa	psi	MPa
1	5000	35	49,300	340
2	4500	31	44,500	310
3	4500	31	44,500	310
4	4500	31	44,500	310
5	5000	35	49,300	340
6	6000	41	58,300	400
7	4500	31	44,500	310
8	6500	45	62,800	430
9	6500	45	62,800	430
10	6000	41	58,300	400
11	5500	38	53,700	370
12	6500	45	62,800	430

Table 3
Summary of Shear Test Data

	Dowel Size					
	None	No. 3	No. 4	No. 5	No. 6	
Dowel area, in. ² mm ²	0.00	0.11	0.20	0.31	0.44	
	0.00	71	129	200	284	
Percentage of steel	0.000	0.019	0.035	0.054	0.076	
ρ_f (w/f _y = 40,000 psi)	0.00	7.64	13.89	21.53	30.56	
ρ_f (w/f _y = 276 MPa)	0.00	0.0527	0.0958	0.1485	0.2108	
Old concrete						
Compressive strength,* psi	5550	6140	6470	5080	5420	
MPa	38.3	42.3	44.6	35.0	37.4	
Flexural strength,* psi	840	825	925	805	830	
MPa	5.8	5.7	6.4	5.6	5.7	
New concrete						
Compressive strength,* psi	4560	5120	5480	5110	4850	
MPa	31.4	35.3	37.8	35.2	33.4	
Flexural strength,* psi	815	755	760	800	715	
MPa	5.6	5.2	5.2	5.5	4.9	
Average shear strength, psi	196	203	234	242	233	
MPa	1.35	1.40	1.61	1.67	1.60	
Number of specimens in averages	2	2	2	2	3	

* Weighted averages based upon number of test specimens from each batch of concrete.

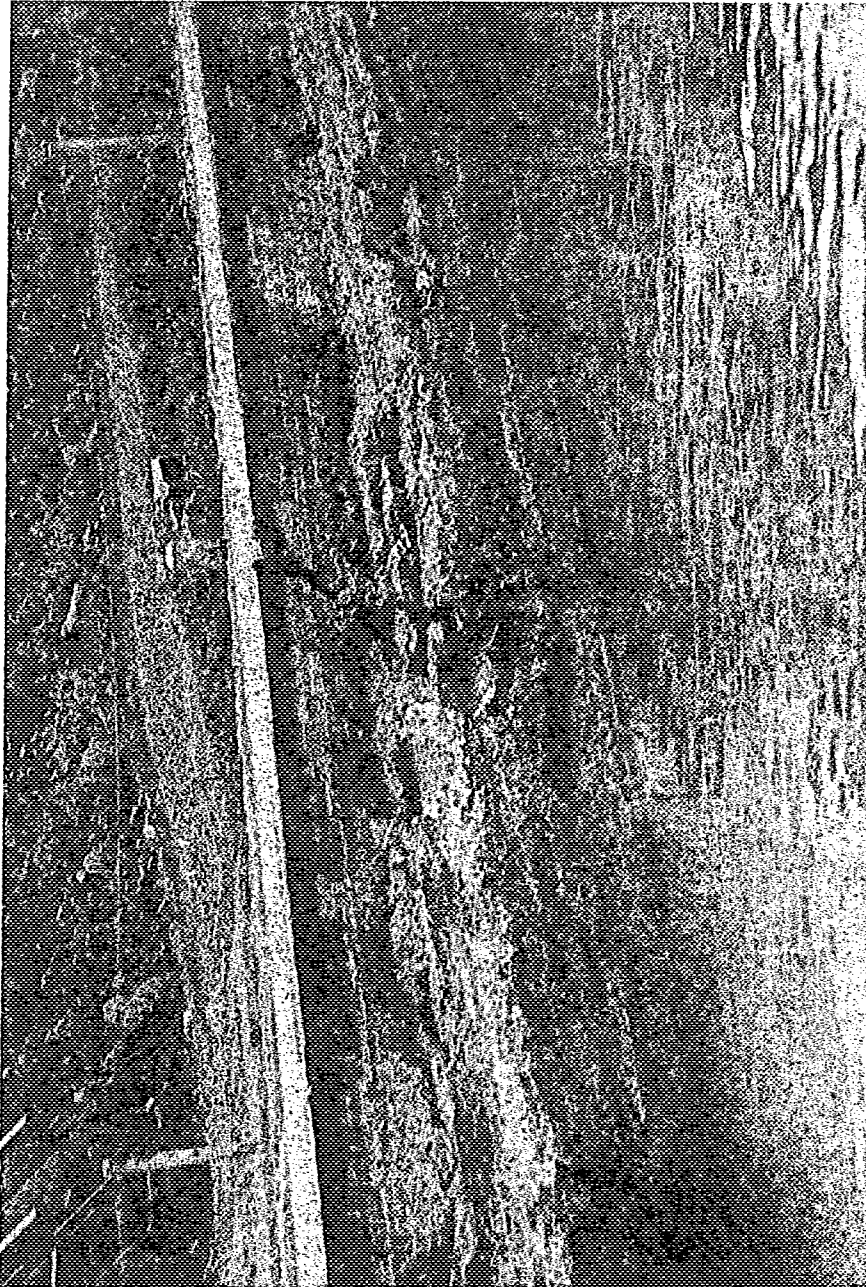


Fig. 1--Typical deteriorated concrete in a navigation lock

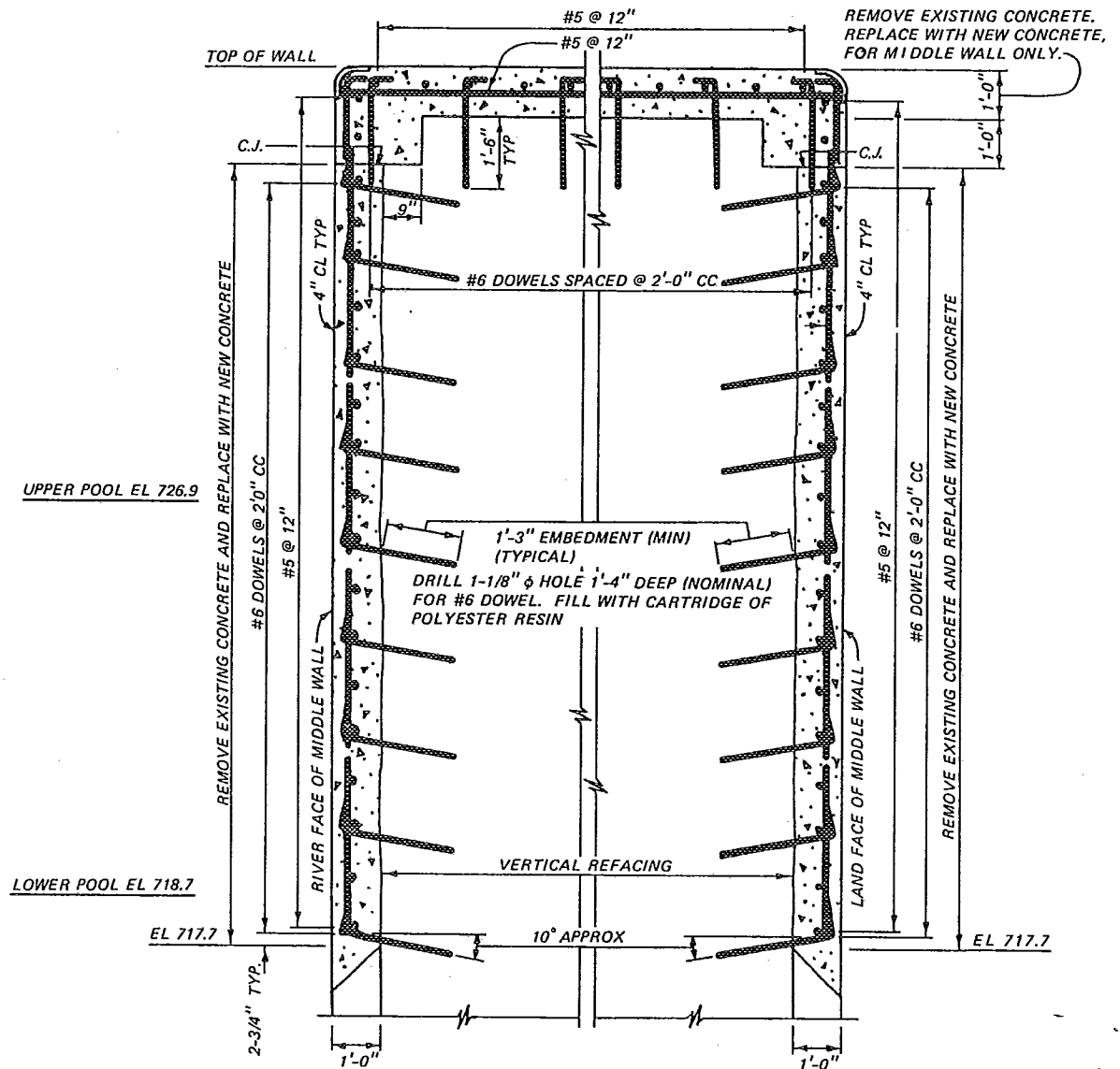


Fig. 2--Renovation details, typical project (US customary units)

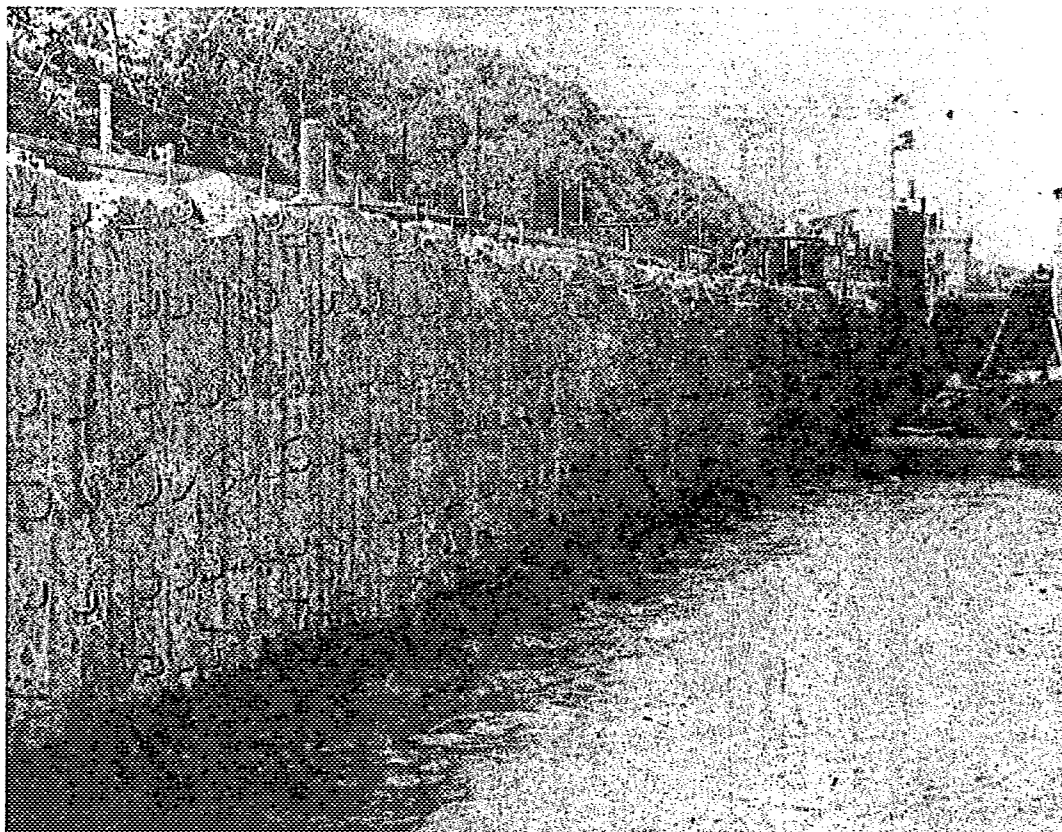


Fig. 3--Section of lock wall with dowels installed, spaced 2 ft. (600 mm) on center in both directions (straight bars are form ties)

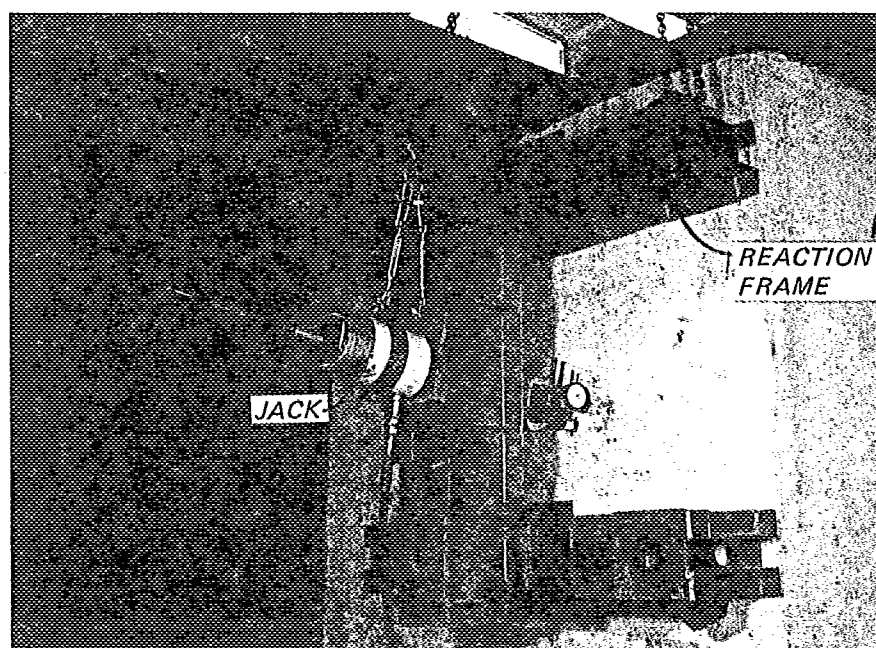


Fig. 4--Test apparatus, laboratory pullout tests

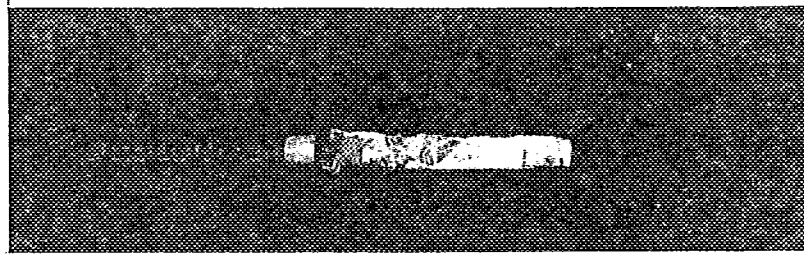


Fig. 5---Typical bar after pullout

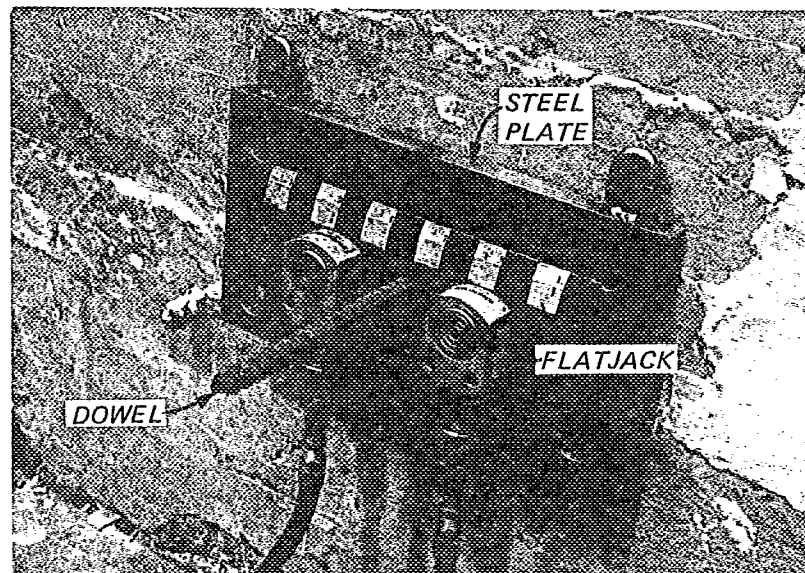


Fig. 6a---Test apparatus, field pullout tests -- Reverse side showing flat jacks

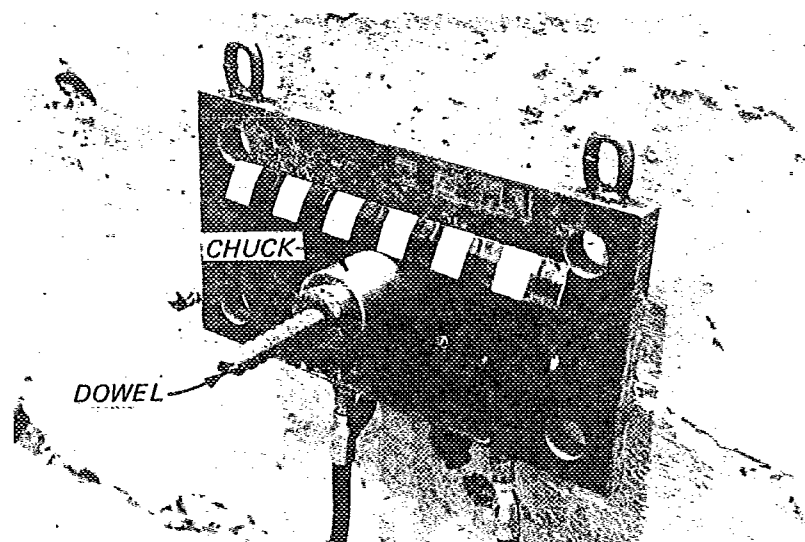


Fig. 6b---Test apparatus, field pullout tests -- Assembled testing apparatus with prestressing chuck gripping dowel (Note: jacks are in operating position on back side of plate)

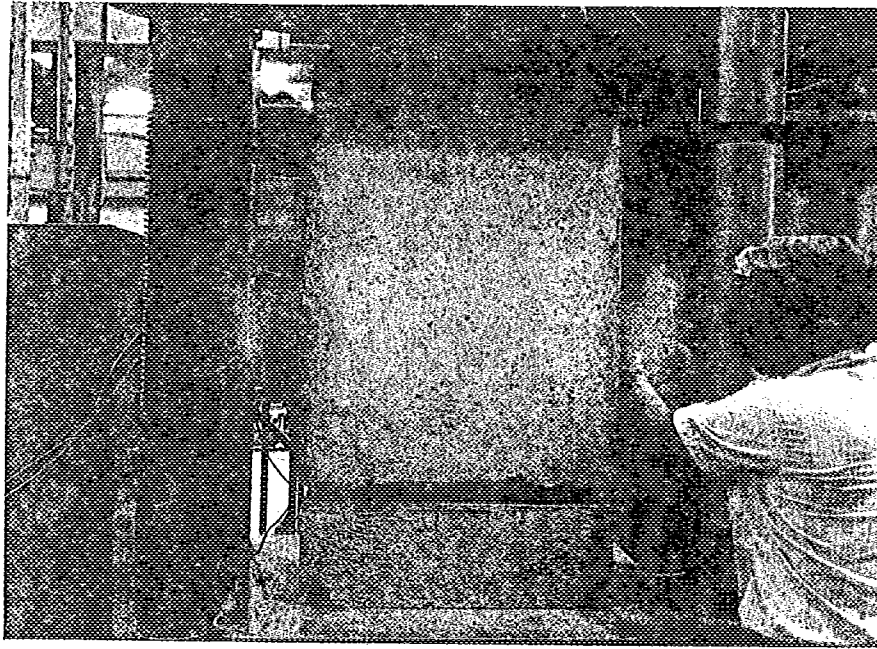


Fig. 7a--Test setup -- Front view of shear block in testing machine prior to loading

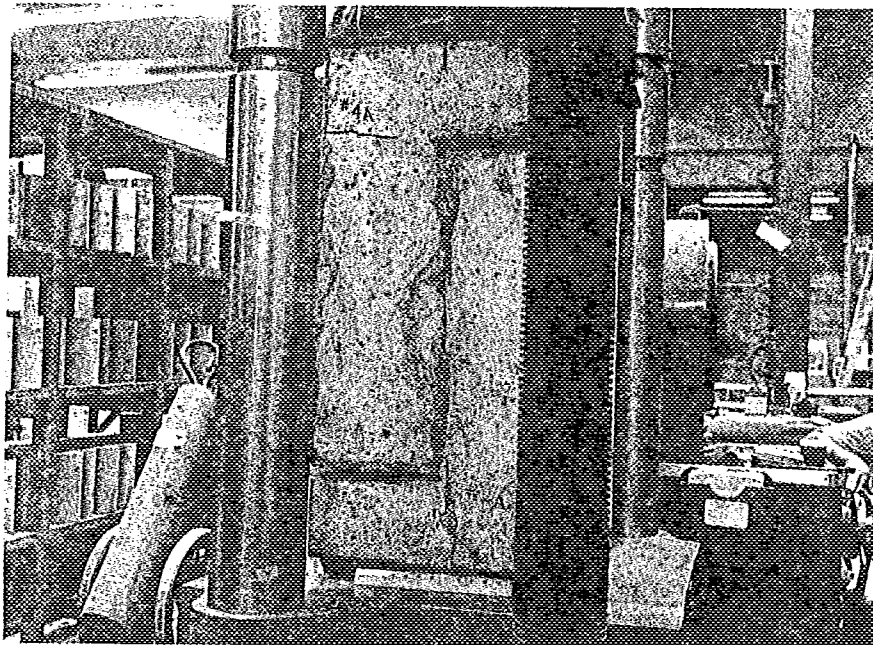


Fig. 7b--Test setup--Side view of shear block in testing machine during loading

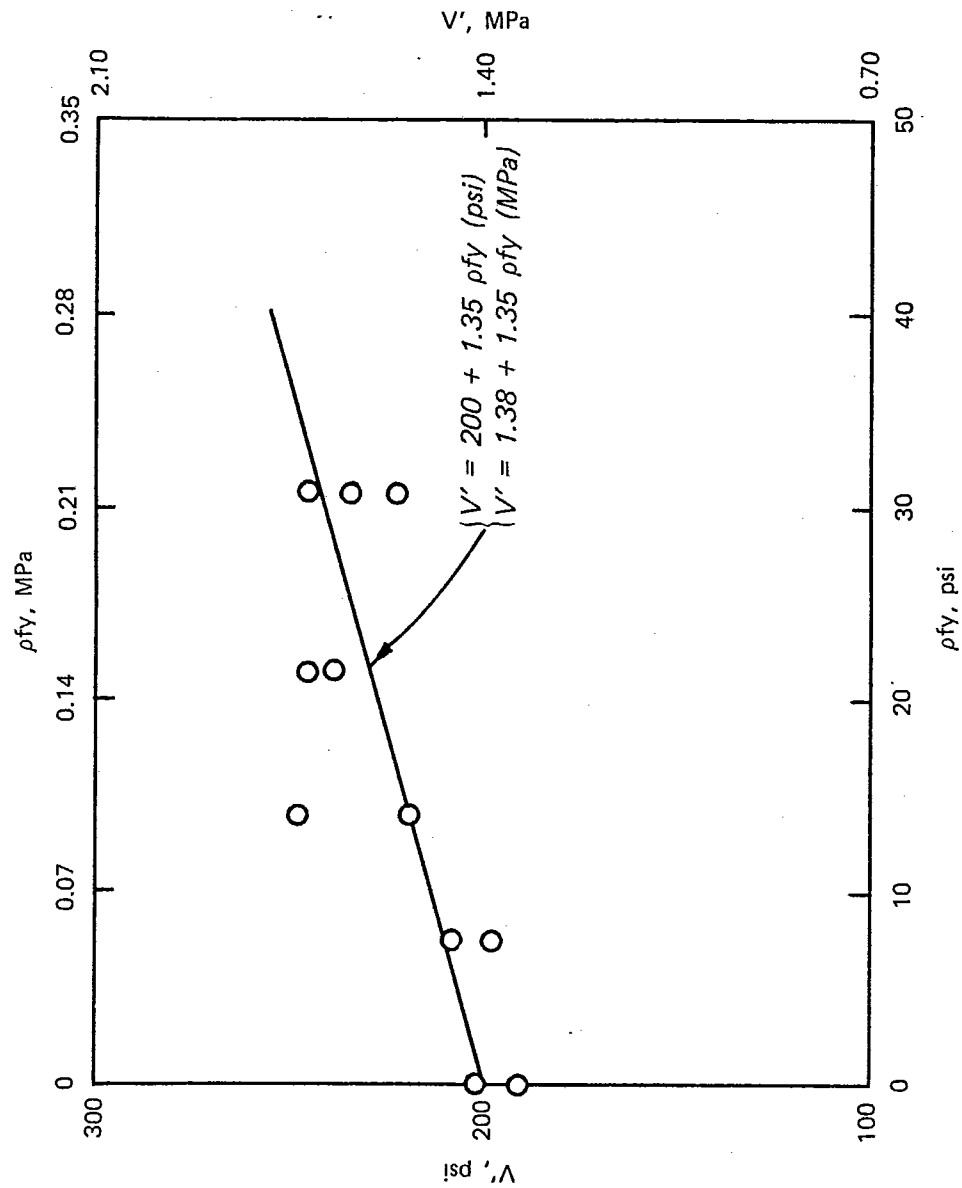


Fig. 8—Relation between shear strength V' and reinforcement parameter of ρf_y .

Friction Connection of Precast Concrete Construction

by C. Lin

Synopsis: This exploratory study was to investigate the feasibility of using friction type connection in precast concrete construction. Tests were conducted to study the relationship between the friction capacity of precast concrete contact surfaces and general design parameters, such as concrete strength, contact pressure, and shimming material. Lead sheet, rubber pad, and teflon sheet were used as shimming material in this study. Test results are presented to illustrate the effect of parameters on the friction capacity of concrete contact surfaces. It was found that the friction capacity of concrete contact surfaces is highly affected by the concrete strength and type of shimming material used on the specimen.

Finally, within the limitations of this test program, the test results indicate that the ultimate shear capacity of the friction type connection can reach the same shear capacity calculated by the shear-friction theory specified in ACI(318-77) for concrete placed against hardened concrete.

Keywords: connections; friction; lead (metal); performance; plastics, polymers, and resins; precast concrete; pressure; rubber; shear properties; strength; torsion

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INTRODUCTION

Dowel bars are normally provided in precast concrete construction to transfer shearing force from one member to the other. The mechanism in transferring of the shearing force was based on the combination of the dowel action of the steel bars and friction force between the contact surfaces of the concrete and steel. Based on the shear friction hypothesis⁽¹⁾ for rough contact surfaces, sliding motion will cause a separation of the two surfaces. The separation will develop tension in the dowel bars which, in turn, provide an external clamping force on the concrete, resulting in compression across the concrete-to-concrete interface. In order to develop the dowel action and the external clamping force, the dowel bars have to be embedded or grouted properly into concrete. However, when structural components have to be replaced periodically, an embedded or grouted joint will present problems.

An approach was considered herein for precast concrete construction that is similar to the friction type connection commonly used in steel construction, where the clamping force is developed by externally torqued bolts. The purpose of this introductory paper is to provide engineers with test data, on which to base decisions as to the feasibility of applying a friction type connection in precast concrete construction. Data provided in this study was to demonstrate the effect of design parameters, such as concrete strength, contact pressure, and shimming material on the friction capacity of precast concrete construction.

SPECIMEN PREPARATION AND TEST PROGRAM

The concrete for the specimens was made from Type I portland cement. The coarse aggregate used was crushed stone with a maximum size of 3/8 in. (9 mm) and the fine aggregate was local sand. The cement, sand, and coarse aggregate ratios selected were 1:2.5:3.5 and 1:2:4 by weight with a water cement ratio of 0.67. Three 2x4x40 in. (51x102x1016 mm) concrete bars with eight 3/4 in. (38 mm) diameter holes were cast from each batch and cured using standard laboratory procedures. Before testing, each concrete bar was saw cut into four 2x4x10 in. (51x102x254 mm) segments. Three segments, in which the loading and the bearing surfaces were capped with sulphate compound, were assembled as a set, as shown in Fig. 1. Two #3 rebars were used as tie rods, and both ends of the rebars were threaded. Quarter-inch steel plates, $\frac{1}{4}$ 2-1/2x2x1/4 in. (38x51x6 mm), were used as washers at both ends. A teflon sheet was used between the steel plate and the circular washer to reduce the friction. A hand torque wrench was used to apply torque to

the tie rods to generate a clamping tension. Subsequently, the contact pressure between concrete surfaces in contact was developed. The torque-tension relationship of the wrench was calibrated by the strain gages installed on the tie rod. By using the calibrated torque wrench, the tie rods were torqued sufficiently to produce the desired clamping force. The specimen was then placed in the testing machine. Three deflection dials were installed and set to zero as the initial reading. The compression force was applied and the corresponding deflections were recorded. A schematic drawing of the testing system is presented in Fig. 1.

A total of eleven specimens were tested in this study. Specimens were subjected to different contact pressures with or without shimming material between their contact surfaces. Lead sheets, rubber pads, and teflon sheets were used as shimming material in this investigation. A complete listing of the specimens and their properties is presented in Table 1. The results of the experimental work are reported in a series of graphs. Fig. 2 shows the results of calibration of the torque-tension relationship of the tie rod. Figs. 3 to 14 show the relationship between the shear friction and the slippage between the contact surfaces for the specimens tested.

TEST RESULTS

Torque-Tension Relationship

When torque was applied on the tie rod to develop the clamping force, some of the torque might have been absorbed by the friction forces between the washer and concrete.

Strain gages, one on each side, were installed on the tie rod to calibrate the relationship between the torque applied by the hand torque wrench and the tension generated in the tie rod. In Fig. 2, the torque applied to the tie rod is plotted against the corresponding average strain obtained from these two strain gages. A comparison was made between the test results and the equation used in steel construction for correlating the torque-tension relationship⁽²⁾,

$$T = KPD \quad (1)$$

where D is the nominal bolt diameter, P is the bolt tension, K is a dimensionless coefficient of proportionality, and T is the applying torque. A fair correlation was established for precast concrete elements by taking $K = 0.3$.

To evaluate the ultimate clamping tension that could be developed in the tie rod, the ultimate torque capacity of the tie rod was also tested. Two tests were performed and an average value of 50 ft.-lb. (68 N.m) was obtained. In both tests, failures occurred at the threaded part of the tie rods.

Effect of Contact Pressure

From examination of the test results shown in Table 2, it can be seen that there are no significant effects of the contact pressure on the friction capacity. The friction capacity remained proportional to the contact pressure applied on the specimen. However, during the latter stage of loading, when higher contact pressure was applied, a slight reduction in friction capacity was observed in specimens B2 and B4.

It is believed that the friction capacity of the specimen with shimming material was contributed by the shear resistance of the shimming materials, instead of the direct interlocking between the concrete contact surfaces. The shimming material in the specimen was subjected to a state of combined compressive and shearing stresses, in which the corresponding shear capacity is not necessarily proportional to the compressive stress applied to the material.

Effect of Concrete Strength

Initially, the same concrete mixing ratio of 1:2.5:3.5 by weight was used in batches A, B, and C. However, a slight difference in the concrete strength of each batch was observed, as shown in Table 1. Results of the test indicated that the friction capacity is affected by that slight difference in concrete strength. In order to verify the effect of the concrete strength on the friction capacity, a different concrete mix with a ratio of 1:2:4 by weight was used in batch D. Lower concrete strength for specimen D1 was observed, as shown in Table 1. Test results indicated that the friction capacity of specimen D1 is significantly reduced. Thus, it clearly indicated that the concrete strength does have a remarkable effect on the friction capacity of concrete contact surfaces. The real relationship between concrete strength and friction capacity is yet to be determined.

Effect of Shimming Material

The test results as summarized in Table 2 indicate that the friction coefficient is strongly affected by the types of shimming material used in the specimen. Within the limitations of the tests, the actual friction coefficient when using shimming material, varied from 0.27 to 0.92. Specimen A2, with lead sheet as the shimming material, showed the highest friction coefficient, 0.92, while specimen B2, with a thick rubber pad as the shimming material, showed the lowest coefficient of 0.27.

The shear-slippage relationships of specimens subjected to the same torque 25 ft.-lb. (24 N.m), but with different shimming material, were plotted in Fig. 14. Compare the results of specimen A1 with the specimens using different shimming material; the friction capacity is generally reduced when a shim is used. The overall behavior of specimen A2, with lead sheet as the shimming material, was very similar to specimen A1, in which a shim was not

used. In both cases, the friction coefficients were about 0.90 and the maximum friction capacities were developed when slippage reached 20×10^{-3} in. (0.5 mm). When teflon sheet was used in the specimen, a lower friction capacity was observed. The maximum friction capacity developed at a lower slippage and the friction capacity was reduced when slippage was further increased.

As seen in Fig. 5, specimen A3, with a thin rubber pad as the shimming material, reached its friction capacity with a very high corresponding slippage. After the applying force was removed, a significant portion of the slippage was recovered. It is believed that the recovery of slippage was attributed to the elastic deformation of the rubber pad. The residual slippage was the result of the sliding between concrete surfaces and the shimming material.

It should also be noted that during the pretensioning process, when torque was applied on the specimens without shimming material, localized stresses were generated and some specimens were cracked. However, when a shimming material was introduced in the specimen, the clamping force was more uniformly distributed and the localized stress was significantly reduced; and, therefore, no cracks were observed.

Consistency of Test Results

Even in the same set of testing specimen, many factors, such as surface condition of the concrete, shimming material, tie rod, nut, and washer, could have an effect on the coefficient of the friction between concrete and concrete or concrete and shimming material. Unfortunately, these factors are random parameters; therefore, they are very difficult to appraise for the design purpose. Tests were performed in this investigation to study the effect of those factors on the consistency of the performance. The same specimen with the same test parameters was tested twice. During the first loading cycle, a typical test was performed to obtain the shear force and slippage relationship for that specimen. The specimen was then completely disassembled and then re-assembled following the same procedure with the same equal torque loading being applied. A test similar to that in the first loading cycle was conducted. The shear forces and the corresponding slippages were again recorded. This type of test was performed on three specimens, C₁, C₂, and C₃. Results of these tests are shown in Figs. 10 to 12. In examination of the figures, although a slight difference in slippage corresponding to the maximum friction capacity was observed, the magnitude of the shear capacity is about equal for both tests performed on each specimen. Since the shear capacity is the main concern of this study, it may be concluded that fairly consistent results were obtained from this test.

Friction Type Connection

Based on the tested ultimate torsional capacity of the tie rod and the average friction capacity per unit torque listed in Table 2, the friction capacity for each specimen was calculated and summarized in Table 1. Compare the calculated friction capacity with the shear-friction theory (Eq. 11.26) specified in ACI code (318-77)(3)

$$V_n = A_{vf} F_y \mu \quad (2)$$

where V_n is the shear strength, A_{vf} is the area of shear-friction reinforcement, and μ is the coefficient of friction. The corresponding coefficient of friction, μ , for each specimen was calculated and summarized in Table 1. As indicated in the table, specimen A1 without shimming material and A2 with lead sheet as shimming material have a friction coefficient of 0.76 and 0.74, respectively, when they were calculated based on gross steel area. Nevertheless, when a friction type connection is used, the tie rod has to be threaded; and the cross sectional area is, therefore, reduced⁽⁴⁾. If the reduced tensile stress area is used in Eq. (2) to calculate the shear capacity of the friction type connection, the coefficient of friction, μ , can be taken as unity. This is the same value used in ACI code (318-77) when concrete is placed against hardened concrete. It should be noted, per ACI (section 11.7.9), when concrete is placed against previously hardened concrete, the interface for shear transfer shall be intentionally roughened to a full amplitude of approximately 1/4 in. (7 mm). In this experiment, the contact surfaces were the formed surface which was not intentionally roughened.

Low friction coefficients were observed for specimens with a rubber pad as shimming material. A large deformation was observed corresponding to the friction capacity developed. If high ductility is the main requirement in a joint construction, rather than the shear capacity of the joint, a rubber pad may be considered as shimming material.

SUMMARY AND CONCLUSIONS

This exploratory test program demonstrated the feasibility of using friction type connections in precast concrete construction. Within the limitations of this test program, the following conclusions may be drawn from the results of these tests.

- (1) The torque-tension relationship of the tie rod can be fairly correlated by Eq. (1) with $K = 0.3$.
- (2) The material properties of shimming material appear to have a significant influence on the friction coefficient. The behavior of the specimen with a lead sheet as the shimming material was similar to that of the comparable specimens without the shimming material. A significant reduction in the friction capacity was observed in the specimen using either a rubber pad or a teflon sheet as the shimming material.

- (3) The contact pressure between concrete surfaces does not affect the coefficient of friction of the specimens without shimming material but affects the specimen with a rubber pad or the teflon sheet as the shimming material subjected to high pressure. The friction coefficient was slightly reduced with increased contact pressure.
- (4) Due to a greater reduction in the localized stress and better distribution of contact pressure to a larger area, lead sheets offer good shear transfer in friction type connection without a significant reduction in shear transferring capacity.
- (5) Consistent performance was observed in this experiment. Therefore, with careful arrangement, a friction type connection may be used to transfer shearing force between contact surfaces in precast concrete construction. However, the concrete strength has a remarkable effect on the friction coefficient. The real relationship between concrete strength and friction capacity is yet to be determined.
- (6) The effect of concrete creep and steel relaxation on clamping tension in tie rod and the effect of temperature on lead sheet should also be carefully examined.

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APPENDIX - NOTATIONS

A_{vf}	=	area of shear friction reinforcement
D	=	nominal bolt or tie rod diameter
f_y	=	specified yield strength of non-prestressed reinforcement
K	=	dimensionless coefficient
P	=	bolt or tie rod tension
T	=	applied torque
V_n	=	shear strength
μ	=	coefficient of friction, shear friction theory
μ_f	=	actual coefficient of friction, when $K = 0.3$

Table 1 Details of Specimens and Test Results

Specimen Number ^a	f'_c Psi (MPa)	Shimming Material ^b	Test Result	Ultimate Friction Capacity Kips (KN)	Shear-Friction Coefficient μ	Actual Friction μ_f
A1	3850(26.5)	None	Fig. 3	20.0(89.0)	0.76	0.94
A2	3850(26.5)	Lead	Fig. 4	19.6(87.2)	0.74	0.92
A3	3850(26.5)	Rubber-Thin	Fig. 5	6.2(27.6)	0.24	0.29
B1	3458(23.8)	None	Fig. 6	17.6(78.3)	0.66	0.82
B2	3458(23.8)	Rubber-Thick	Fig. 7	5.9(26.2)	0.22	0.27
B3	3458(23.8)	Teflon-1X	Fig. 8	12.8(56.9)	0.48	0.60
B4	3458(23.8)	Teflon-2X	Fig. 9	7.4(32.9)	0.28	0.35
C1	3051(21.0)	None	Fig. 10	17.5(77.8)	0.66	0.82
C2	3051(21.0)	Lead	Fig. 11	13.7(60.9)	0.52	0.64
C3	3051(21.0)	Rubber-Thick	Fig. 12	6.7(29.8)	0.25	0.31
D1	2415(16.7)	None	Fig. 13	12.8(56.9)	0.48	0.60

a A1: Alphabetic order is the concrete mixing batch.

b Material thicknesses: Lead = 1/16 in., Rubber-Thin = 1/32 in.,

Rubber-Thick = 1/8 in., Teflon-1X (each layer) = 1/32 in.

Table 2 Friction Capacity Per Unit Torque

Specimen Number	Applied Torque on Specimens, ft.-lb. (N.m)					Average
	10 (13.6)	15 (20.3)	20 (27.1)	25 (33.9)	35 (47.5)	40 (54.2)
	Friction Capacity Per Unit Torque, lb./ft.-lb. (N/N.m)					
A1	380(1690)	436(1939)	395(1757)	390(1735)	-	400(1781)
A2	-	447(1988)	-	390(1735)	343(1525)	392(1744)
A3	-	-	-	120(534)	128(569)	124(522)
B1	409(1801)	317(1410)	-	356(1584)	-	351(1560)
B2	-	140(623)	-	112(498)	83(369)	117(519)
B3	-	216(961)	-	292(1299)	257(1143)	255(1134)
B4	-	226(1005)	-	96(427)	123(547)	148(660)
C1	-	-	-	-	-	350(1556)*
C2	-	-	-	-	-	273(1214)*
C3	-	-	-	-	-	133(592)*
D1	225(1001)	276(1228)	265(1179)	-	-	255(1134)

* Average Value at Torque = 30 ft.-lb.

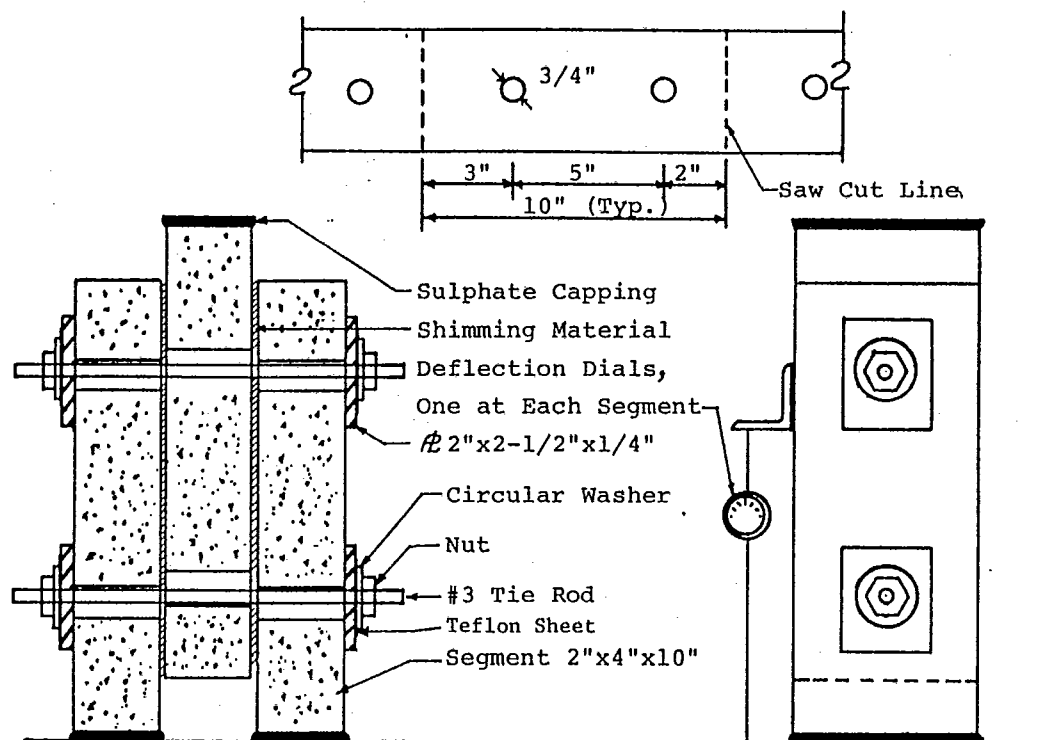


Fig.1--Test specimen and assembly

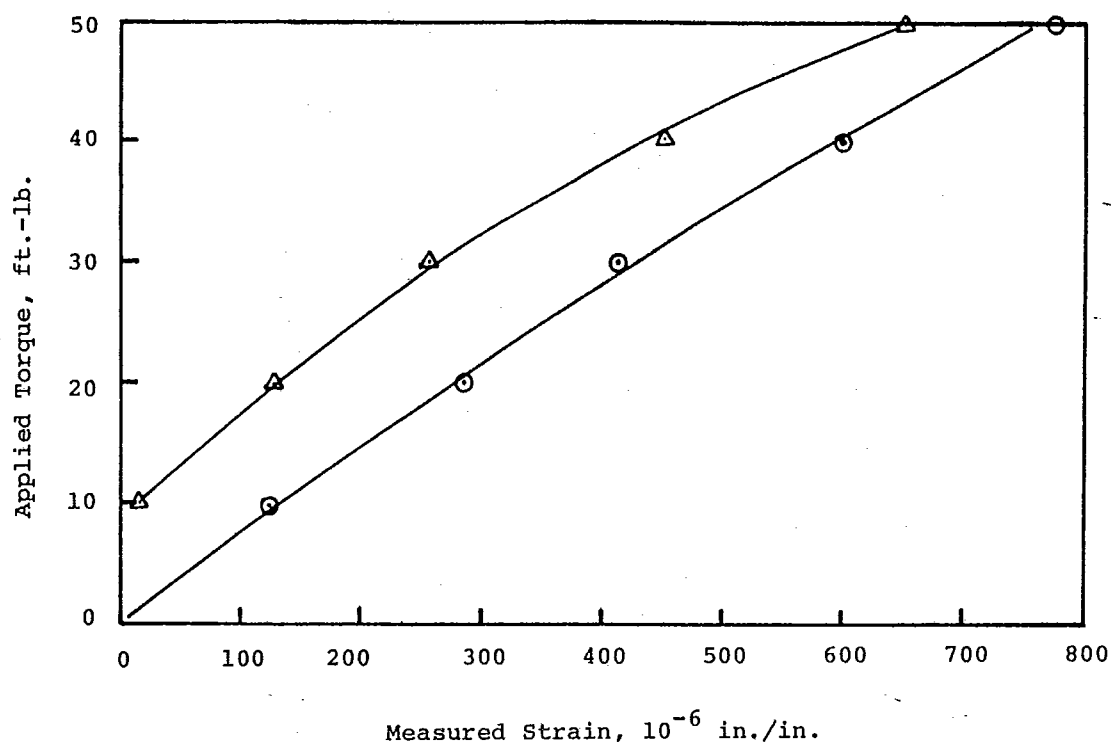


Fig. 2--Measured strain-applied torque relationship for #5 rebar

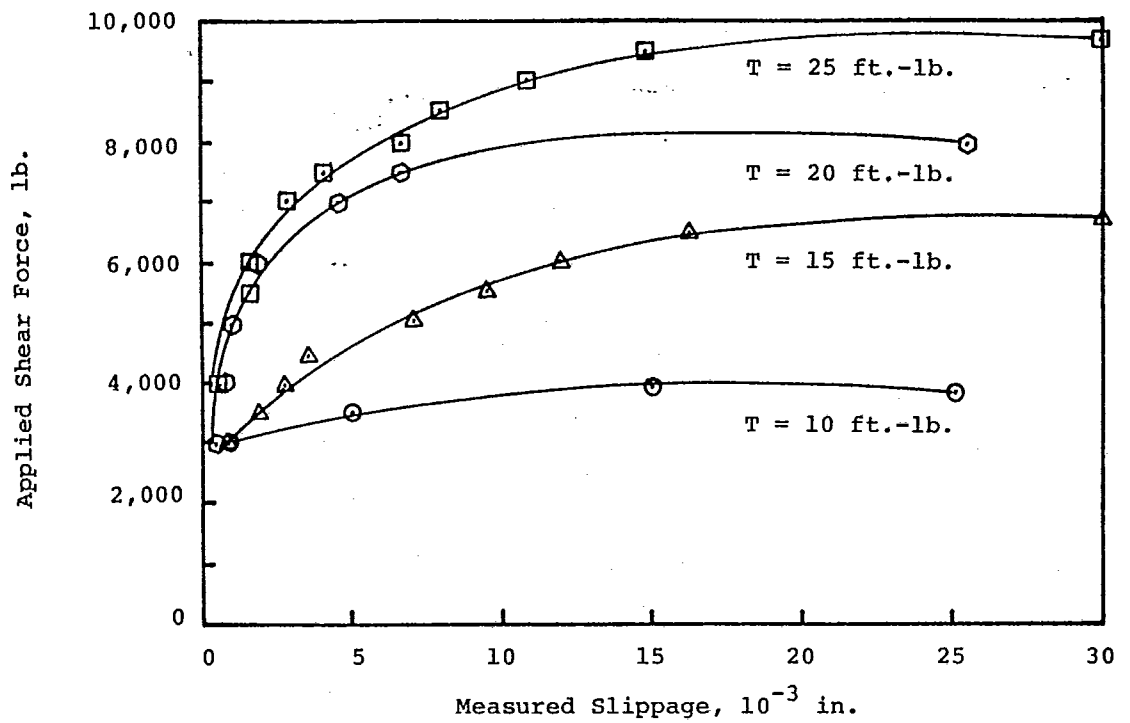


Fig. 3—Shear-slippage relationship for specimen A1 (no shimming material)

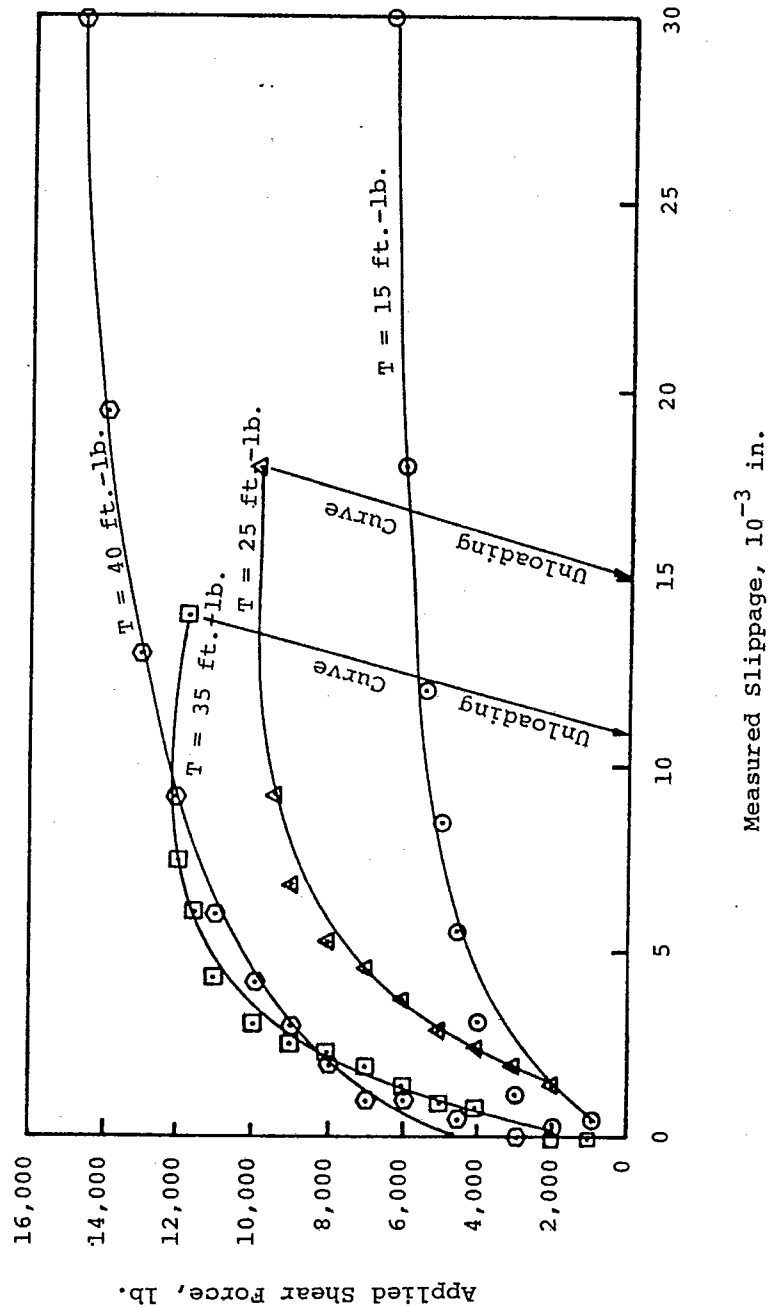


Fig. 4--Shear-slippage relationship for specimen A2 (lead shimming material)

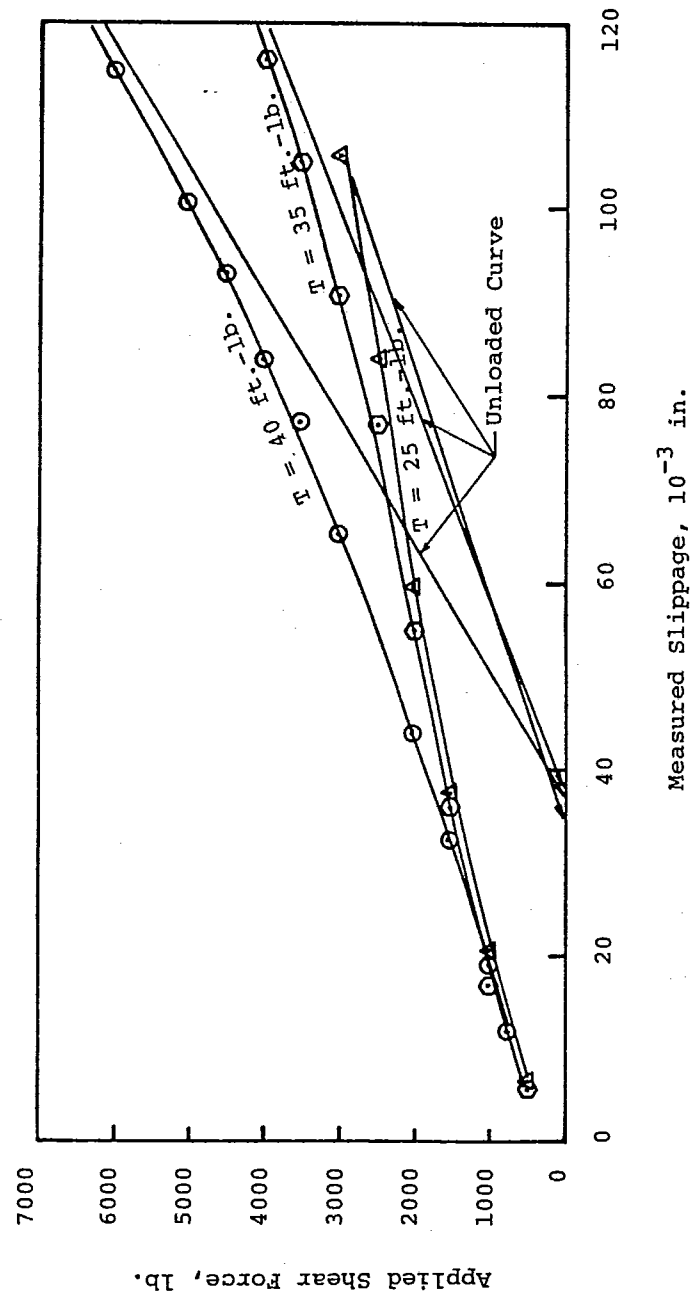


Fig. 5--Shear-slippage relationship for specimen A3 (thin rubber shimming material)

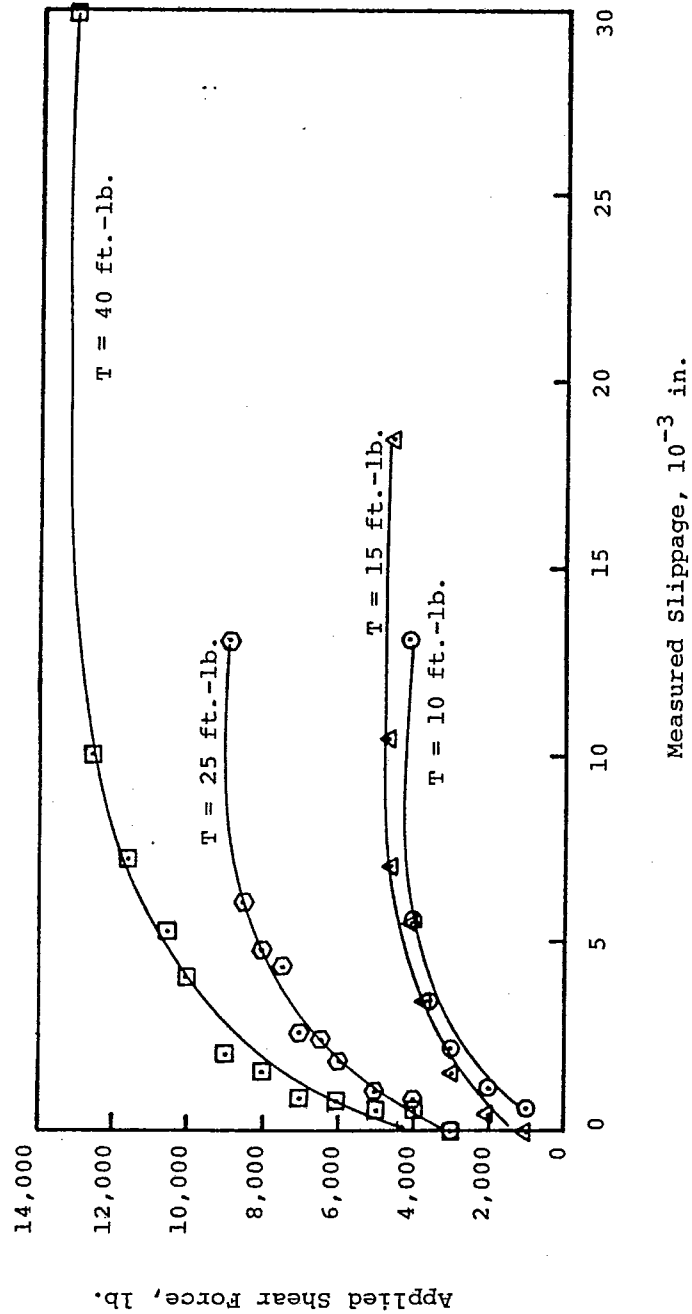


Fig. 6--Shear-slippage relationship for specimen B1 (no shimming material)

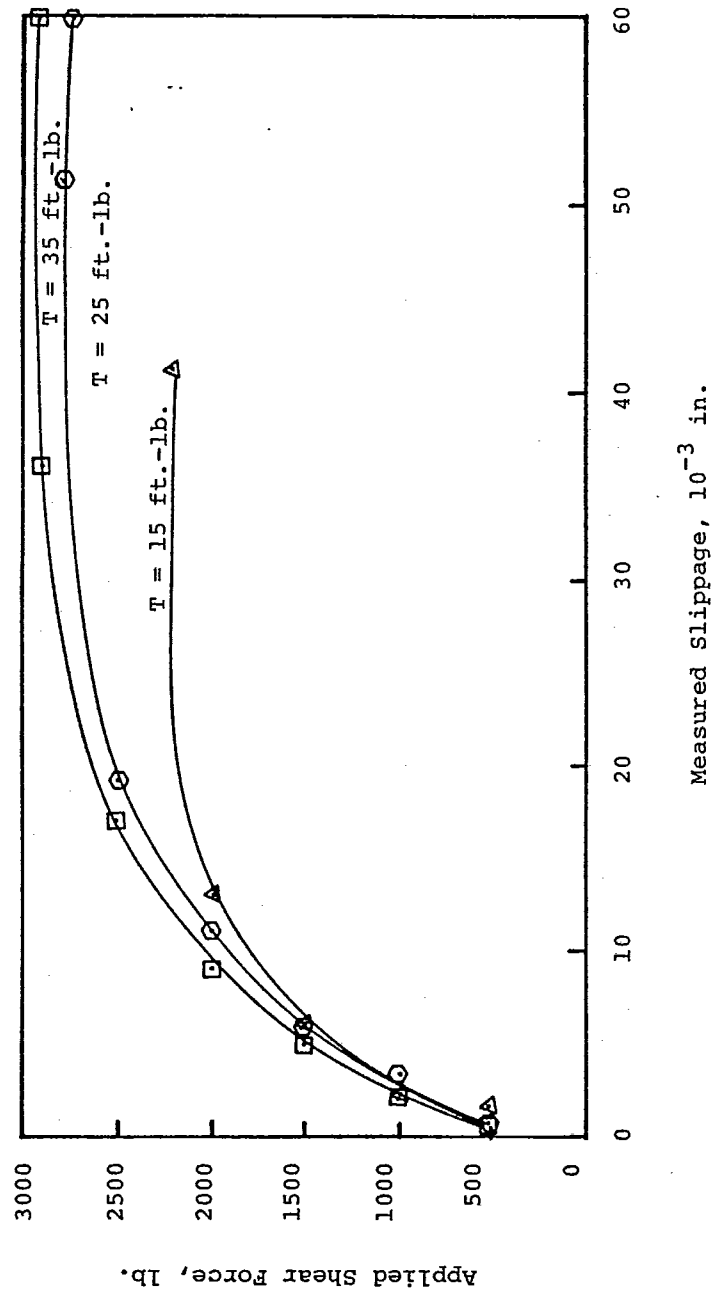


Fig. 7--Shear-slippage relationship for specimen B2 (thick rubber shimming material)

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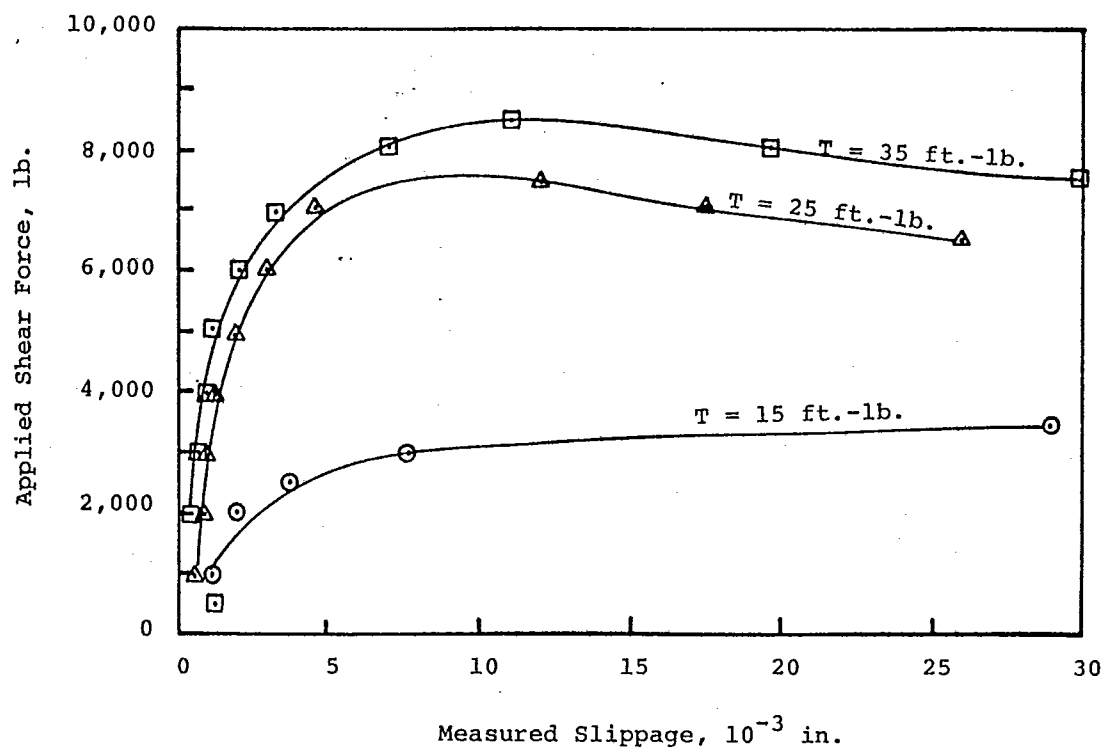


Fig. 8--Shear-slippage relationship for specimen B3 (one layer Teflon shimming material)

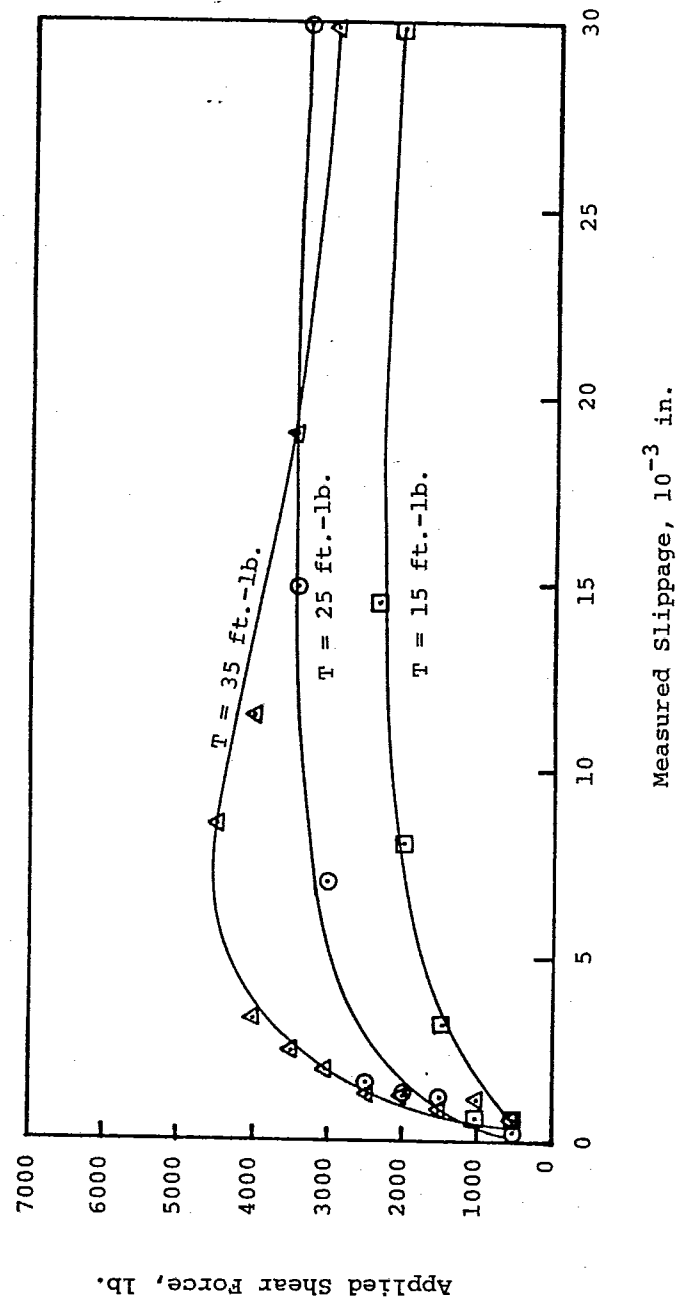


Fig. 9--Shear-slippage relationship for specimen B4 (two layers Teflon shimming material)

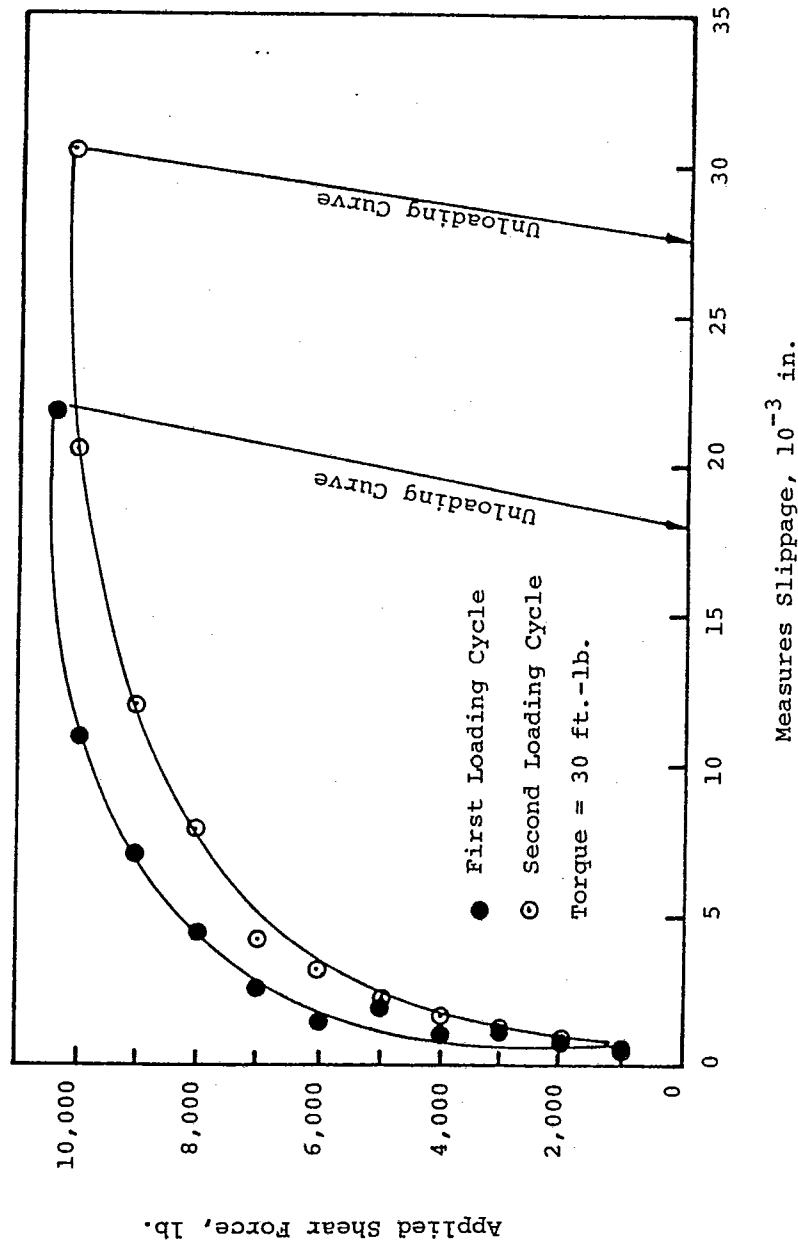


Fig. 10--Shear-slippage relationship for specimen C1 (no shimming material)

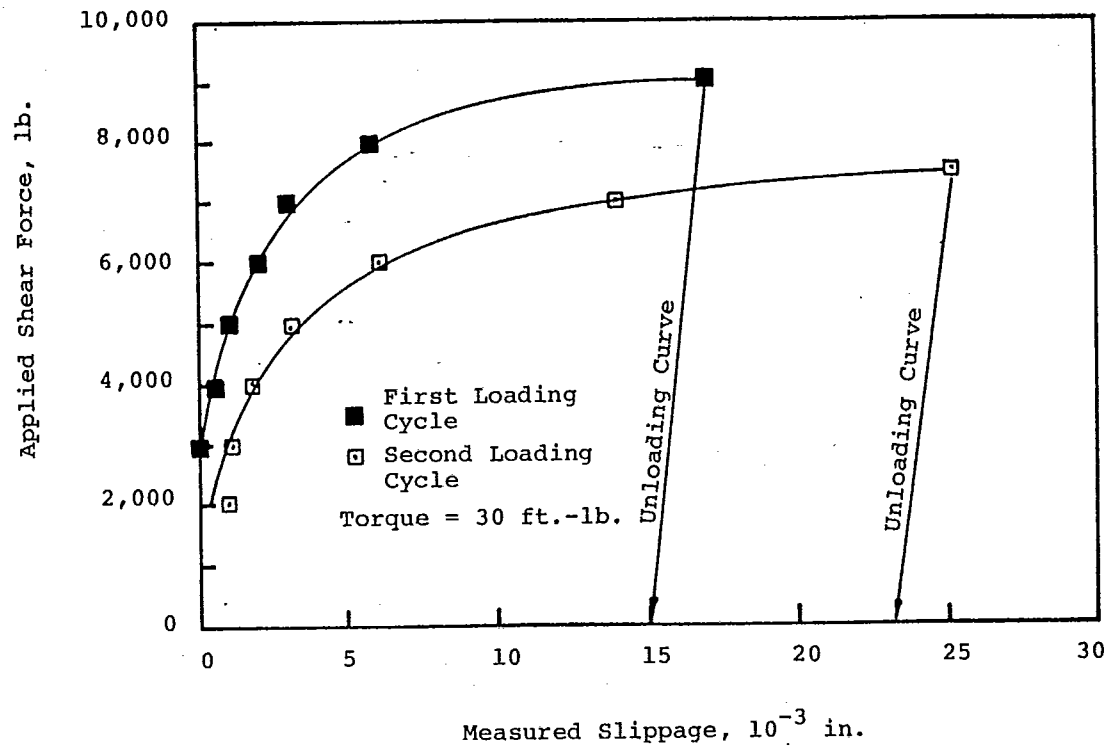


Fig. 11--Shear-slippage relationship for specimen C2 (lead shimming material)

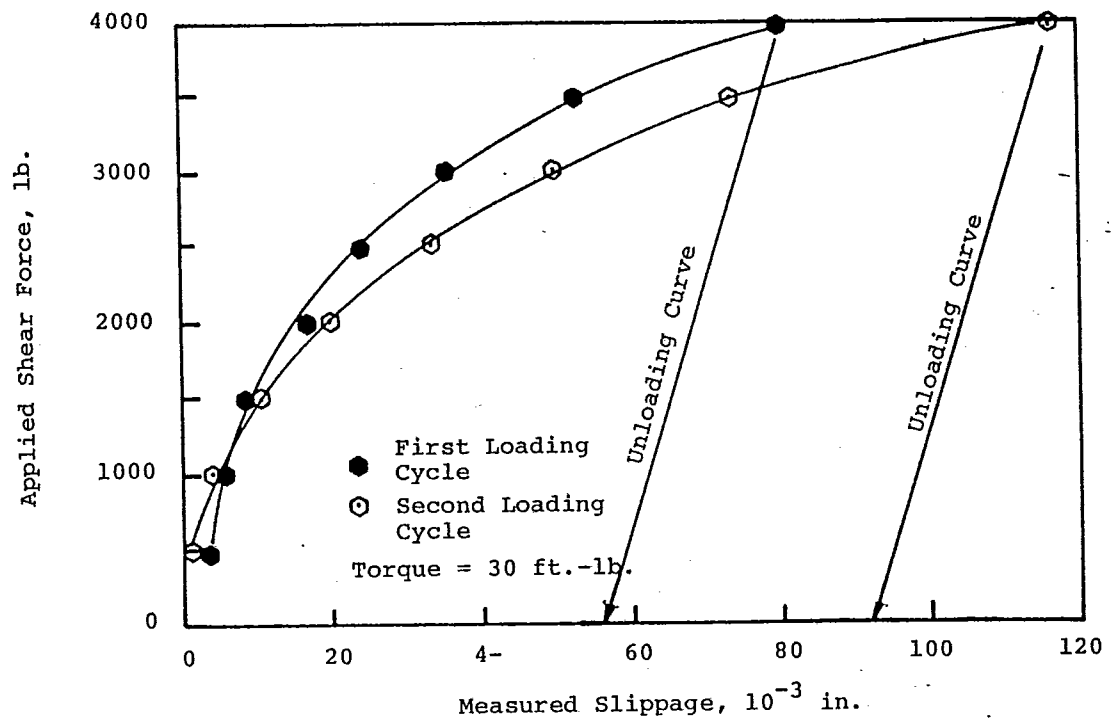


Fig. 12--Shear-slippage relationship for specimen C3 (thick rubber shimming material)

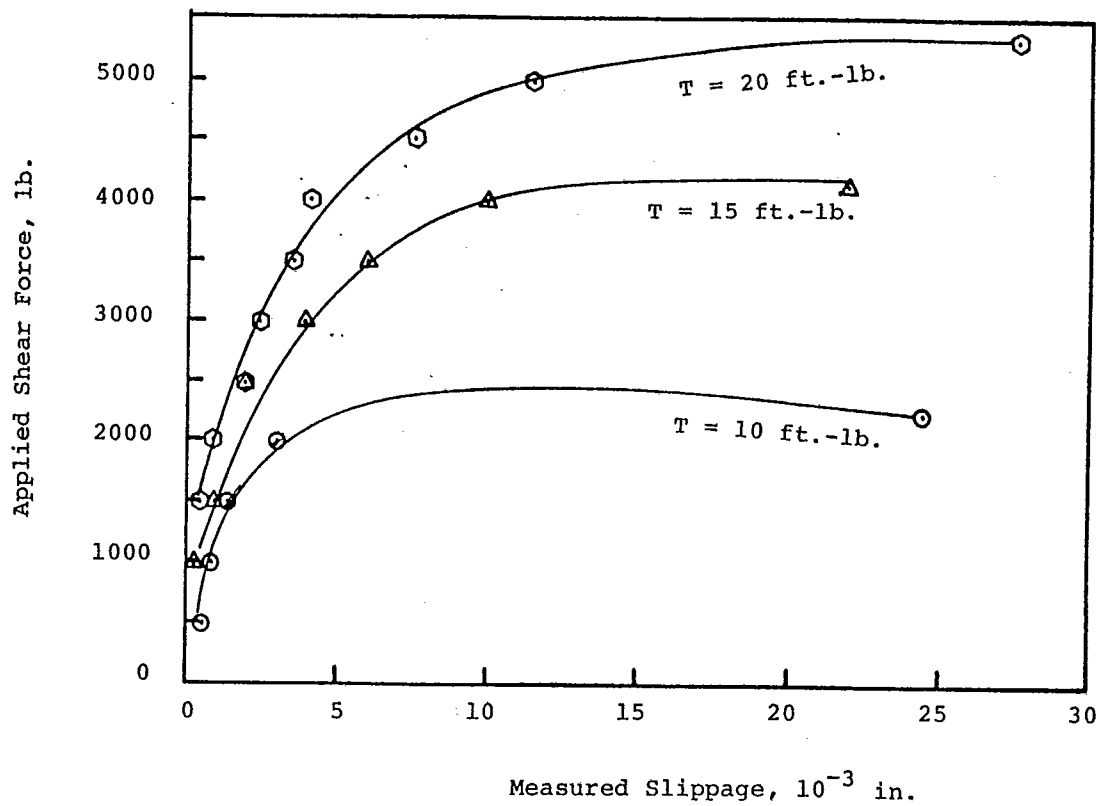


Fig. 13--Shear-slippage relationship for specimen D1 (no shimming material)

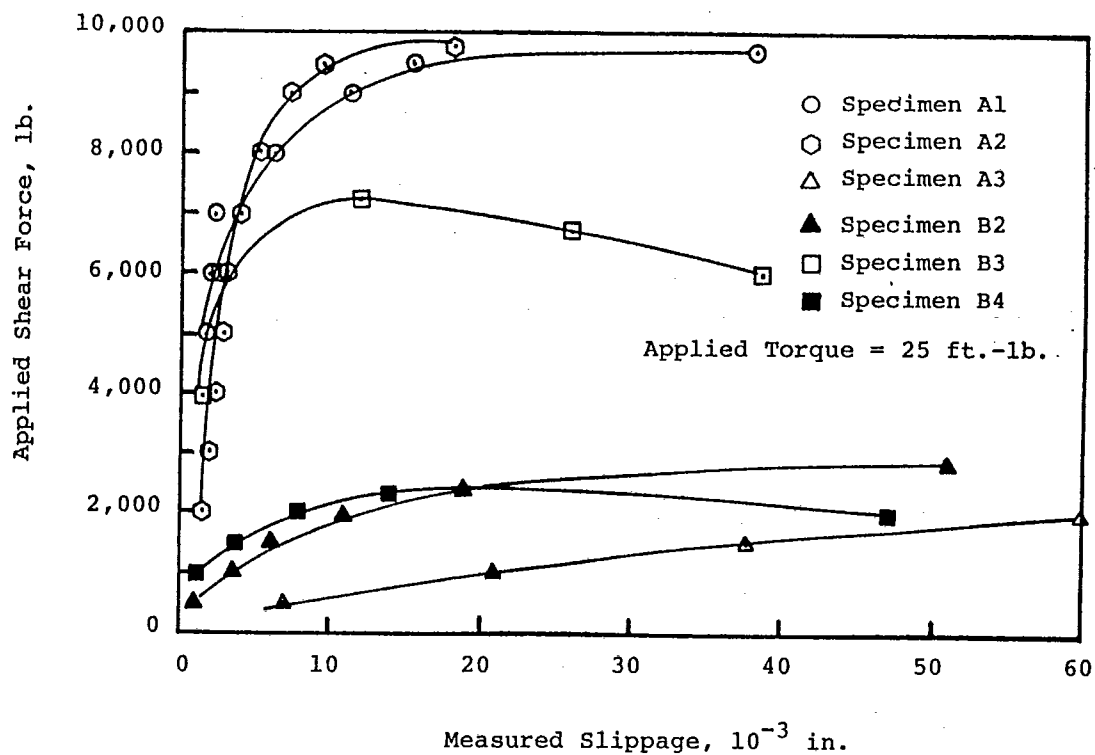


Fig. 14--Shear-slippage relationship for specimens A1, A2, A3, B2, B3, B4