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STRENGTH AND BEHAVIOR OF ANCHOR BOLTS EMBEDDED NEAR EDGES OF CONCRETE PIERS

By

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J. E. Breen, and K. Lo

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16. Abstract

The primary objective of this investigation was to evaluate the effects of bolt diameter, embedment length, clear cover, and bearing area on the behavior of high-strength anchor bolts. Both model and full-scale tests were conducted. In addition, a series of exploratory full-scale tests were run to determine the influence of cyclic load, lateral load, bolt groups, and transverse reinforcement on the behavior. The tests showed that the anchor bolt transfers loads to the concrete member by a sequence involving steel-to-concrete bond, bearing against the washer of the anchorage device, and, finally, wedging action by a cone of crushed and compacted concrete in front of the anchorage device. The parameters were evaluated by comparing bolt stress slip curves. A regression analysis of the test results was used to develop a design equation for anchor bolts.

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Conducted for

Texas
State Department of Highways and Public Transportation

In Cooperation with the
U. S. Department of Transportation
Federal Highway Administration

by

CENTER FOR HIGHWAY RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

May 1977

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

P R E F A C E

This report presents an extensive study of the strength and behavior of anchor bolt installations. The objective of the project was to develop design procedures for high-strength anchor bolt applications.

This is the final report on work conducted under Project 3-5-74-29 "Strength and Behavior of Anchor Bolts." An earlier report (29-1) "A Guide to the Selection of High-Strength Anchor Bolt Materials" describes work done to examine the types and characteristics of materials available for use as anchor bolts.

The work was sponsored by the Texas Department of Highways and Public Transportation and the Federal Highway Administration and administered by the Center for Highway Research at The University of Texas at Austin. Close liaison with the Texas Department of Highways and Public Transportation has been maintained through Mr. Warren Grasso, the contact representative, and with Mr. Jerry Bowman of the Federal Highway Administration.

S U M M A R Y

The primary objective of this investigation was to evaluate the effects of various factors on the ultimate load capacity and the behavior of high-strength anchor bolts embedded near edges of concrete piers. The investigation involved both full-scale and model tests. The main factors studied were bolt diameter, embedment length, clear cover (from concrete surface to anchor bolt), and bearing area of the end anchorage. A series of exploratory full-scale tests was run to examine the importance of the cyclic loading, lateral forces, bolt grouping, and transverse reinforcement on anchor bolt behavior.

The test results indicated that there were three distinct failure modes that could be characterized by the geometry of the anchor bolt. The modes of failure include bolt yielding, cover spalling (localized loss of cover near end anchorage device), or wedge splitting (general loss of cover over bolt and end anchorage device). Examination of these failure modes led to the development of a general description of the load-carrying mechanisms of high-strength anchor bolts. The tests showed that the anchor bolt transfers loads to the concrete member by a sequence involving steel-to-concrete bond, bearing against the washer of the anchorage device, and, finally, wedging action by a cone of crushed and compacted concrete in front of the anchorage device. In addition, the effects of the main factors listed above are evaluated by comparing bolt stress versus slip of the anchor bolt relative to the concrete. In the development of a design equation for anchor bolts, several design parameters are discussed and a design equation is presented.

I M P L E M E N T A T I O N

The test results indicated that the clear cover and bearing area were the prime variables influencing the strength of anchor bolts. In order to incorporate the variables into an equation for predicting the strength of isolated anchor bolts subjected to tension only, a regression analysis of the data was carried out. Because the mode of failure influenced strength, only the results of specimens failing in a wedge-splitting mode were considered. From the regression analysis, the following design equation for the tensile capacity (in lbs) of an anchor bolt was developed.

$$T = \varphi 140 A_b \sqrt{f'_c} \left[0.7 + \ln \left(\frac{2C'}{D_w - D} \right) \right] \leq A_{sm} f_y$$

where A_b is the net bearing area (in.^{2}), D and D_w are the bolt and washer diameter (in.), C' is the clear cover to the bolt (in.), and φ is a capacity reduction factor to account for scatter in the test results and for variations in material properties or dimensional inaccuracies (0.75) for anchor bolts). The design equation provides a reasonable estimate of strength when compared with the test results and reflects the critical parameters observed in the test program. It should be noted that the equation is intended to apply only to cases where an isolated bolt is embedded near the edge of a concrete pier and where the embedment length is sufficient to preclude a failure not involving side cover spalling. The proposed design equation will provide guidance in an area where current design recommendations offer none. Implementation of the design equation should result in better control of the parameters influencing anchor bolt strength.

The exploratory tests indicated lateral forces normal to the edge significantly reduce the strength. The strength of bolts in groups was drastically reduced over that for an isolated bolt. The use of transverse reinforcement around the anchor bolt improved both strength and ductility of the anchor bolt installation. However, the number of tests conducted was not sufficient to permit extensions of the proposed equation to include these parameters and additional research is needed to clarify these aspects of anchor bolt behavior.

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CHAPTER 1

INTRODUCTION

1.1 General

Anchor bolts are commonly used in highway structures to connect appurtenances such as light standards, sign supports, and rails, as well as shoes and other supports for steel members, to structural concrete members. Despite their extensive use in structures of all types, current standard design codes do not contain provisions for the design of anchor bolts. During the period from 1963 to 1966, two investigations[1,2] were conducted at The University of Texas at Austin to study the factors affecting the strength and the behavior of anchor bolts.

The primary objective of the first study[1] was to determine the required embedment length for A7 (33 ksi yield stress) anchor bolts with an end anchorage consisting of a nut or a nut and a standard washer. Bolts with diameters from 1-1/4 in. to 3 in. were tested. It was found that an embedment length of 10 bar diameters was sufficient to develop bolts with diameters less than 2-1/2 in., but that 15 bar diameters was required for the 3 in. diameter bolts. It was determined that the main load-carrying element of the anchor bolt was the end anchorage, and that concrete-to-steel bond played a relatively minor role in the development of the strength of an anchor bolt installation. It was also found that a significant parameter governing the strength of an anchor bolt installation was the amount of concrete cover over the bolt. In order to establish a basis for design criteria for anchor bolts, an empirical expression was developed relating an increase in ultimate bolt strength to an increase in clear cover normalized with respect to bolt diameter (C'/D). Since the range of variables examined in this study was limited, it was recommended that the investigation be extended in order to determine more definitive design recommendations.

The second study[2] investigated the effects of clear cover, low-cycle repeated loading, circular shape of the specimen, low concrete strength, 90° bends as anchorage devices, and the method of loading. This study used 60 ksi bolts with diameters of 1-1/4 in. and 2 in. Each bolt had an embedment length of 10 diameters and an end anchorage consisting of a standard nut. It was found that the method of loading did not significantly influence the bolt strength, but the presence of a lateral compressive force in the length of the bolt, characteristic of the loading method used in the first study, increased the stiffness of the anchor bolt. The results indicated that concrete strength was an important factor that affected the bolt ultimate strength and the behavior. Low concrete strength caused a definite reduction in bolt strength and a significant reduction in stiffness near the ultimate load. It was shown that the effects of low-cycle loads and specimen shape were relatively minor. The tests on bolts with an anchorage device of a 90° bend were limited, but the results indicated that the 90° bends were not as efficient as the standard nut anchorage.

The main result of the second study was the establishment of clear cover as the single most significant factor in the development of the strength of an anchor bolt installation. Using the results of both studies, an expression was developed that related ultimate bolt strength to the ratio of clear cover to bolt diameter.

The expression developed in the second study[2] was suitable for use in the design of anchor bolts. However, as the magnitude of the force increases, larger diameter bolts of high strength steels are being required. Therefore, in order to extend the results of the earlier studies to materials and loading conditions commonly encountered in current practice, a study was undertaken to evaluate the performance of high-strength anchor bolts (yield strengths on the order of 110 ksi). The material used was ASTM-A193 - Grade B7 alloy steel, which was readily available at reasonable cost and was determined to be one of the most acceptable materials from the work done earlier in this study.[3]

1.2 Object and Scope

The primary objective of this investigation was to evaluate the effects of various factors on the ultimate load capacity and the behavior of high-strength anchor bolts. The investigation involved both full-scale[6] and model tests.[5] The main factors studied were:

(1) Bolt Diameter - Bolts of 1 and 1-3/4 in. diameter were used in the full-scale tests and 1/2 in. diameter bolts were used in all the model tests.

(2) Embedment Length - Bolts with embedment lengths of 10, 15, and 20 bar diameters were tested.

(3) Clear Cover - Values of clear cover ranged from 1.0 in. to 4.5 in. for the 1 in. bolts, and from 2.5 in. to 6.0 in. for the 1-3/4 in. bolts in the full-scale tests. Clear cover in the model tests ranged from 0.5 in. to 2.25 in.

(4) Bearing Area - Various washer sizes were used in both full-scale and model tests and provided a variation in bearing area with variation in clear cover.

A series of exploratory tests was run to determine the influence of the following parameters on anchor bolt behavior:

(1) Cyclic Loads - Repetition of load for 40-50 cycles to evaluate changes in stiffness or strength (two tests).

(2) Lateral Load - Application of lateral load normal to edges to determine influence on splitting (two tests).

(3) Bolt Groups - Two-bolt groups were tested to determine influence of interaction on performance (three tests).

(4) Transverse Reinforcement - Hairpin bars were placed along the anchor bolt to determine whether the bars would resist the tendency for splitting of the cover over the anchor bolt (two tests).

All exploratory tests were run using full-scale specimens.

The test results indicated that there were three distinct failure modes that could be characterized by the geometry of the anchor bolt. Examination of these failure modes led to the development of a general description of the load-carrying mechanisms of high-strength anchor bolts. It is shown that the anchor bolt transfers loads to the concrete member by a sequence involving steel-to-concrete bond, bearing against the washer of the anchorage device, and finally, wedging action by a cone of crushed and compacted concrete in front of the anchorage device. In addition, the effects of the main factors listed above are evaluated by comparing stress-slip curves. In the development of a design equation for anchor bolts, several possible design parameters are discussed and a design equation is presented.

C H A P T E R 2

SPECIMENS AND TESTING EQUIPMENT--FULL-SCALE TESTS

2.1 Description of Specimens

2.1.1 Specimen Geometry. The test program consisted of 35 anchor bolts embedded in nine specimens. Each test consisted of a single bolt loaded to simulate conditions in a typical drilled shaft footing.

Each bolt is identified by a specific designation, as shown below:

1.75 × 15D × 4.50(4.00)B

Washer Diameter, in.

Clear Cover, in.

Embedment Length, bolt diameters

Bolt Diameter, in.

A suffix indicates a second bolt test of a particular geometry (B), or special confinement conditions (U or H), or an exploratory test; cyclic loading (C), lateral load (V), or two-bolt group (G). Table 2.1 summarizes the full scale test program.

Figure 2.1 shows general specimen dimensions and reinforcement details. All nine specimens were identical except for the anchor bolts. Each specimen, measuring 3 ft. × 3 ft. × 8 ft., was cast in a vertical position with one anchor bolt positioned vertically along the centerline of and parallel to each side. The large magnitude of bolt load that was anticipated dictated the size of the loading equipment, which in turn dictated the size of the specimen. Four bolts were placed in each specimen in order to enable as many tests as possible to be performed with a minimum amount of material and labor.

TABLE 2.1 SUMMARY OF FULL-SCALE TESTS

Bolt	Bolt Diameter (in.)	Clear Cover (in.)	Embedment Length (in.)	Washer Diameter (in.)	f'_c (psi)	Ultimate Load (kips)	Ultimate f_{sm} (ksi)
1.00x15Dx1.00(2.50)	1	1.00	15.0	2.50	5500	62.0	102.3
1.00x15Dx2.50(2.50)	1	2.50	15.0	2.50	3910	77.0	127.1
1.00x15Dx3.50(2.50)	1	3.50	15.0	2.50	3520	76.8	126.7
1.00x15Dx3.50(2.50)B	1	3.50	15.0	2.50	4290	81.6	134.7
1.00x15Dx4.50(2.50)	1	4.50	15.0	2.50	4910	78.3	129.2
1.75x15Dx2.50(4.00)	1-3/4	2.50	26.25	4.00	3950	139.8	67.1
1.75x15Dx3.50(4.00)	1-3/4	3.50	26.25	4.00	3630	149.4	71.8
1.75x15Dx4.50(4.00)	1-3/4	4.50	26.25	4.00	4680	178.3	85.6
1.75x15Dx4.50(4.00)B	1-3/4	4.50	26.25	4.00	4310	168.0	80.7
1.75x15Dx6.00(4.00)	1-3/4	6.00	26.25	4.00	3980	212.9	102.3
1.00x20Dx2.50(2.50)	1	2.50	20.0	2.50	3880	79.3	130.9
1.00x20Dx3.50(2.50)	1	3.50	20.0	2.50	3930	75.9	125.2
1.75x20Dx3.50(4.00)	1-3/4	3.50	35.0	4.00	3680	143.4	68.9
1.75x20Dx4.50(4.00)	1-3/4	4.50	35.0	4.00	4910	188.3	90.4
1.00x10Dx2.50(2.50)	1	2.50	10.0	2.50	5110	61.0	100.7
1.75x10Dx3.50(4.00)	1-3/4	3.50	17.5	4.00	5480	139.6	67.1
1.75x10Dx6.00(4.00)	1-3/4	6.00	17.5	4.00	5120	157.0	75.4
1.00x15Dx2.50(3.25)	1	2.50	15.0	3.25	5480	81.7	134.8
1.00x15Dx4.50(4.50)	1	4.50	15.0	4.50	4290	81.7	134.8
1.75x15Dx3.50(3.00)	1-3/4	3.50	26.25	3.00	2640	68.0	32.7
1.75x15Dx3.50(3.25)	1-3/4	3.50	26.25	3.25	4300	155.4	74.6
1.75x15Dx3.50(3.50)	1-3/4	3.50	26.25	3.50	5470	148.9	71.5
1.75x15Dx3.50(5.00)	1-3/4	3.50	26.25	5.00	2770	117.8	56.6
1.00x15Dx2.50(2.50)U	1	2.50	15.0	2.50	5260	79.8	131.7
1.75x15Dx3.50(4.00)U	1-3/4	3.50	26.25	4.00	5380	163.5	78.5
1.75x15Dx3.50(5.00)U	1-3/4	3.50	26.25	5.00	3960	157.0	75.4
1.00x15Dx2.50(2.50)H	1	2.50	15.0	2.50	5260	76.2	125.7
1.75x15Dx3.50(4.00)H	1-3/4	3.50	26.25	4.00	5380	207.9	99.9
1.00x15Dx2.50(2.50)C	1	2.50	15.0	2.50	5050	77.7	128.2
1.75x15Dx3.50(4.00)C	1-3/4	3.50	26.25	4.00	5050	161.2	77.4
1.75x15Dx3.50(4.00)V1	1-3/4	3.50	26.25	4.00	4300	114.4	54.9
1.75x15Dx3.50(4.00)V2	1-3/4	3.50	26.25	4.00	4280	87.0	41.8
1.00x15Dx2.50(2.50)G5*	1	2.50	15.0	2.50	2650	32.44**	53.5
1.00x15Dx2.50(2.50)G10*	1	2.50	15.0	2.50	3900	49.47**	81.6
1.00x15Dx2.50(2.50)G15*	1	2.50	15.0	2.50	2810	37.94**	62.6

Mean Stress Area - A_m = 0.606 in.^{2} (1 in. diameter)
A_m = 2.082 in.^{2} (1-3/4 in. diameter)

*The number after the (G) indicates the centerline spacing in inches of the two bolts in the group.
**Load per bolt.

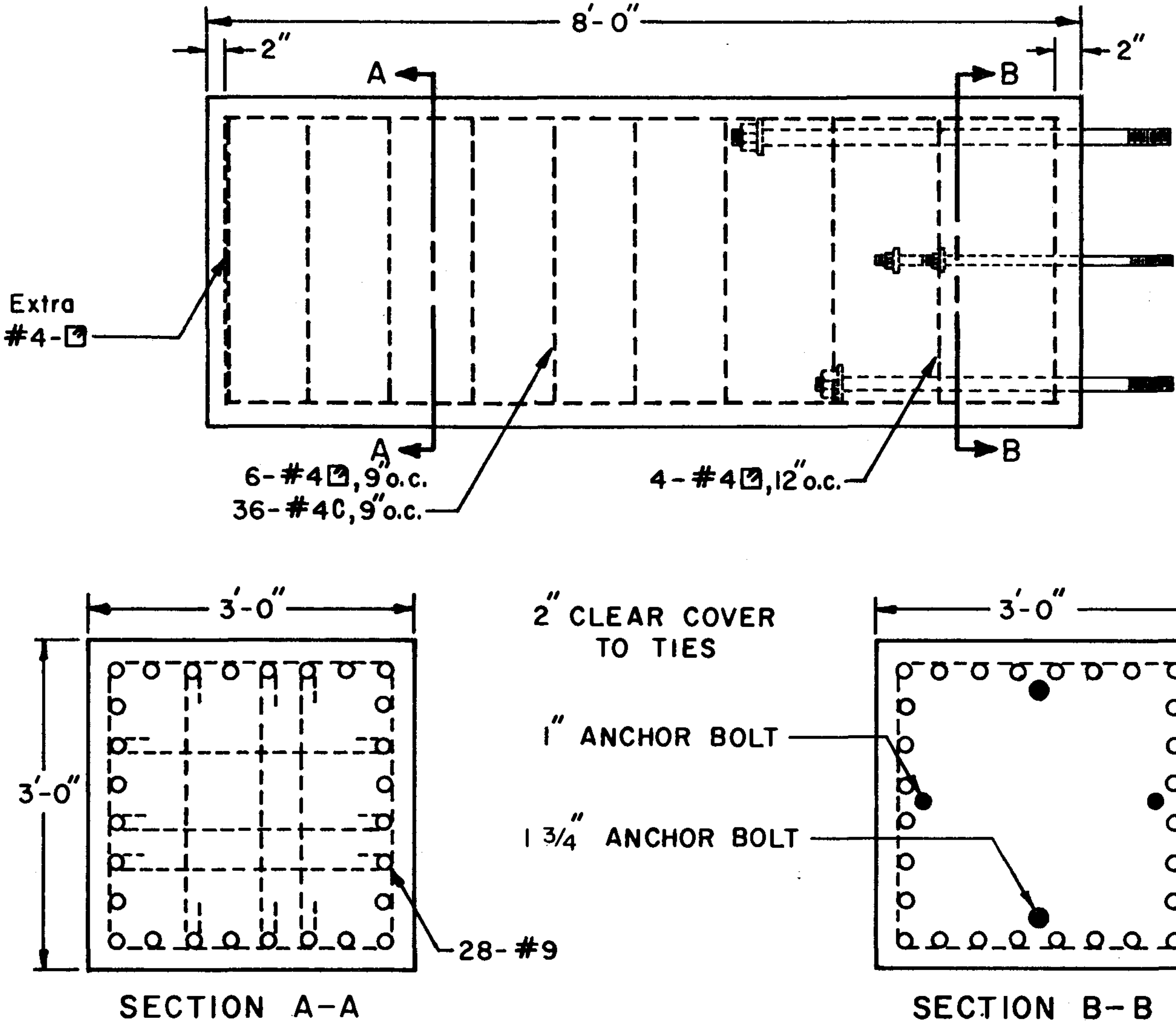


Fig. 2.1 Typical specimen geometry

The specimens were more heavily reinforced than standard practice would normally dictate. However, in order to isolate the anchor bolt failure mechanism, it was felt that the heavy reinforcement was necessary to prevent a shear and/or a flexural failure of the specimen from developing. The main reinforcement was held constant in all four sides of the specimen and was determined from the flexural requirements of the specimen with a 1-3/4 in. bolt under its maximum expected load. The transverse reinforcement in the region of the anchor bolts was similar to that found in typical drilled shaft footings.

It was important to try to ensure that damage from a given test did not influence subsequent tests on the same specimen. It was found that the 1 in. bolts rarely caused extensive damage to the specimen as a whole, and that it was possible to combine the 1-3/4 in. diameter bolts in the five specimens in such a manner that the first 1-3/4 in. bolt to be tested in each specimen did not damage the specimen severely, or at least left the anchorage region of the remaining bolt undamaged. Therefore, by testing both 1 in. bolts in a given specimen first, and then testing the 1-3/4 in. bolts in a specific order, it was usually possible to avoid damage to the specimen that interfered with subsequent tests on the specimen.

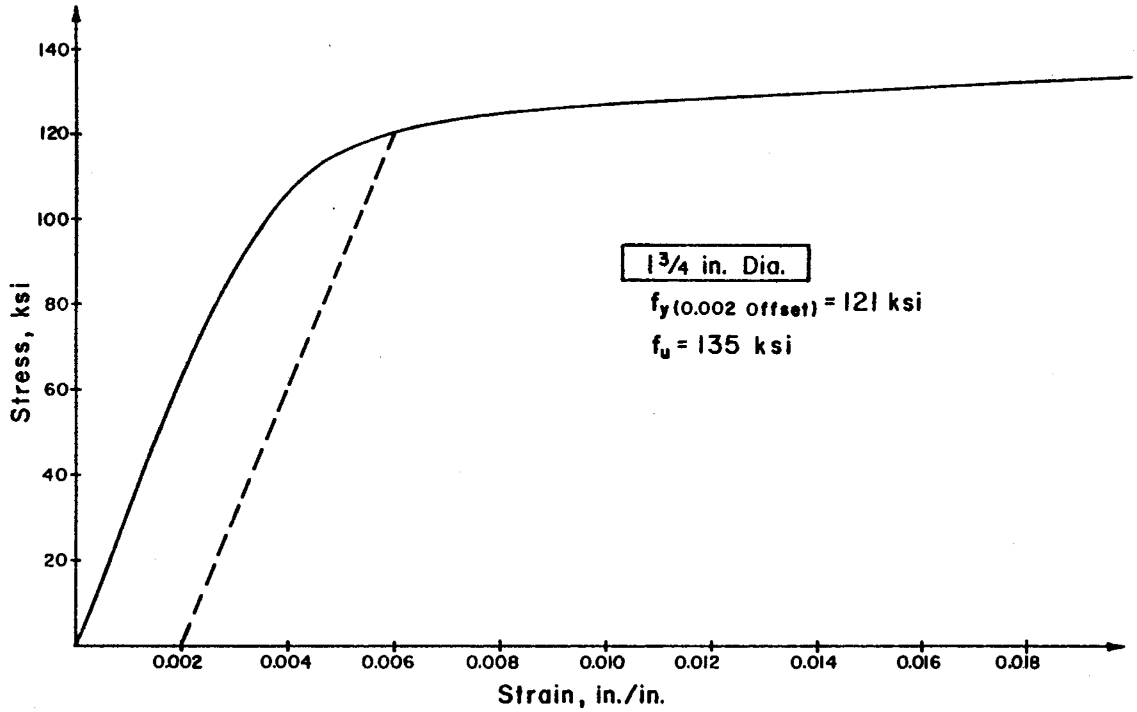
2.1.2 Materials. All specimens were cast with commercially obtained ready-mix concrete, using Type I cement and Colorado River sand and gravel. The maximum aggregate size was 1 in. Ideally, the water-to-cement ratio was 0.55. An air-entraining agent, Septair, was added at the rate of 2.5 oz/yd^3 and a set-retarding agent, Airsene L, was added at the rate of 30 oz/yd^3. Due to variations in quality control, the compressive strength ranged from 2640 psi to 5500 psi and the slump ranged from 4 in. to 10 in. for the nine specimens. Concrete strengths are listed in Table 2.1.

The reinforcing steel was Grade 60. The #9 and #4 bars were fabricated commercially to reduce the amount of time required for the construction of specimens in the laboratory.

The anchor bolts used in the test program were fabricated from bar stock which was cut to desired lengths in the laboratory and threaded at a local machine shop. The bar stock was AISI Grade 4140 steel, heat-treated to meet the requirements of ASTM Specification A193, Grade B7, which is commonly used to specify material for high-strength anchor bolts.[3] ASTM A193, Grade B7, provides a minimum tensile strength of 125 ksi, a minimum yield strength of 105 ksi, and a minimum elongation in 2 in. of 16 percent.[8] Figure 2.2 shows the stress-strain curves for the 1 in. diameter and the 1-3/4 in. diameter material used in the test program.

The end anchorage for each bolt consisted of a 1/2 in. thick washer and an ASTM Specification A194, Grade 2H nut, which is commonly used for many high-strength bolting applications.[9] Previous research[1] showed that a single standard-diameter washer was not fully effective in bearing; the single washers bent backwards around the nuts and bearing stresses calculated over the full area of the washer were significantly lower than bearing stresses encountered in tests on similar bolts with end anchorages consisting of a nut alone. It was felt, however, that a washer, i.e., an increase in bearing area of the anchorage, would have a beneficial effect on the strength and behavior of an anchor bolt provided the washer could be made fully effective in bearing. A nut and a 1/2 in. thick, standard-diameter washer is often specified for the anchorage device of an anchor bolt installation.

To facilitate fabrication of the anchor bolt assembly for the test program several standard-diameter, standard-thickness, plain hardened steel washers were welded together to form a single washer approximately 1/2 in. thick which was used as the end anchorage washer. All further discussion concerning "1/2 in. thick, standard-diameter washer" refers to the 1/2 in. thick washers with standard inside and outside diameters used in this test program. Where nonstandard size washers were required, they were fabricated from 1/2 in. steel plate machined to the required diameter.



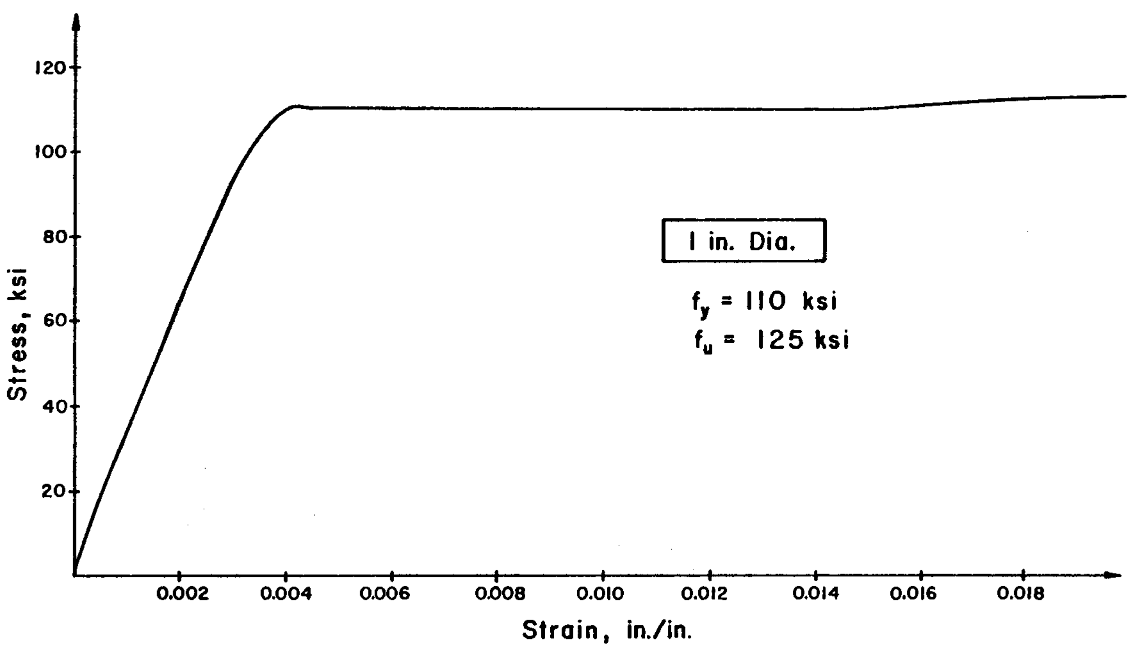


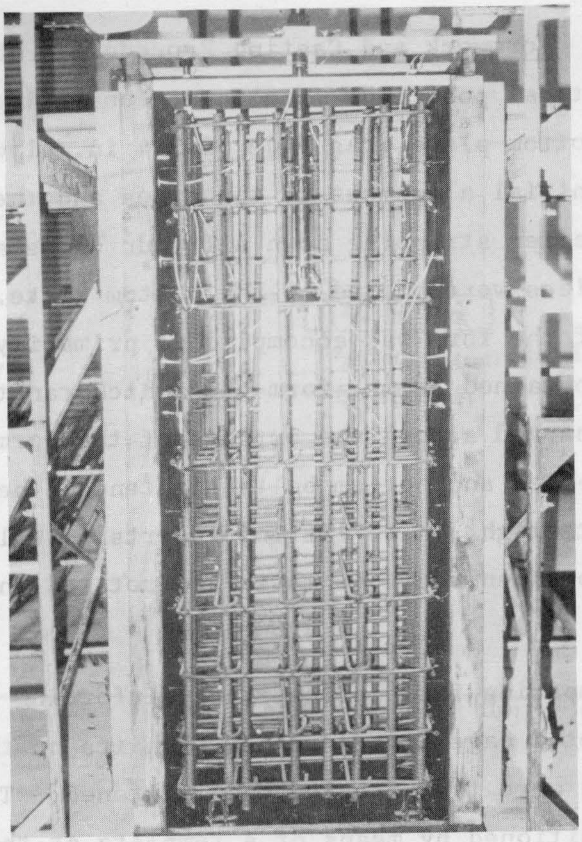
Fig. 2.2 Typical stress-strain curves for the 1 in. and the 1-3/4 in. bolt material

2.1.3 Formwork and Casting Procedure. The specimens were cast in a vertical position. The form, consisting of four separate sides and a bottom plate, was made of 3/4 in. plywood braced with 2x4 studs. Initial alignment of the sides and the bottom was provided by 3/4 in. chamfer stripping down all four sides and along the bottom edges. The sides were bolted to the bottom plate. Final alignment and sealing of the form was accomplished primarily by tightening the commercially obtained 36 in. form-ties which ran through the form in both directions and across the outside of the form along two sides. With all form-ties and bottom bolts tightened, the form was rigid and reasonably watertight. Two lifting inserts were located in each side so that the specimen could be moved and rotated in a horizontal position.

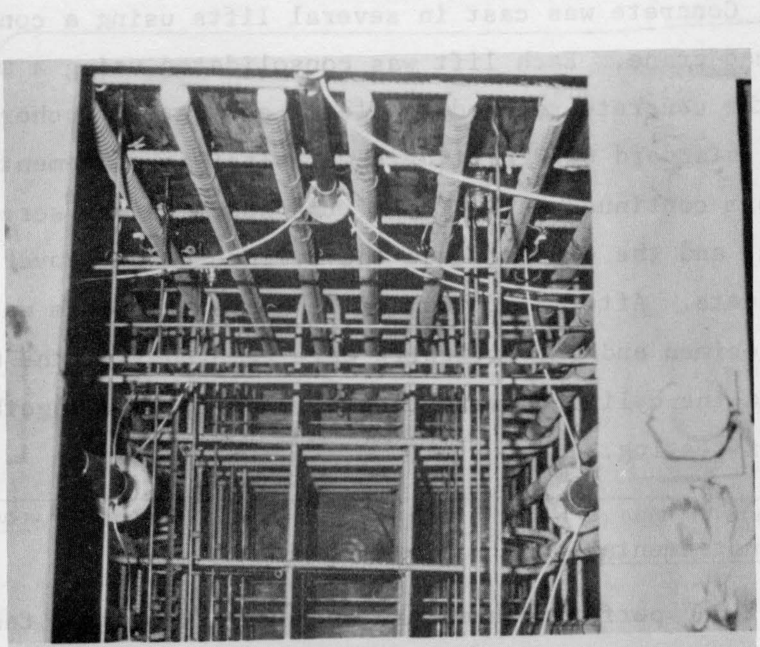
In preparing for casting, the reinforcing steel was pretied, and the completed cage placed on heavy chairs on the bottom plate. The four sides were then erected and tightened. The four instrumented bolts were positioned by means of a template at the top of the form. Figure 2.3 shows the anchor bolts and the cage in position in the form. Concrete was cast in several lifts using a concrete bucket and overhead crane. Each lift was consolidated using a mechanical vibrator. When the concrete reached the level of the end anchorages of the anchor bolts, standard 6x12 cylinders were cast as placement of concrete in the form continued. The top of the specimen was screeded and troweled smooth, and the specimen and the cylinders were covered with polyethylene sheets. After a minimum of 24 hours, the form was stripped from the specimen and the cylinders were removed from the molds; the specimen and the cylinders were then allowed to cure together until the time of testing.

2.2 Instrumentation

The performance of the anchor bolts during testing was measured by means of strain gages and slip wires. Figure 2.4 shows typical locations of the instrumentation on the anchor bolts.



(a) Side view



(b) Top View

Fig. 2.3 Form prior to casting

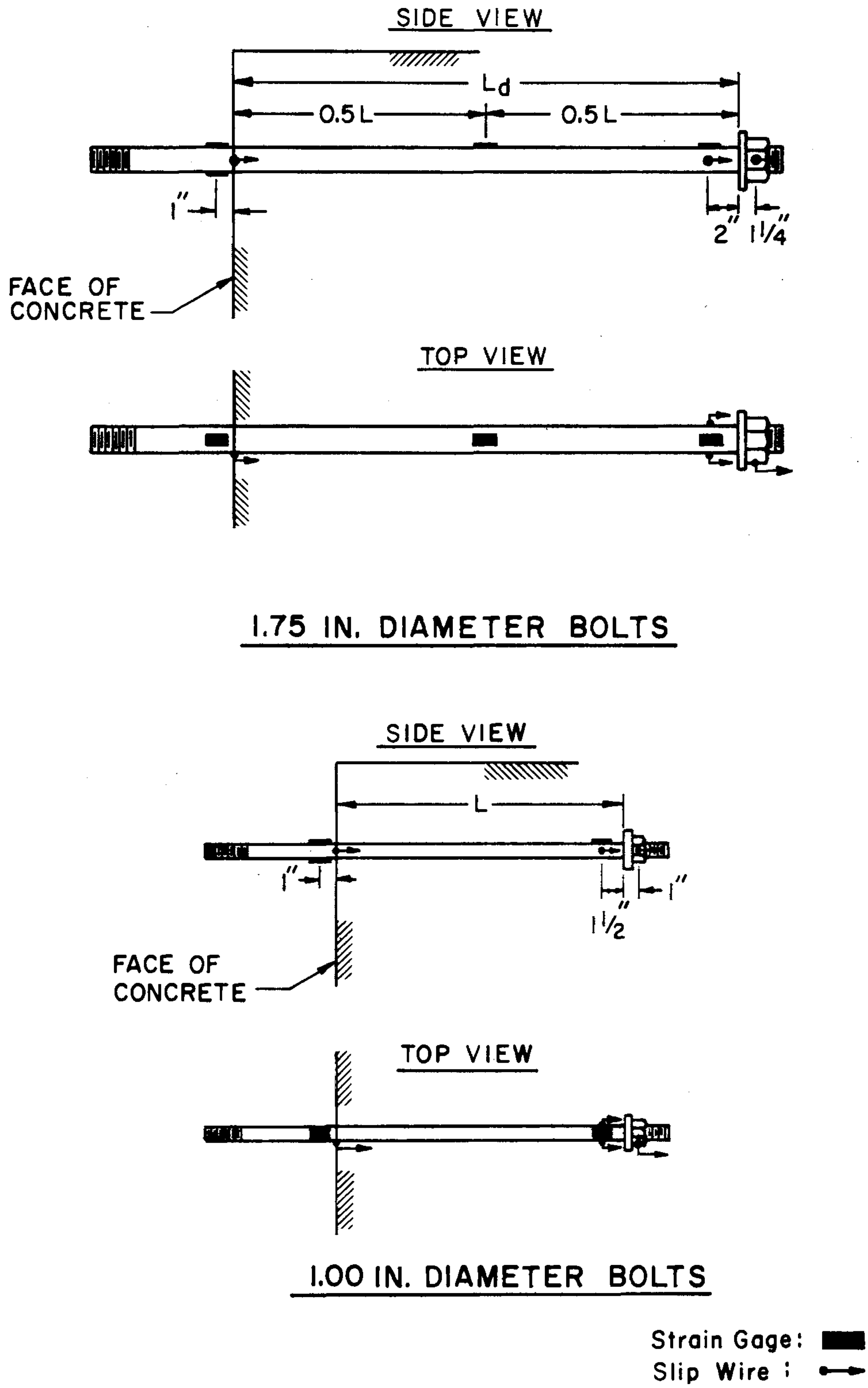


Fig. 2.4 Location of instrumentation

2.2.1 Strain Gages. Paper-backed electrical-resistance strain gages with a gage length of 0.64 in. were used to measure steel strains. The gages were attached with an epoxy adhesive and allowed to cure for 24 hours. After attaching lead wires the strain gage and lead wire connection was waterproofed by applying a polymer rubber pad which was then coated with a silicone rubber sealer. The pad and sealer covered an area typically 3/4 in. wide and 1-1/2 in. long.

As shown in Fig. 2.4, all bolts had a strain gage in front of the washer to measure bolt stress in the anchorage region and a pair of strain gages opposite one another on a vertical axis outside the concrete surface on the protruding bolt to measure stress in the anchor bolt at the face of the concrete. In addition, several of the 1-3/4 in. bolts had a strain gage positioned in the middle of the embedment length. The midbolt gage was not used on the 1 in. bolts to preserve as much original bar surface as possible.

2.2.2 Slip Wires. Slip of the anchor bolt relative to the concrete was measured using a procedure originally developed by Minor.[7]

A 0.059 in. diameter piano wire was attached to the anchor bolt at selected locations by making a short 90° bend at the end of the wire and inserting it into a hole of equal diameter drilled in the anchor bolt. The wire was oriented parallel to the bolt axis in the expected direction of slip. As shown in Fig. 2.4, the slip was typically measured at three points along the bolt. After the wire was placed in the bolt, a plastic tube was placed over the entire length of the wire to prevent bonding and to allow free movement of the piano wire. Figure 2.5 shows details of the instrumentation at the anchorage region. The plastic tube was sealed at the bolt end to prevent cement from entering the tube. The amount of sealer was small and the loss of bond surface area was kept to a minimum.

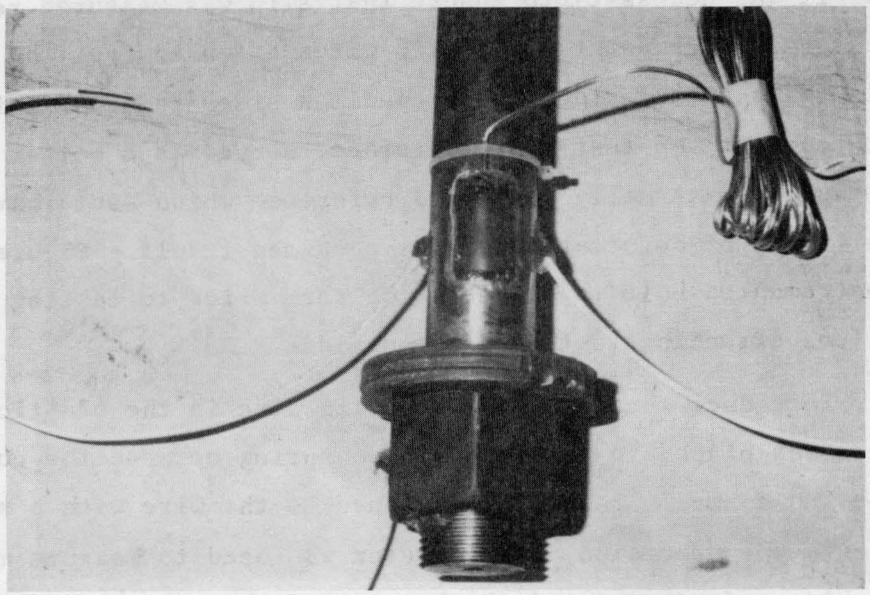


Fig. 2.5 Instrumentation at anchorage

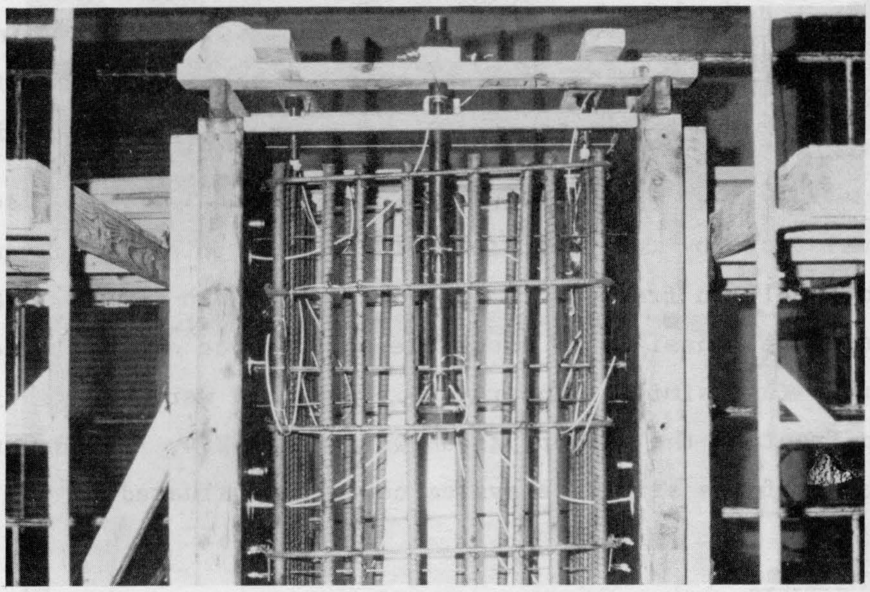


Fig. 2.6 Anchor bolts in form prior to casting

It was necessary to ensure that slip was measured relative to a stable reference point. The slip wires extended from the anchor bolt to the specimen sides. The specimen sides remained fairly undamaged throughout the test and, therefore, served as a better reference point than an externally supported reference which would have shown the influence of deformation of the specimen itself. Figure 2.6 shows the instrumented bolt in place in the form prior to casting with the slip wires extending to the specimen sides.

To reduce the wobble of the slip wire in the plastic tube, the wire was placed in tension using a spring between the concrete surface and a small brass plug fastened to the wire with a set-screw. A plunger-type precision potentiometer was used to measure movement of the wire and was mounted on the side of the specimen, as shown in Fig. 2.7. The plunger of the potentiometer rested against the brass plug at the end of the slip wire. All slip wires were oriented such that they were pulled, rather than pushed, as the anchor bolt slipped forward. The resulting change in resistance in the potentiometer was measured by a digital voltmeter. A constant voltage was maintained across the potentiometers which allowed the changes in resistance to be converted into deformations. Slip was measured to 0.001 in.

In addition to the slip wire, loaded end slip was also measured in several tests by two potentiometers mounted on a yoke attached to the anchor bolt in front of the specimen, as shown in Fig. 2.8. It was, therefore, possible to check the performance of the loaded end slip wire. In a similar manner, two slip wires were mounted in several tests in front of the washer, as shown in Fig. 2.4, in order that the consistency of the slip wire system could be evaluated.

2.3 Test Frame

In the previous anchor bolt study conducted at The University of Texas at Austin by Lee and Breen,[2] a method of loading was developed which was shown to realistically model the reaction conditions actually encountered in the field in typical drilled shafts. For the

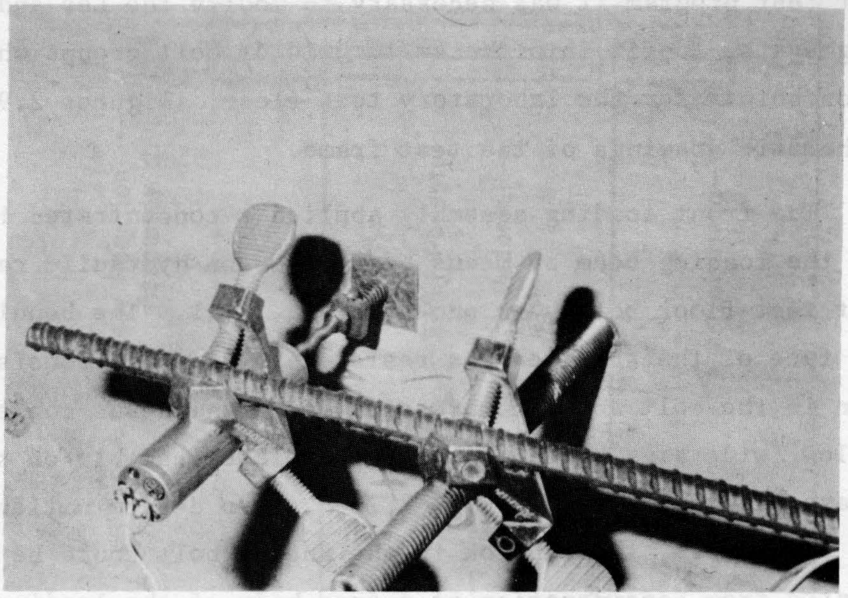


Fig. 2.7 Slip potentiometer mounting

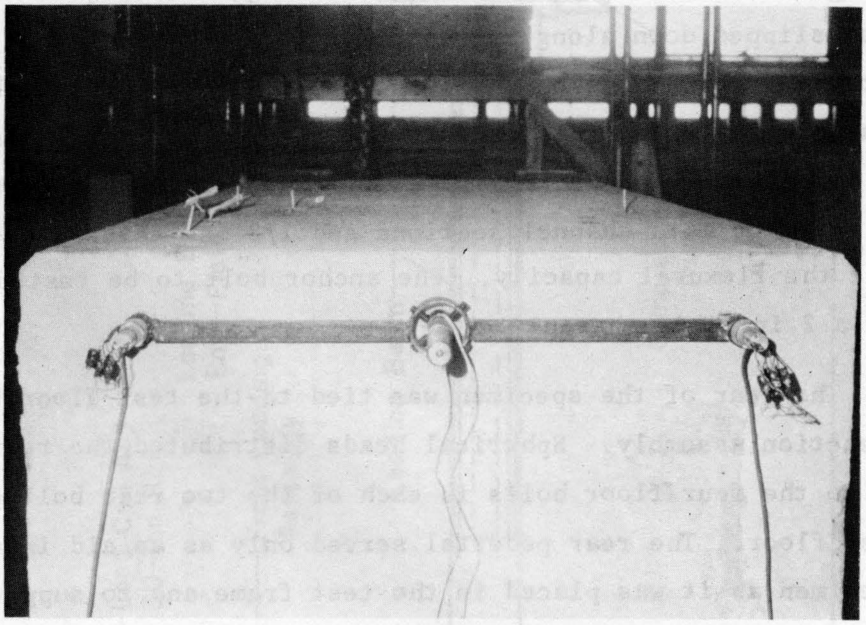


Fig. 2.8 Front yoke lead slip measuring device

present test program it was necessary to modify the Lee and Breen loading system to fit into the 4-ft. grid of bolt groups which provide reaction points for the laboratory test floor. Figures 2.9 and 2.10 are schematic drawings of the test frame.

The front loading assembly applied a concentrated load to the end of the loading beam by means of two 30-ton hydraulic rams reacting against test-floor bolts, as shown in Fig. 2.11. The bending moment at the face of the specimen was resisted by a couple consisting of the tension in the bolt and a compressive force which was concentrated over a shallow, wide area by the compression plate assembly on the loading beam baseplate, as shown in Fig. 2.10. After determination of the arm of the couple, the tension in the anchor bolt could be readily calculated from consideration of a free body of the loading beam (Fig. 2.12).

It was originally assumed that the layer of hydrostone would provide most of the shear transfer between the specimen face and the loading beam. It was observed during testing, however, that the loading beam slipped down along the specimen face during loading, imposing undesirable deformations on the anchor bolt. To prevent such deformation jacks were left in place under the compression plate assembly during the test, as shown in Fig. 2.10. The loading beam was a W16x64 built up with channel sections and 1/2 in. thick plate to increase the flexural capacity. The anchor bolt to be tested extended through a 2 in. thick baseplate.

The rear of the specimen was tied to the test floor by the rear reaction assembly. Spherical heads distributed the rear reaction evenly to the four floor bolts in each of the two rear bolt groups in the test floor. The rear pedestal served only as an aid in aligning the specimen as it was placed in the test frame and to support the specimen until testing.

Figure 2.13 shows the test frame as modified for the two lateral load tests. An intermediate reaction assembly (A) was placed

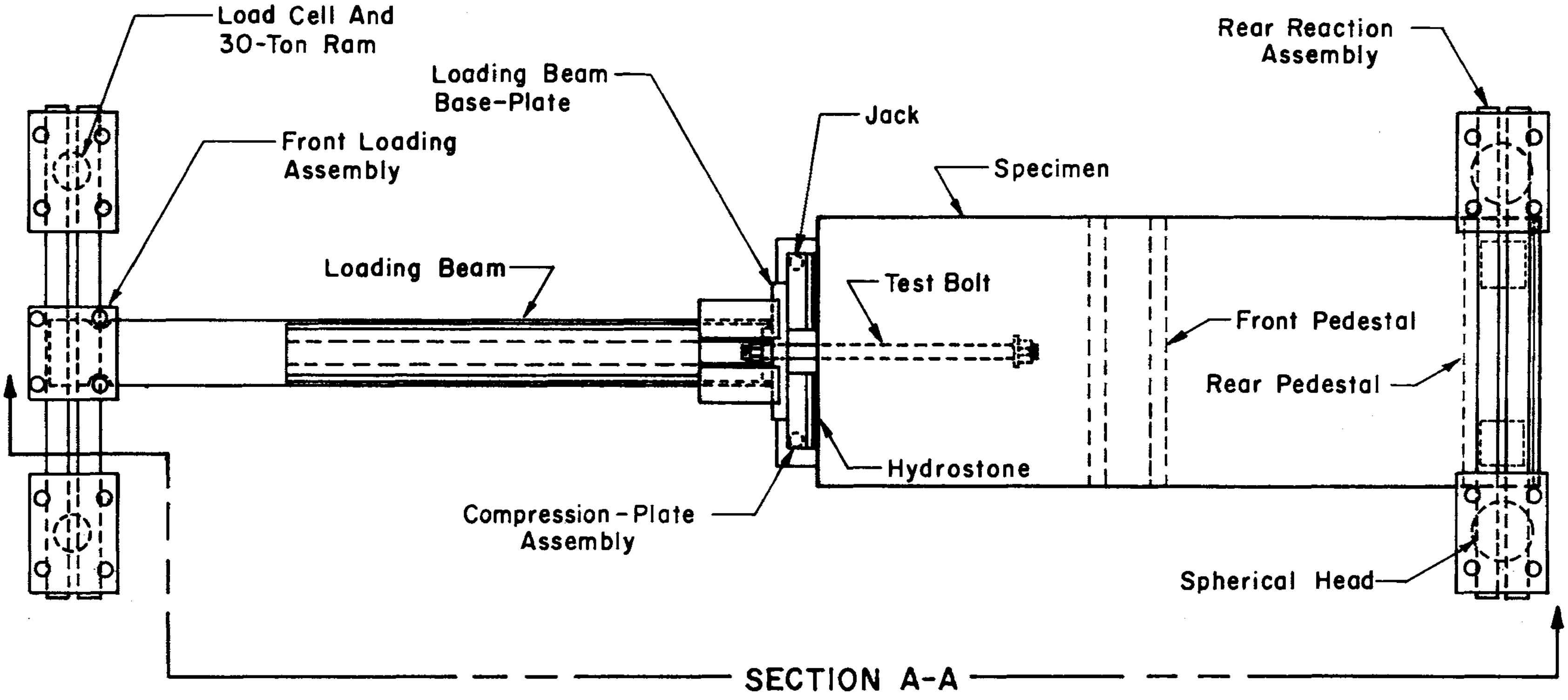


Fig. 2.9 Top view of the test frame

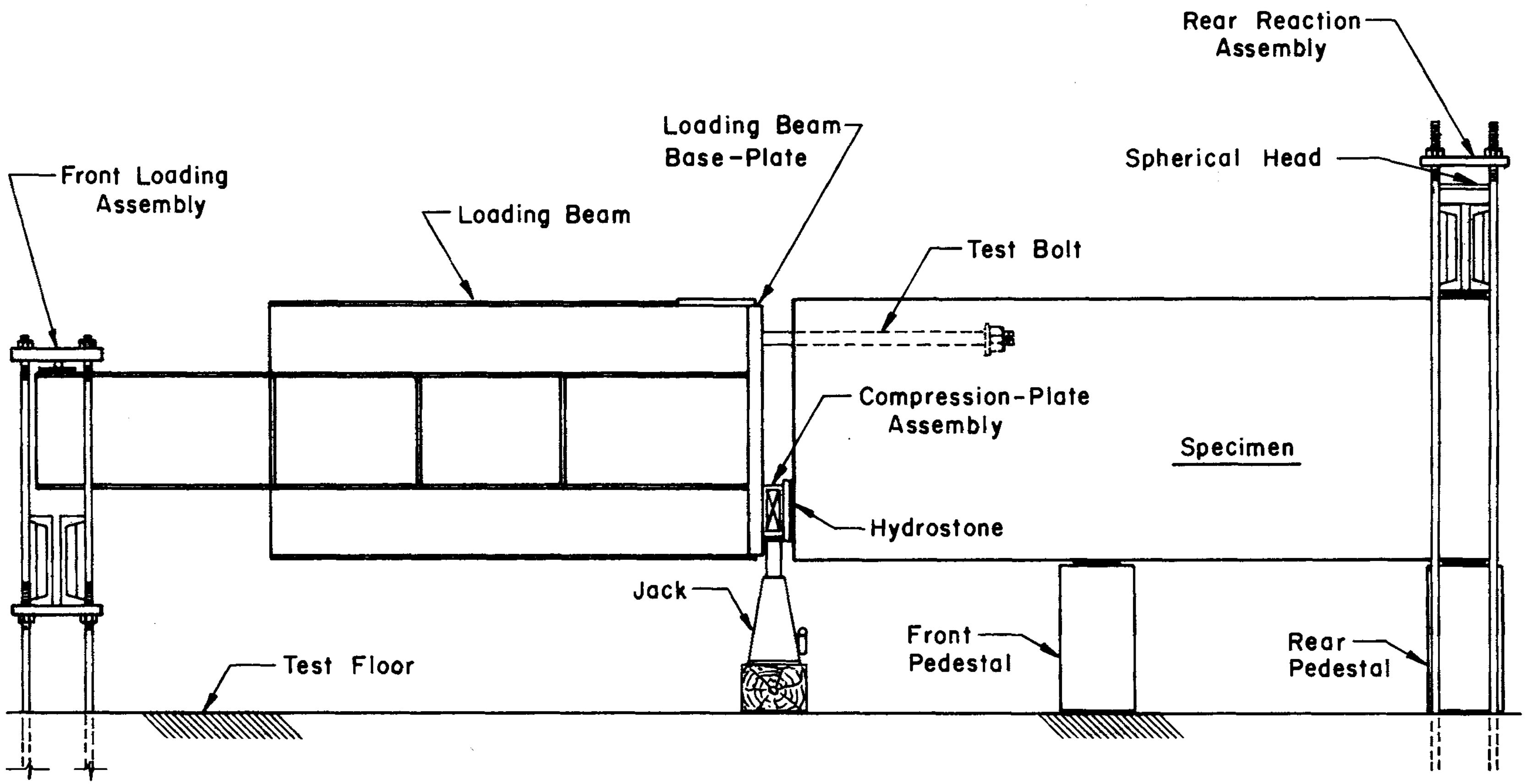


Fig. 2.10 Elevation of test frame--Section A-A

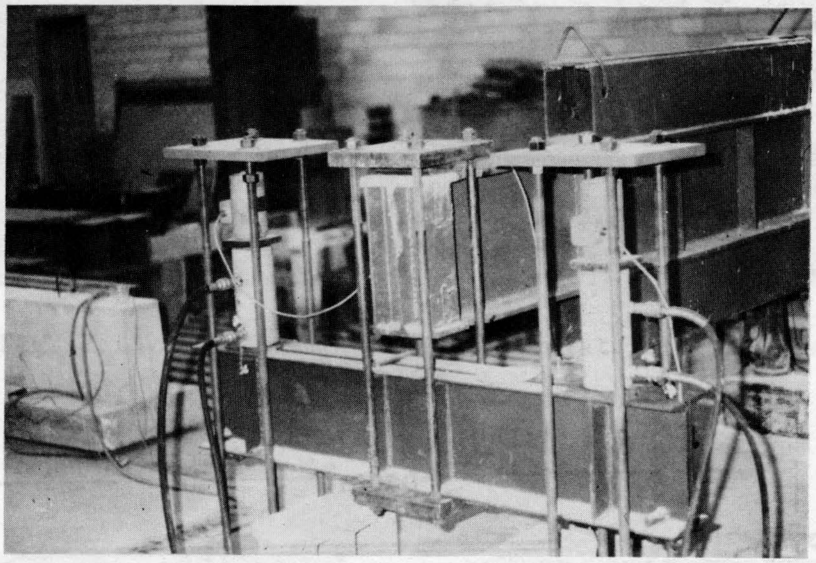
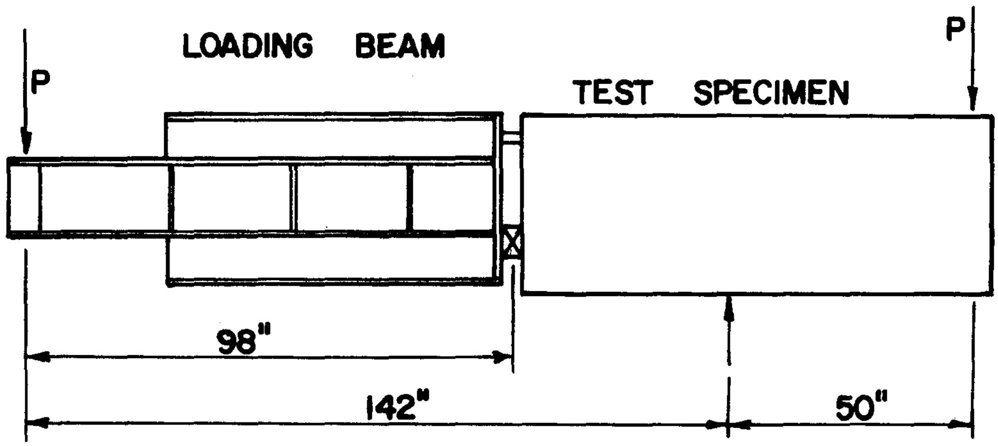
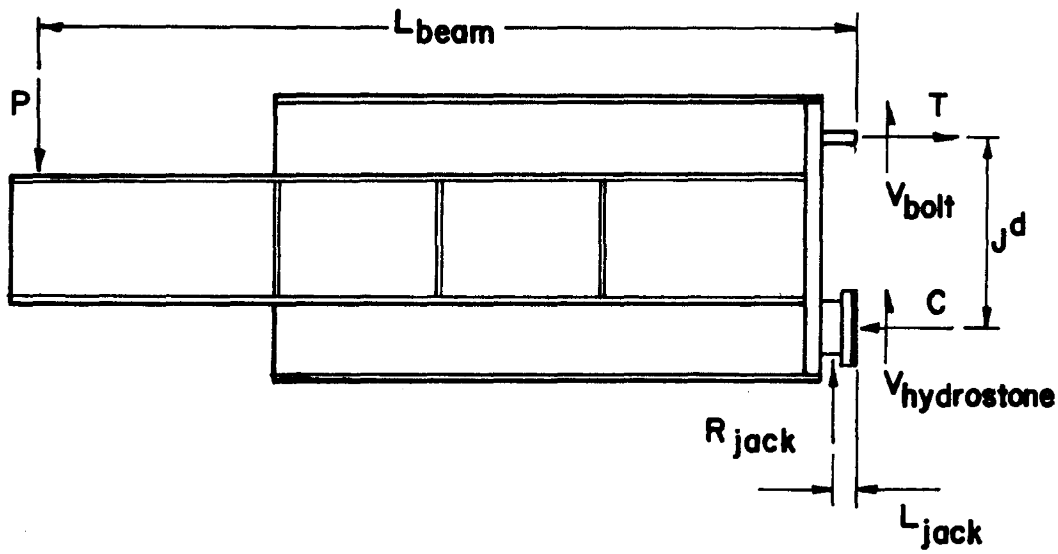


Fig. 2.11 Front loading assembly of the test frame



(a) SPECIMEN AND LOADING BEAM



(b) FORCES ON A LOADING BEAM

Fig. 2.12 Free body diagram of loading beam

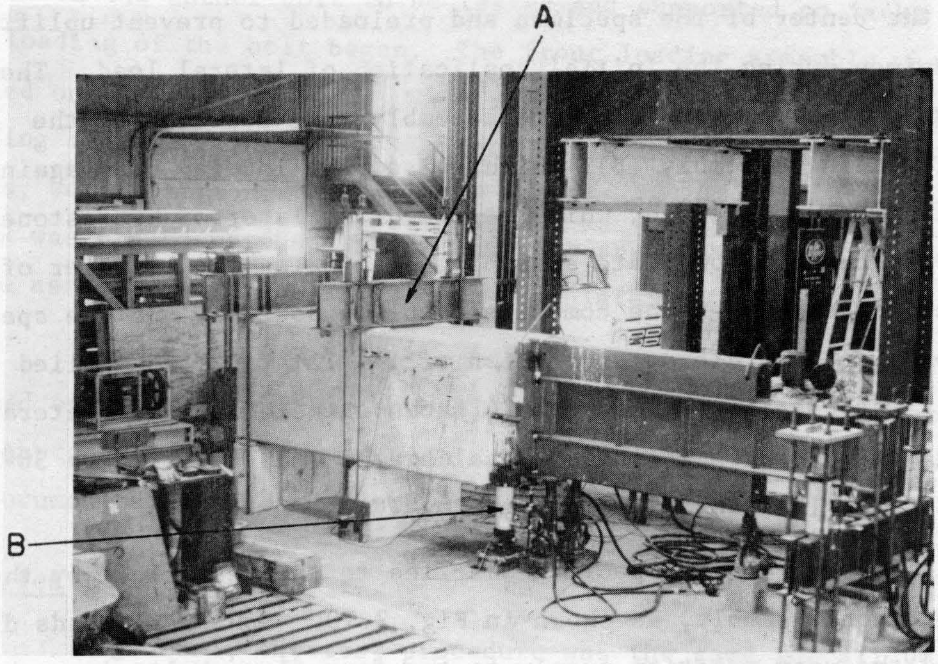


Fig. 2.13 Test frame modified for lateral load tests

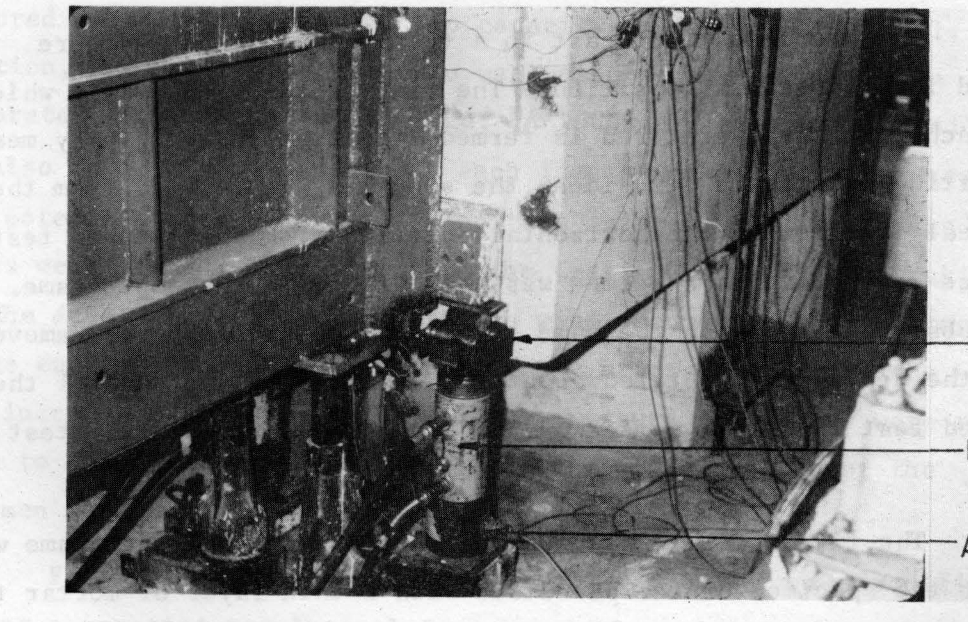


Fig. 2.14 Lateral loading assembly

across the center of the specimen and preloaded to prevent uplift of the specimen during the initial application of lateral load. The jacks under the compression plate assembly were replaced by the lateral loading assembly (B), which raised the loading beam against the bolt. A neoprene pad, which replaced the layer of hydrostone behind the compression plate assembly, eliminated the transfer of vertical force between the compression plate assembly and the specimen; therefore, an accurate determination of the lateral load applied to the bolt was possible. Figure 2.14 shows one side of the lateral loading assembly, consisting of a calibrated load cell (A), a 30-ton hydraulic ram (B), and a roller bearing (C).

The rear of the specimen was tied to the test floor by the rear reaction assembly, as shown in Fig. 2.10. Spherical heads distributed the rear reaction evenly to the four floor bolts in each of the two rear bolt groups in the test floor. The rear pedestal served only as an aid in aligning the specimen as it was placed in the test frame and to support the specimen until testing.

2.4 Preparation for Testing

The specimens were cast in a vertical position and were tested in a horizontal position. The side of the specimen in which the anchor bolt was centered is termed "the test surface". By means of lifting inserts in its sides, the specimen was lowered from the vertical position to the horizontal position with the desired test surface face up. The specimen was then placed in the test frame. For subsequent tests on the same specimen, the specimen was removed from the test frame, rotated about its longitudinal axis until the desired test surface was face up, and then reinserted in the test frame.

The first step in placing the specimen in the test frame was to set the specimen on the pedestals with a thin layer of mortar to ensure an even bearing surface between the specimen and the 6 in. x 36 in. bearing plate on the front pedestal. The rear reaction assembly was then erected on the specimen. Next, the loading beam was placed on the anchor bolt to be tested and supported on jacks until the loading of the bolt began. The front loading assembly was then placed on the loading beam. When all the components of the front loading assembly, including the load cells and the hydraulic rams and hoses, were positioned, a layer of hydrostone approximately 3/8 in. thick was placed between the face of the specimen and the compression plate assembly on the loading beam baseplate.

Immediately prior to the test the location of the washer was marked on the test surface and all instrumentation was connected to the appropriate measuring device. When necessary, pictures were taken to document damage to the specimen from previous tests.

2.5 Test Procedure

In general, the test procedure was the same for all bolts. Load increments of 2.0 kips for the 1-3/4 in. bolts and of 1.0 kips for the 1 in. bolts were applied at each load stage by the hydraulic rams operated by an electric pump. The resulting bolt tension increment was on the order of about 8.0 kips for the 1-3/4 in. bolts and about 4.0 kips for the 1 in. bolts. The level of the load was measured at each ram by a 100 kip capacity calibrated load cell. In addition, hydraulic hose pressure was measured at the pump by a calibrated 10000 psi pressure gage. A 10000 psi pressure transducer was also used in most tests. At each load stage all strain gages and slip potentiometers were read. The test surface was examined and cracks were marked. Key points in the test, such as first cracking, and the development of crack patterns were documented with photographs. At the end of each load stage immediately prior to applying the next load increment, the load cells and the pressure transducer were read again to allow an evaluation of the degree of relaxation of the specimen and the loading system during the load stage.

For the lateral load tests, the test procedure was modified so that a constant net lateral force could be maintained on the bolt. First, the desired level of lateral load was applied to the bolt with the hydraulic rams in the lateral loading assembly. Then the bolt tension was applied in the usual manner. At each load stage, after the tension increment was applied by the hydraulic rams in the front loading assembly, the lateral load was adjusted to bring the net lateral force on the bolt back to the desired level. Data were then recorded in the usual manner.

A test was terminated when the anchor bolt would not carry additional load or would only maintain a reduced level of load with continued pumping of the rams. Normally, loading was then halted to avoid causing unnecessary damage to the specimen which might interfere with subsequent tests on the same specimen. For several of the 1 in. bolts which reached yield, loading was continued until necking was observed.

CHAPTER 3

SPECIMENS AND TEST EQUIPMENT--MODEL TESTS

3.1 General

In an earlier study by Lee and Breen,[4] it was found that reduced scale models could be effectively used in anchorage studies in combination with a limited number of full-size tests. To evaluate the strength and behavior of high strength (yield about 110 ksi) anchor bolts, the use of reduced models together with a number of full-size specimens make possible the study of wider ranges of variables at a lower cost. Model tests were run only where the end bearing was the prime factor influencing the strength of the installation. The study by Lee and Breen[4] showed that if bearing was the prime factor, good correlation between full-scale and model studies was obtained.

3.2 Development of Specimen

Initially, the model study was planned using a 1/3.5 scale model of the prototype specimens. It was intended to use 1/4 and 1/2 in. diameter anchor bolts to model approximately the 1 and 1-3/4 in. diameter bolts used in the prototype test specimens.

In the prototype, the anchor bolts consisted of plain round stock threaded at both ends. The anchor end which was embedded in concrete had a mechanical anchor consisting of a nut and washers. The same arrangement was initially used in the model bolts. However, the small scale bolts all failed by a fracture in the threaded portion of the bolts which extended outside the concrete (or the loaded end of the bolts). To prevent fracture in the threads in subsequent tests, a chuck was used in place of the nut and washers at the loaded end. The use of the chuck called for longer exposed bolt lengths to accommodate the chuck length in all subsequent tests. This arrangement worked quite well with the 1/2 in. model bolts. However, the 1/4 in. bolts still fractured in the threads embedded in the concrete at the anchor end. Because of these difficulties, only the 1/2 in. bolts were used in subsequent tests and will be reported here.

3.3 Specimen Details

Figure 3.1 shows a typical specimen with model bolts. The specimen had a square cross section 9 × 9 in. and a length of 36 in. A tabulation of the specimen details is provided in Table 3.1. All specimens were cast vertically to simulate actual field practice where anchor bolts are used in the top of light standard piers, bent caps, bridge abutments, etc.

To designate each bolt, the following notation was used; e.g., 1/2 × 15D × 1.25(1.375). The first number represents the diameter of the anchor bolt in inches, followed by the embedment length in terms of number of bar diameter, the clear cover in inches, and the washer diameter in inches. The designation above indicates a 1/2 in. diameter anchor bolt with 15 diameter embedment length, a clear cover of 1.25 in., and standard 1-3/8 in. diameter washers.

In the prototype test, failure occurred locally, and since only the one face was damaged the specimen was rotated and bolts cast on other faces could be tested in the same way. To minimize the number of prototype specimens cast, four bolts (one at the center of each face) were placed in each specimen. Since specimen size was not critical in the models, bolts were placed only in two opposite faces unless it was felt that a particular combination of parameters would lead to failure which might damage the entire specimen and render the remaining bolt useless. (Bolts embedded with a large clear cover usually caused severe cracking and splitting of the concrete.) To allow the testing of both bolts, the procedure followed was to initially test the bolt with the smaller concrete cover.

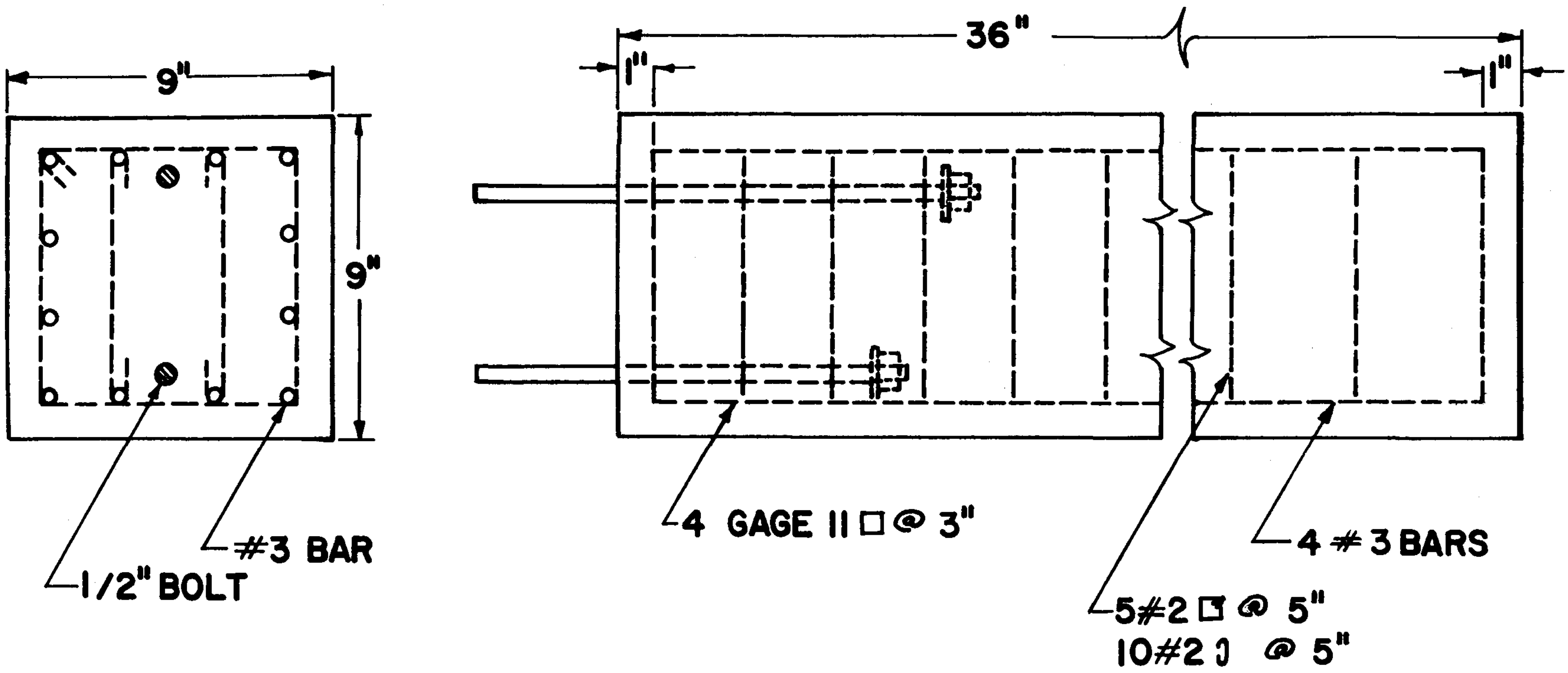


Fig. 3.1 Typical specimen details

TABLE 3.1 SUMMARY OF SCALE MODEL TESTS

Bolt	Bolt Diameter (in.)	Clear Cover (in.)	Embedment Length (in.)	Washer Diameter (in.)	f'_c (psi)	Ultimate Load (kips)	Ultimate f_{sm} (ksi)
0.50x15Dx0.50(1.375)	1/2	0.5	7.5	1.375	3460	11.50	81.0
0.50x15Dx0.75(1.375)	1/2	0.75	7.5	1.375	3460	16.00	112.7
0.50x15Dx0.75(1.375)L	1/2	0.75	7.5	1.375	3260	16.82	118.5
0.50x15Dx1.00(1.375)	1/2	1.00	7.5	1.375	5025	16.86	118.7
0.50x15Dx1.00(1.375)	1/2	1.00	7.5	1.375	5450	19.00	133.8
0.50x15Dx1.00(1.375)	1/2	1.00	7.5	1.375	3090	13.00	91.5
0.50x15Dx1.00(1.06)	1/2	1.00	7.5	1.06	3260	15.33	108.0
0.50x15Dx1.00(1.75)	1/2	1.00	7.5	1.75	3260	13.00	91.5
0.50x15Dx1.25(1.375)	1/2	1.25	7.5	1.375	5960	21.41	150.8
0.50x15Dx1.25(1.375)	1/2	1.25	7.5	1.375	3660	15.05	106.0
0.50x15Dx1.25(1.06)	1/2	1.25	7.5	1.06	3450	14.20	100.0
0.50x15Dx1.25(1.75)	1/2	1.25	7.5	1.75	3260	17.84	125.6
0.50x15Dx1.50(1.375)	1/2	1.50	7.5	1.375	3950	15.48	109.0
0.50x15Dx1.75(1.375)	1/2	1.75	7.5	1.375	3400	18.00	126.8
0.50x15Dx1.75(1.06)	1/2	1.75	7.5	1.06	3090	10.48	73.8
0.50x15Dx2.00(1.375)	1/2	2.00	7.5	1.375	3090	18.75	132.0
0.50x15Dx2.25(1.06)	1/2	2.25	7.5	1.06	3500	21.00	147.9
0.50x20Dx0.75(1.375)	1/2	0.75	10.0	1.375	2970	14.95	105.3
0.50x20Dx1.00(1.375)	1/2	1.00	10.0	1.375	5000	16.50	116.2
0.50x20Dx1.00(1.375)	1/2	1.00	10.0	1.375	3090	13.96	98.3
0.50x20Dx1.00(1.375)	1/2	1.00	10.0	1.375	5655	21.50	151.4
0.50x20Dx1.25(1.375)	1/2	1.25	10.0	1.375	3460	17.50	123.2
0.50x20Dx1.25(1.06)	1/2	1.25	10.0	1.06	3500	16.54	116.5
0.50x10Dx1.00(1.375)	1/2	1.00	5.0	1.375	4530	9.48	66.8
0.50x10Dx1.25(1.375)	1/2	1.25	5.0	1.375	4025	11.50	81.0
0.50x10Dx1.50(1.375)	1/2	1.50	5.0	1.375	5040	15.50	109.2
0.50x10Dx1.50(1.375)	1/2	1.50	5.0	1.375	3610	8.41	59.2
0.50x10Dx1.75(1.375)	1/2	1.75	5.0	1.375	3430	9.61	67.7
0.50x10Dx2.00(1.375)	1/2	2.00	5.0	1.375	3120	13.40	94.4

Mean Steel Area, $A_{sm} = 0.142$ in.^{2}

Gross Steel Area, $A_g = 0.196$ in.^{2}

3.4 Materials

Concrete. All specimens were cast with a job-mixed concrete. The first four specimens were cast using Type I portland cement. However, in subsequent tests, high early-strength cement (Type III) was used to permit testing specimens at an earlier age. Colorado River sand and gravel were used throughout. Maximum aggregate size was 3/8 in. which was chosen to approximate a 1/3.5 scale model of the 1 in. coarse aggregate used in the prototype. Typically, the water-cement ratio was around 0.7. An air-entraining agent (Septair) was added. Slumps ranged from 3 to 4 in. Most of the specimens had a compressive strength at testing of 3000 to 4000 psi; however, some specimens reached a strength of 6000 psi. Table 3.1 lists the concrete strength for the specimens.

Steel. The anchor bolts used were fabricated from heat-treated high strength alloy steel round stock with a yield strength of 110 ksi. Grade 4140, hot-rolled quenched and tempered round bar stock was used. The threaded sections were machined to conform to USS thread specifications (National Coarse). A typical stress-strain curve is shown in Fig. 3.2.

Heavy hex, Grade 2H nuts (13 threads per in.) conforming to ASTM A194-72 were used. Washers were plain hardened steel washers. Three different washer sizes were used as follows:

(1) 1-3/8 in. O.D. 0.094 in. thick
(2) 1-1/16 in. O.D. 0.078 in. thick
(3) 1-3/4 in. O.D. 0.094 in. thick

To closely model the thickness of the washer in the prototype, where three standard washers were welded together to become a 1/2 in. thick anchor plate, two washers were put together as a unit, resulting in a scale factor of about 3 instead of 3.5.

Reinforcement in the specimens consisted of #3 Grade 60 deformed bars, and transverse reinforcement was fabricated from 1/4 in. diameter plain bars and 10 gage steel wire.

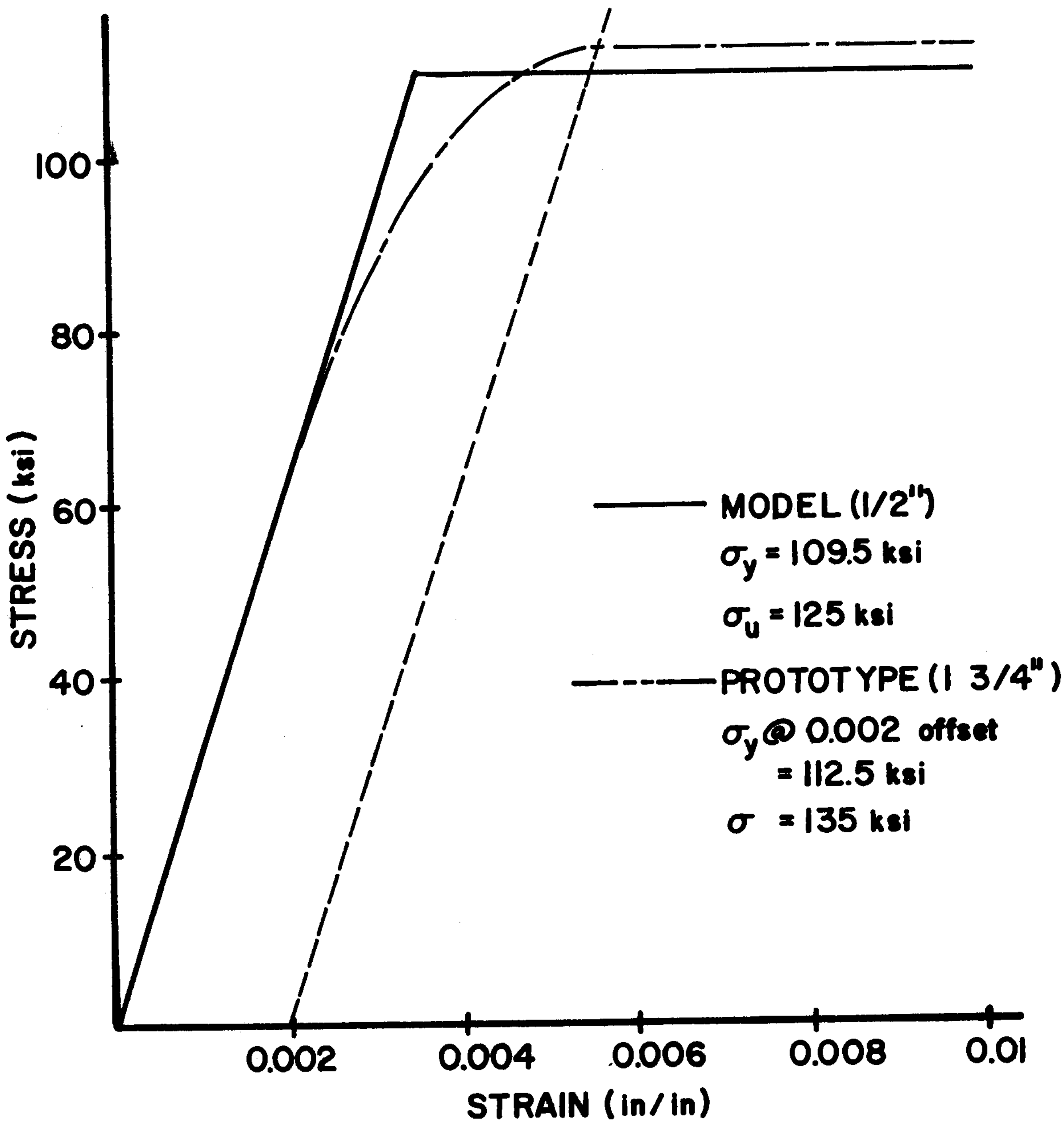


Fig. 3.2 Stress-strain curves for high strength anchor bolt

3.5 Fabrication of Specimen

The reinforcing cage and the steel form can be seen in Fig. 3.3. Steel wire (10 gage) was used where shear forces were lower and only minimum transverse reinforcement was required.

The form was fabricated from 10 gage steel sheets mounted with two angles at the edges. Before the cage was put inside the form, the form was oiled. The cage rested on a 1-1/2 in. chair at the bottom of the form and was held in place by 5/8 in. chairs on the four sides. Two coil inserts were then put into place for lifting purposes. Slip measuring instrumentation was placed on the bolts prior to their placement in the form. To hold the bolts in place, a special support was clamped to the two angles at the top on opposite sides of the form. The desired clear cover on the bolt could be

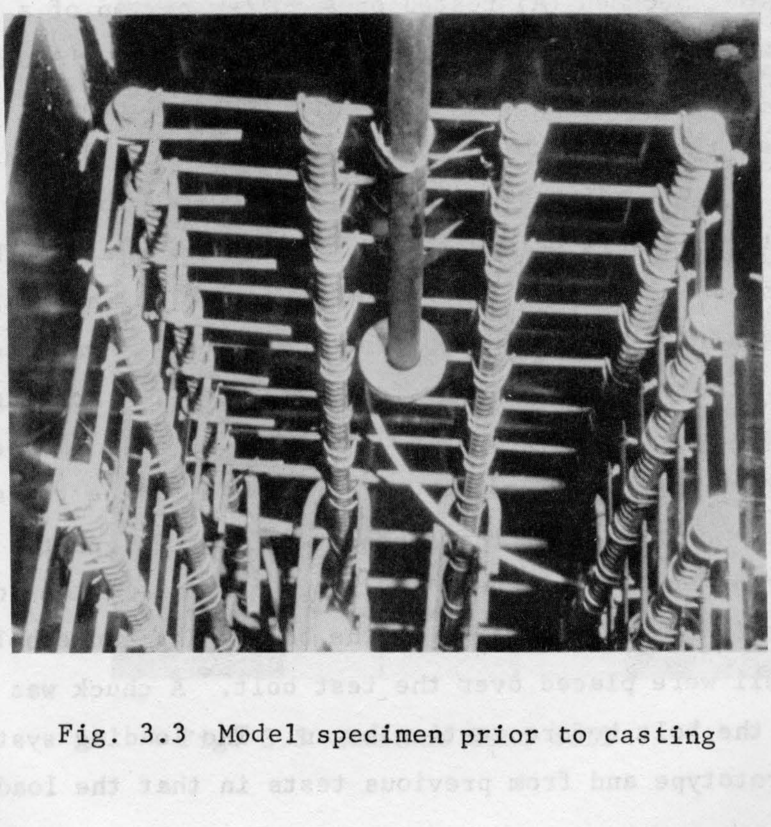


Fig. 3.3 Model specimen prior to casting

easily achieved by moving the support along the two angles. After the anchor bolt was lowered to the desired embedment length, it was locked in position using two plastic tie bands, one above and one below the support channel. With the bolts in position, the specimen was ready to be cast.

Specimens were cast vertically to approximate field situations. Concrete was placed in three lifts with each lift filling approximately one-third of the form and vibrated using an internal vibrator. Specimens were cast in pairs. Control cylinders were cast from the batch which was placed at the bolt (top) section of the specimens. Cylinders and specimens were usually left in the forms for two or three days and covered by a large plastic sheet before they were stripped and cured in the laboratory.

3.6 Loading System

The loading system and the test setup are shown in Figs. 3.4 and 3.5. The specimen (A) rested on a roller on top of a concrete pedestal, with the test bolt (B) at the top surface. Another roller was placed 2 in. from the end of the specimen and grouted to form a support for the square tubing (C), which provided a downward reaction at the end. The loading beam (D) was then lifted in place. It was made from two channels welded to a 1/2 in. steel plate. The channels were separated 4 in. apart to provide space for the hydraulic ram (E) and the load cell (F). A 20-ton centerhole ram (E) was used to apply the load. Its reaction was transferred through the loading beam to the floor slab by means of a 1 in. steel rod. Where the beam rested against the specimen and produced a compressive force to balance the tensile force on the bolt, the contact surface was grouted to provide uniform stress in the surface. Two long clamps held the loading beam in place until load was applied to the bolt. The centerhole ram and the load cell were placed over the test bolt. A chuck was finally slipped on the bolt before testing began. The loading system differed from the prototype and from previous tests in that the load was applied

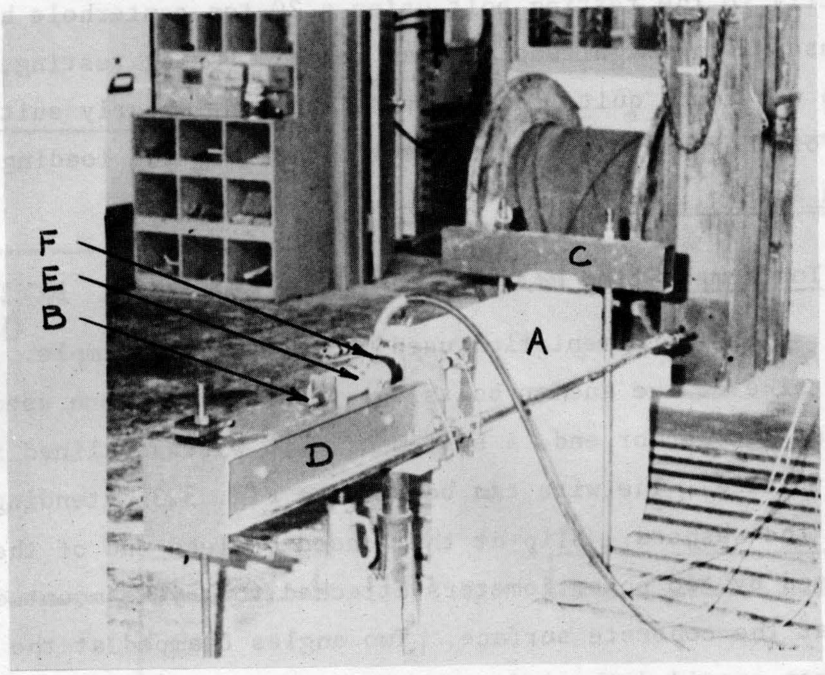


Fig. 3.4 Test setup

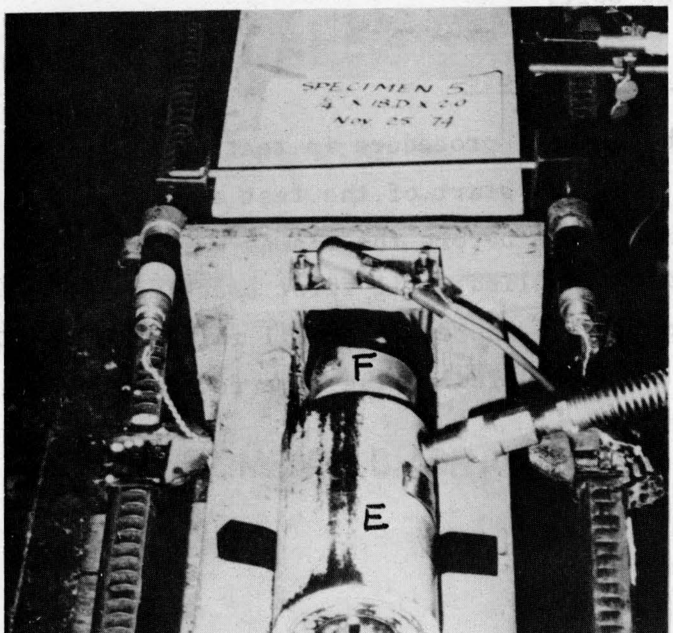


Fig. 3.5 Ram and load cell

directly to the testing bolt using a 20-ton centerhole hydraulic ram. Because of the magnitude of load reached during testing, the loading setup was found quite satisfactory and particularly suitable. A diagram of the specimen and forces generated by the loading system is shown in Fig. 3.6.

3.7 Instrumentation

The instrumentation used was relatively simple. Due to the small size of the anchor bolts, no strain gages were used. To measure slip at the anchor end, a system of slip wires outlined in Chapter 2 was utilized. The wire can be seen in Fig. 3.3 extending downward below the washers. Slip at the loaded or lead end of the bolt was measured by two potentiometers attached to a yoke mounted across the bolt at the concrete surface. Two angles clamped at the sides of the specimen provided the reference point (see Fig. 3.7). The average reading of the two front potentiometers was calculated to be the lead slip of the bolt.

3.8 Testing Procedure

The general procedure in testing almost all of the bolts was identical. At the start of the test a very small load was applied to tighten the system before the clamps were taken off. Load in 1 kip increments was applied until first cracking occurred. It was then applied in 1/2 kip increments until failure. An average test took about two hours, including the time for marking and photographing the cracks.

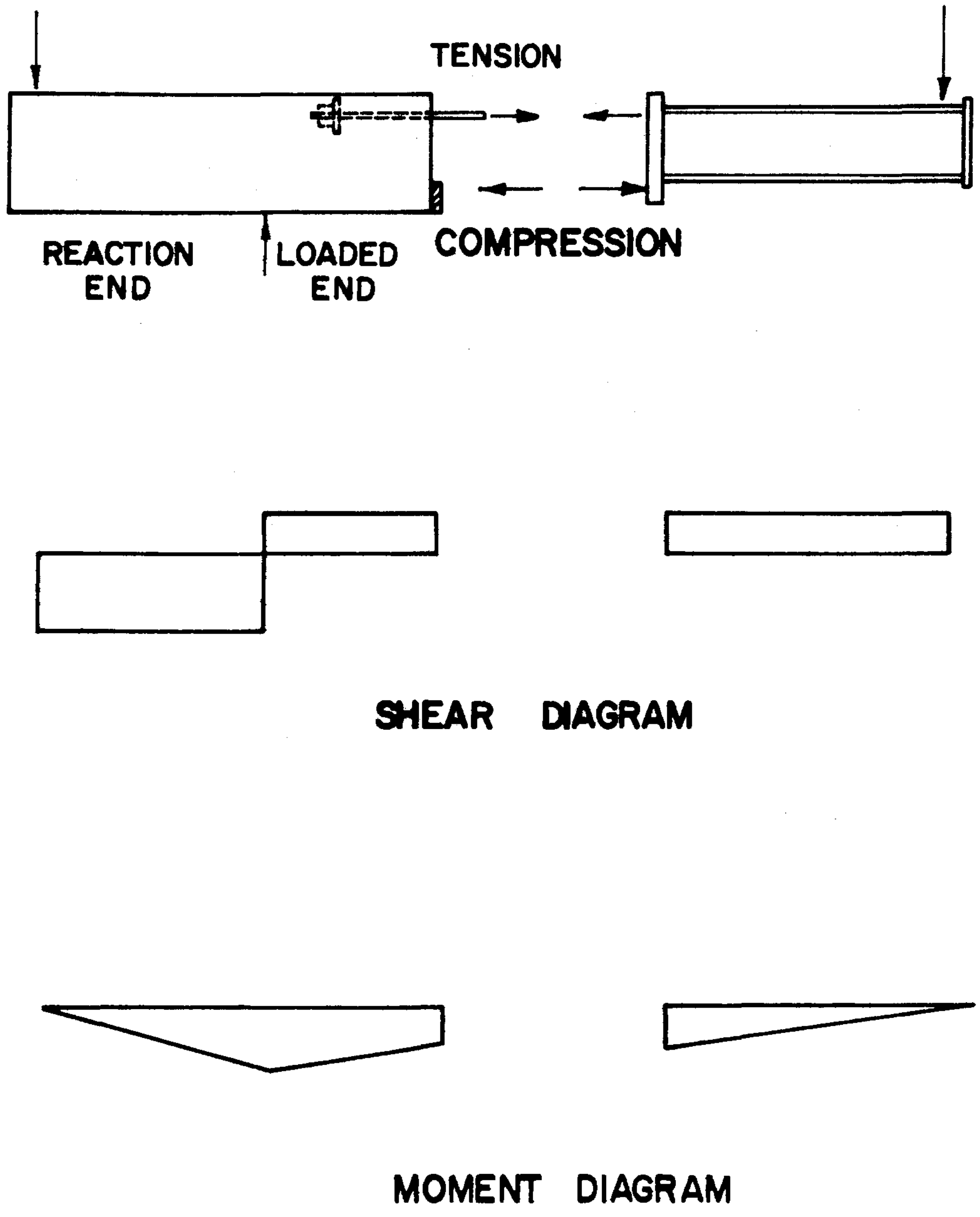


Fig. 3.6 Freebody of specimen and applied forces

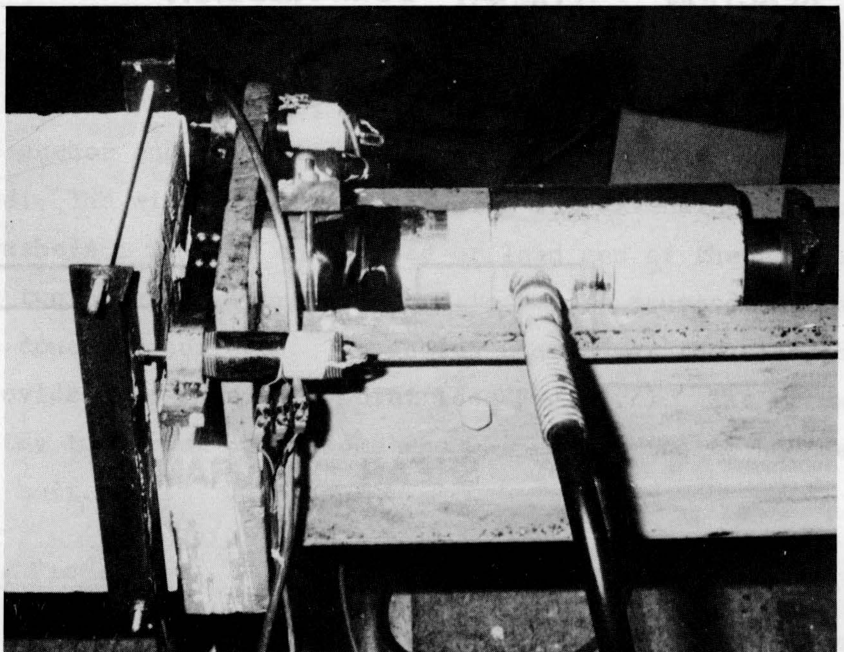


Fig. 3.7 Lead slip measuring device--model tests

CHAPTER 4

TEST RESULTS

4.1 Introduction

In this chapter the modes of failure that were observed in the test program will be discussed. A general description of the behavior of high-strength anchor bolts will be presented and the influence of clear cover, bolt length, bolt diameter, and washer diameter will be discussed.

4.2 Basic Calculations

To avoid ambiguity it is necessary to define the terms that will be used in the discussion of the results of the test program. Figure 4.1 illustrates the terms used to describe the geometry of the anchor bolt installation.

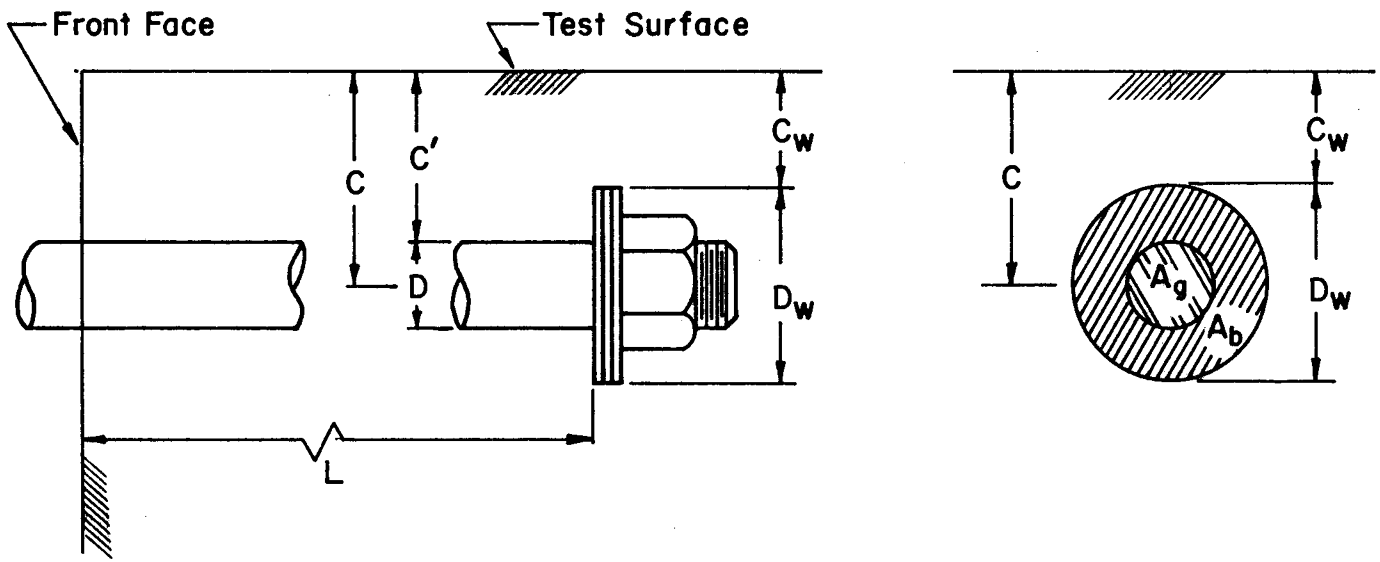
(1) Applied Load (P) - total load measured at the hydraulic rams.

(2) Bolt Load (T) - load on the bolt as determined from the applied load and the test frame geometry. The method of calculating bolt load will be discussed below.

(3) Mean Steel Stress (f_{sm}) - stress on the bolt based on the mean stress area (A_m) as recommended by ASTM Standards to take into account the effect of the partially threaded anchor bolt.[8]

(4) Gross Steel Stress (f_{sg}) - stress on the bolt based on the gross area of the bolt (A_g).

(5) Slip - total extension of the anchor bolt as measured by the slip potentiometers. Slip, therefore, includes the actual



D_w = Washer Diameter
L = Embedment Length
C = Cover to ℄ of Bolt
C′ = Clear Cover Over Bolt
C_w = Clear Cover Over Washer
D = Bolt Diameter
A_g = Bolt Gross Area
A_b = Net Washer Area = Bearing Area

Fig. 4.1 Definition of the terms describing an anchor bolt installation relative displacement between the anchor bolt and the concrete, called true slip, and the elongation of the bolt in its unbonded length.

(6) Lead Slip, Lead Stress - bolt slip and stress measured at the face of the concrete.

(7) Washer Slip - slip measured by the slip wires installed in front of the washer of the anchorage device.

(8) Nut Slip - slip measured by the slip wire installed on the nut of the anchorage device.

(9) Normalized Slip - value of slip divided by the embedment length of the bolt.

(10) Tail Stress - bolt stress determined from the strain gage located in front of the washer of the anchorage device.

The load on the bolt in the full-scale tests was determined from the applied load and the geometry of the test frame, as shown in Fig. 2.12.

The bolt load determined from the applied load was checked against load calculated from the two lead strain gages. Using a linear strain distribution across the bolt defined by the strain readings and the stress-strain curve for the bolt material, a stress distribution was obtained which was numerically integrated over the area of the bolt to obtain the total bolt load. For all the full-scale tests, the bolt load determined from the applied load and the geometry of the test frame agreed well with the bolt load determined from the lead strain gages.

For the model tests, bolt loads were determined directly from the load cell readings, since the load cell was placed over the bolt and the hydraulic ram reacted directly against it.

In order to account for the partially threaded anchor bolt, all steel stresses related to discussion of anchor bolt strength or capacity were calculated using the mean stress area of the bolt according to the recommendations of ASTM Specification A307.[8]

The slip potentiometers measured total or gross slip including the actual relative displacement between the anchor bolt and the concrete, and the elongation in the unbonded length of the bolt. An exact determination of the true slip is impossible due to the highly indeterminate nature of the combined stress conditions along the bolt. It is understood, therefore, that all discussion of slip refers to the gross slip as measured during the test.

As discussed earlier, the slip wires were arranged to provide a check on the consistency of the performance of the slip-measuring system. In general, it was found that the slip wires performed consistently and smoothly. Figure 4.2 shows lead slip as measured by the front yoke plotted against that measured by the lead slip wire. Comparison of the curves illustrates a difficulty that was present in a few tests. It can be seen that there is a consistent offset between the curves. It can be assumed that one of the slip measuring systems did not record initially and the slip wire reading could be adjusted for this offset. However, the adjustment would be different for each test and for lead or washer and nut slip readings. Consequently, in order to maintain consistency, no correction of any kind is applied to any slip measurement and the slip data are plotted as recorded. Therefore, slip readings at early stages of loading may not be as reliable as they are at later stages of loading.

4.3 Failure Modes

Failures of the specimens fell into three categories:

(1) The bolt failed by reaching its ultimate strength, or yielding in the threaded region.

(2) The concrete cover failed by spalling.

(3) The concrete cover failed by wedge-splitting.

These three categories represented distinct failure modes; however, combinations of these modes were observed in several instances.

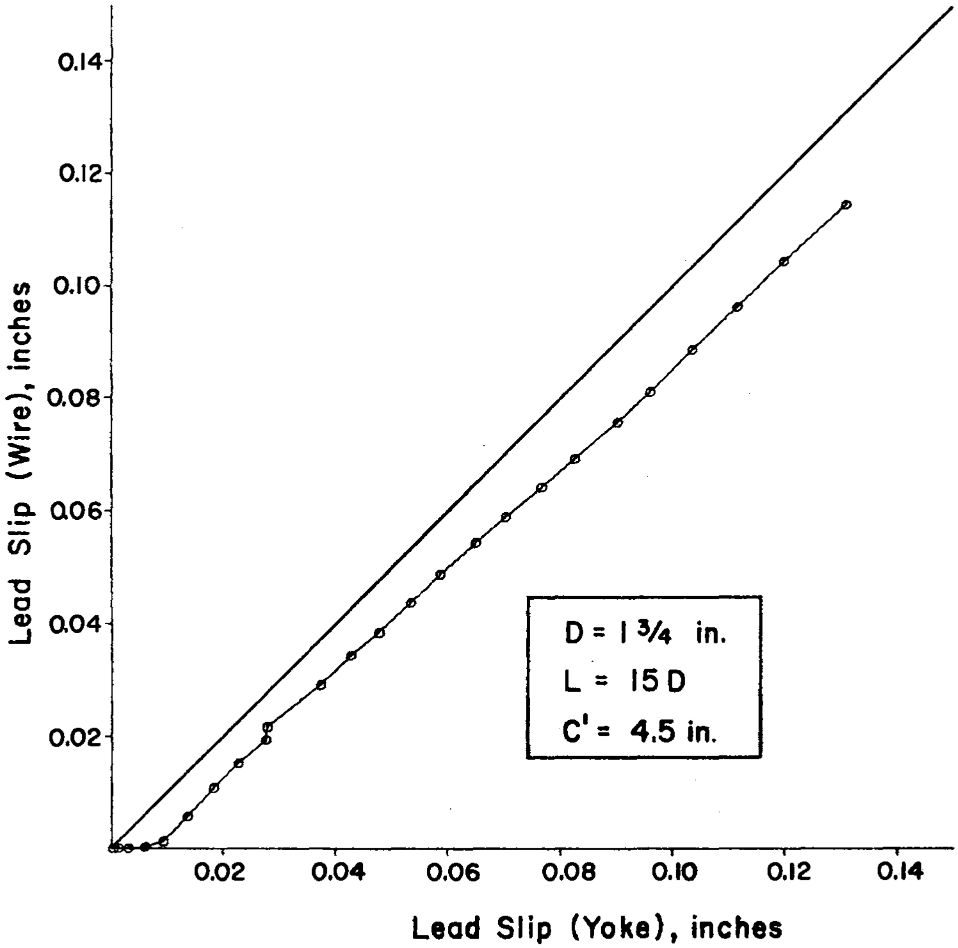


Fig. 4.2 Comparison of lead slip measuring devices--full-scale tests

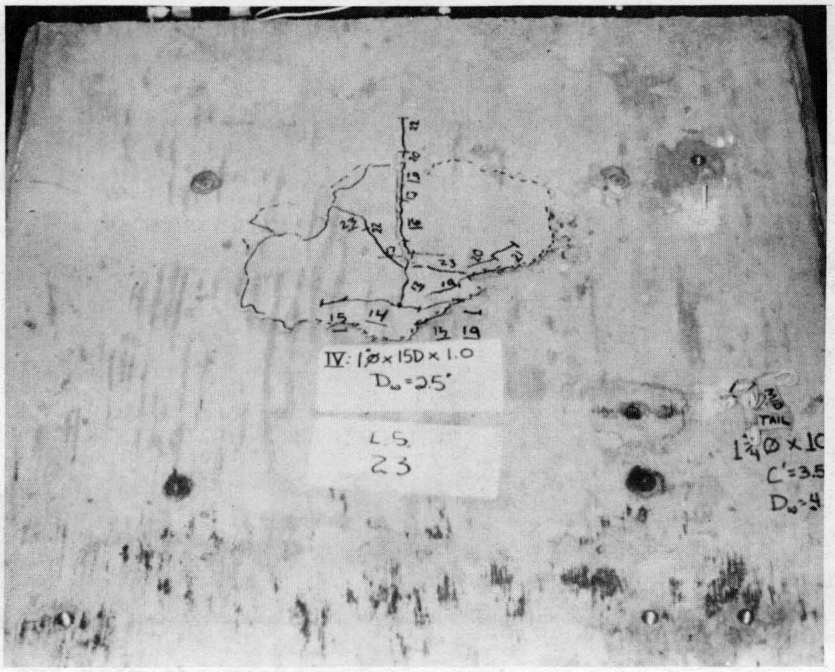
4.3.1 Bolt Failure. Nearly all of the 1 in. diameter bolts tested reached yield, and about half reached ultimate strength, as indicated by the observation of necking in the threaded portion of the bolt. Very little damage to the concrete cover over the bolt was observed at failure. Only one 1 in. bolt, with a clear cover of 1.0 in., was unable to reach yield strength. None of the 1-3/4 in. bolts reached yield.

In the second specimen tested, a 1 in. bolt with a clear cover of 4.5 in. and an embedment length of 15D reached its ultimate material strength without damaging the concrete cover. The remaining 1 in. bolt in the specimen was identical to the first, except that it had an embedment length of 20D. It seemed likely that the 20D bolt would also reach its ultimate strength without damaging the concrete cover, therefore providing little additional information, and, consequently, the 20D bolt was not tested.

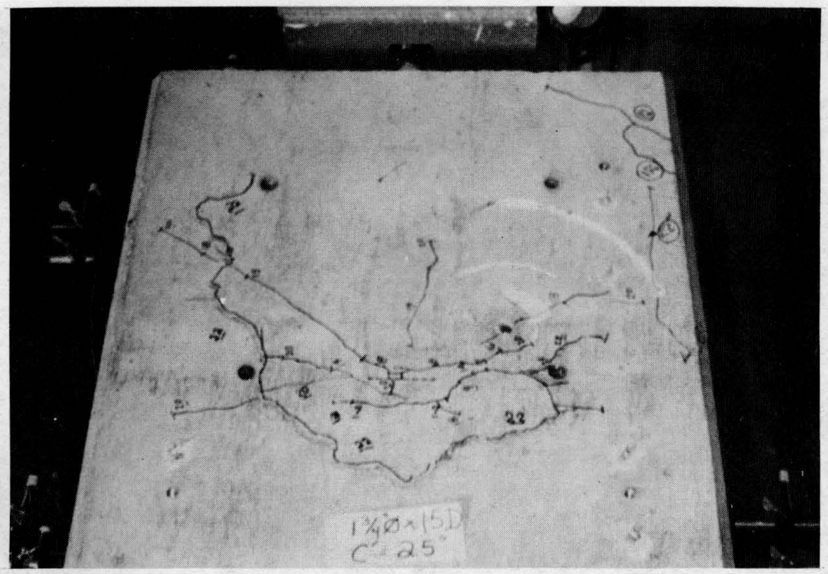
In the model tests only four bolts were able to develop yield. In all cases the embedment length was more than 10 bolt diameters and the concrete strength was high. Only minor cracking was noted prior to failure of the bolt.

4.3.2 Cover Spalling. A relatively sudden spalling of the cover over the anchorage device at low loads characterized the failure of bolts with low values of clear cover, C'.

Figure 4.3(a) shows the failure of a 1 in. bolt with a C' of 1.0 in.; this bolt did not reach its yield strength. The region of spalling was localized and there was little cracking outside this region. Figure 4.3(b) shows the failure of a 1-3/4 in. bolt with a C' of 2.5 in. As might be expected with the deeper cover, the region of spalling was more extensive than that seen in Fig. 4.3(a). The maximum load reached by this 1-3/4 in. bolt was significantly lower than that reached by most of the other 1-3/4 in. bolts in the test series. Figure 4.4 shows similar failures for the model tests.

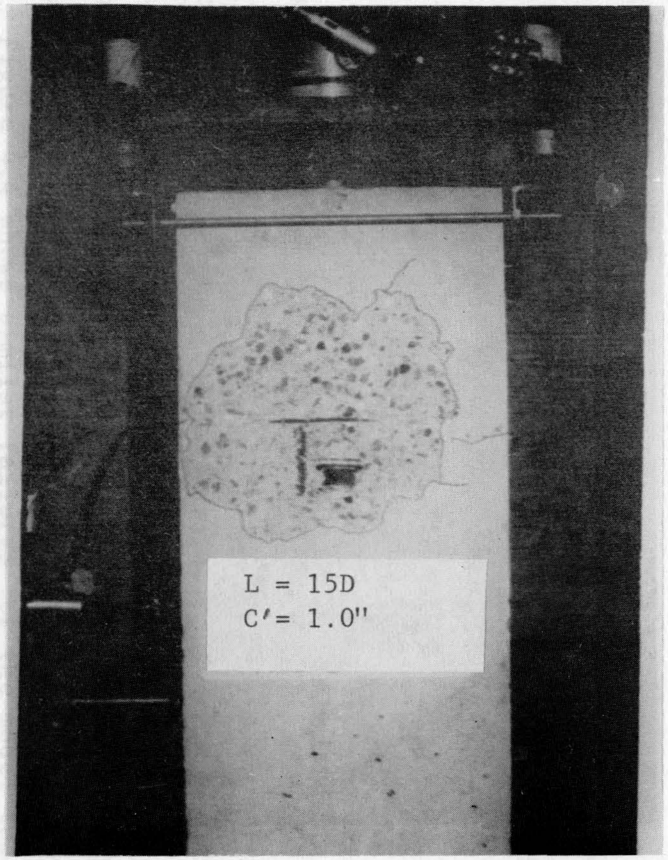


(a) 1 in. bolt, C′ = 1.0 in.

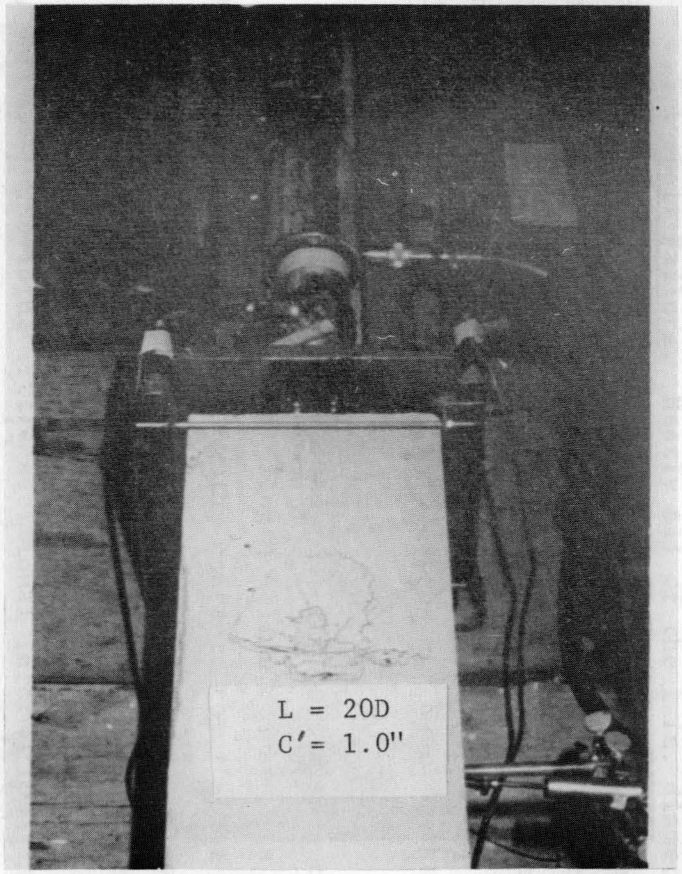


(b) 1-3/4 in. bolt, C′ = 2.5 in.

Fig. 4.3 Cover spalling--full-scale tests



(a) 1/2 in. bolt, C′ = 1.0 in., L = 15D



(b) 1/2 in. bolt, C′ = i.o in., L = 20D

Fig. 4.4 Cover spalling--model tests

4.3.3 Wedge-splitting. For slightly larger values of clear cover, the failure of the specimens was characterized by the splitting of the concrete cover into distinct blocks by the wedging action of a cone of crushed and compacted concrete which formed in front of the washer of the anchorage device. Figure 4.5 shows this cone after failure with the damaged cover removed to facilitate its examination. Figure 4.6 shows the cone in place in front of the washer after the removal of a 1-3/4 in. bolt from the specimen after failure. Most of the failures involved wedge-splitting to some degree.

The distinguishing feature of a wedge-splitting failure was the diagonal cracks which started just in front of the washer on the bolt centerline and extended toward the front and each side of the specimen. These diagonal cracks were frequently accompanied by a longitudinal crack along the bolt axis, and/or a transverse crack parallel to and near the washer of the anchorage device. Cracking generally started near the anchorage device and extended toward the front and/or the sides of the specimen under increased loading. The sequence of cracking is shown in Fig. 4.7 for one of the model tests.

Figure 4.8 shows wedge-splitting with the characteristic diagonal and longitudinal cracking of a 1-3/4 in. bolt with a clear cover of 4.5 in. and an embedment length of 15D. Figure 4.9 shows wedge-splitting for a 1 in. bolt with a clear cover of 2.5 in. and an embedment length of 15D. The primary mode of failure was wedge-splitting, although some indication of cover spalling can be seen.

4.4 General Response under Loading

The mechanism by which the anchor bolt carries load and transfers it to the concrete member is a sequence of three stages involving (1) steel-to-concrete bond, (2) bearing against the washer of the anchorage device, and (3) a wedging action by the cone of crushed and compacted concrete in front of the anchorage device. These three stages are not entirely distinct, of course. The exact nature of the transitions from one stage to the next is, however, highly indeterminant and will only be discussed in a general manner.

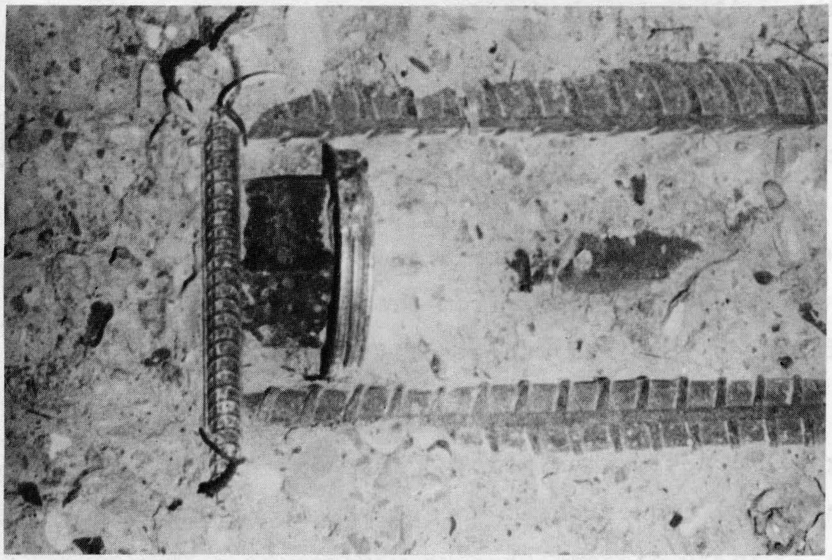


Fig. 4.5 Cone of crushed concrete in front of anchorage device

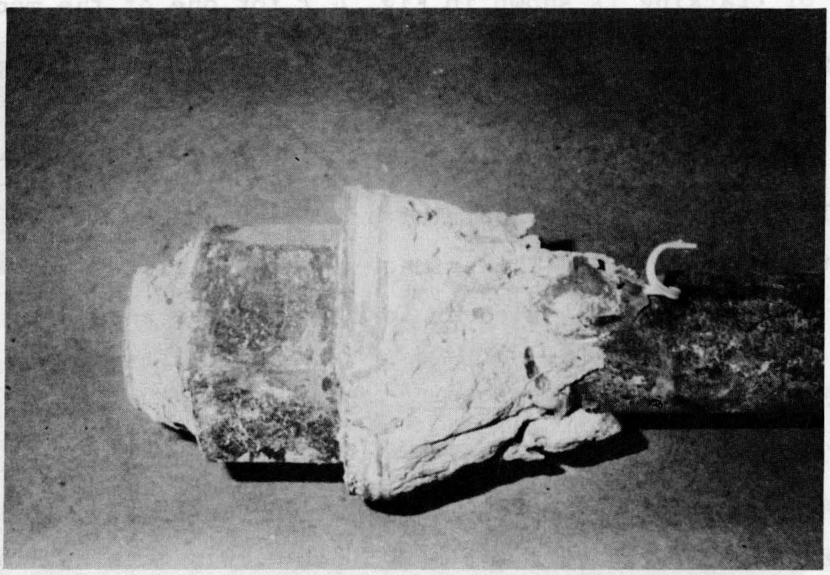
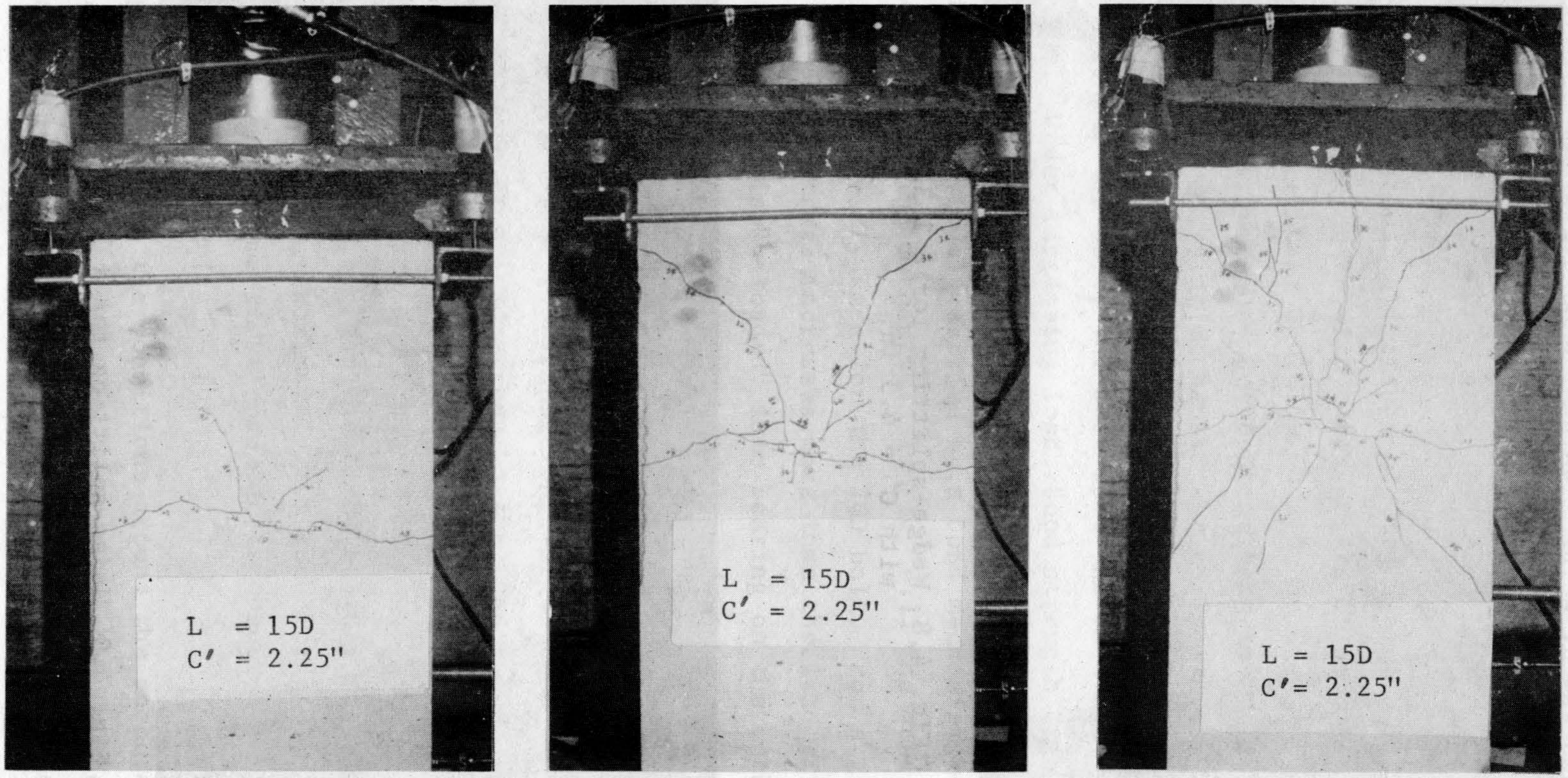


Fig. 4.6 Anchor bolt removed from specimen after failure, showing cone of crushed concrete



(a) Initial cracking (b) Near failure (c) Failure

Fig. 4.7 Sequence of wedge-splitting cracks--model tests
[1/2 x 15D x 2.25 (1.06)]

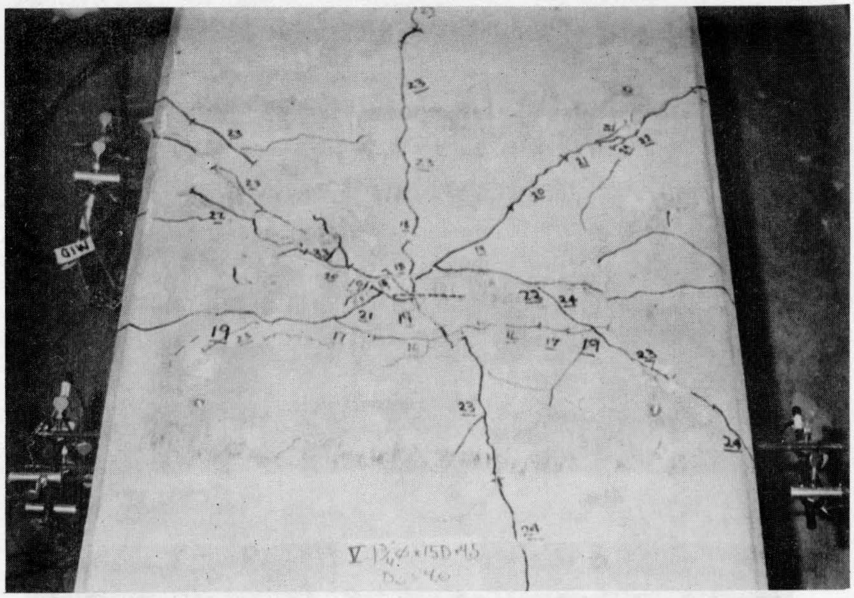


Fig. 4.8 Wedge-splitting for a 1-3/4 in. bolt with C′ = 4.5 in.

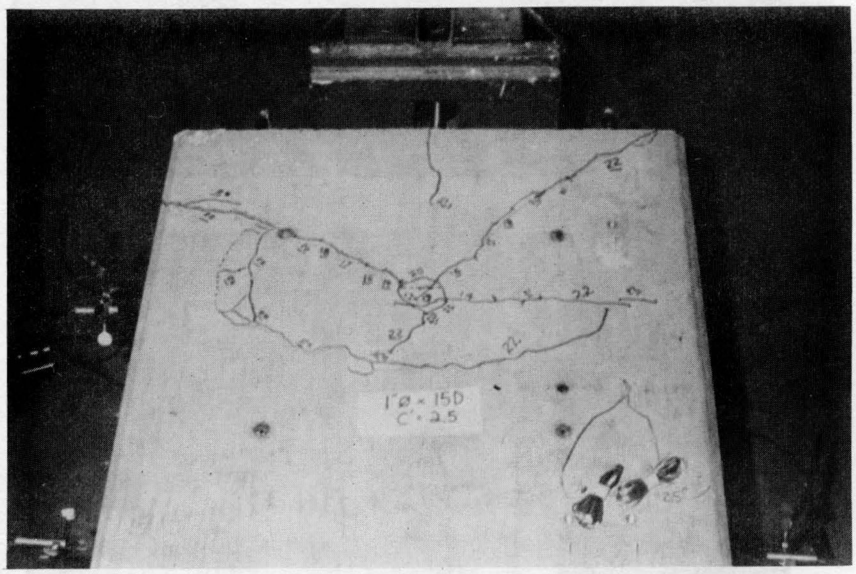


Fig. 4.9 Wedge-splitting for a 1 in. bolt with L = 15D

Figure 4.10 shows a typical stress-slip curve. Lead slip usually started immediately upon loading. Nut slip was usually first observed at a load on the order of 35 to 55 percent of the ultimate bolt load.

Figure 4.11 shows tail stress plotted against lead stress for three 1-3/4 in. anchor bolts with clear covers of 3.5 in. and three different embedment lengths: 10, 15, and 20 bar diameters. As shown in the figure, adhesion or bond between the bolt and the concrete is the predominant load-carrying mechanism for early stages of loading; very little increase in tail stress is observed with increasing lead stress. The longer the bolt, the more load the bolt can carry by the bond mechanism. Under increasing load, bond strength deteriorates along the length of the bolt until the tail stress begins to increase. The load that was previously carried by a bond mechanism must be transferred to a bearing mechanism. In Fig. 4.11 the bond to bearing transition is most clearly seen for the 20D bolt. For a given load increment, the tail stress increases more than the lead stress as the load carried by bond is "unloaded" into bearing on the anchorage device. The bond-bearing transition is dependent on the embedment length of the bolt; the shorter the bolt, the shorter and less well-defined the transition.

After the bond-bearing transition, tail stress increases uniformly with increasing lead stress as the load is carried by bearing or by wedging action. The wedging action is initiated by the formation of the cone of crushed concrete in front of the anchorage device, as seen in Fig. 4.6. For the three bolts in Fig. 4.11, first cracking was observed at approximately the same lead stress. It can be assumed that by the time first cracking is observed the cone of concrete is already beginning to form.

Figure 4.12 shows the conditions around the anchorage device after the cone has formed. The bolt load is carried by the cone wedging against the concrete cover. The characteristic diagonal

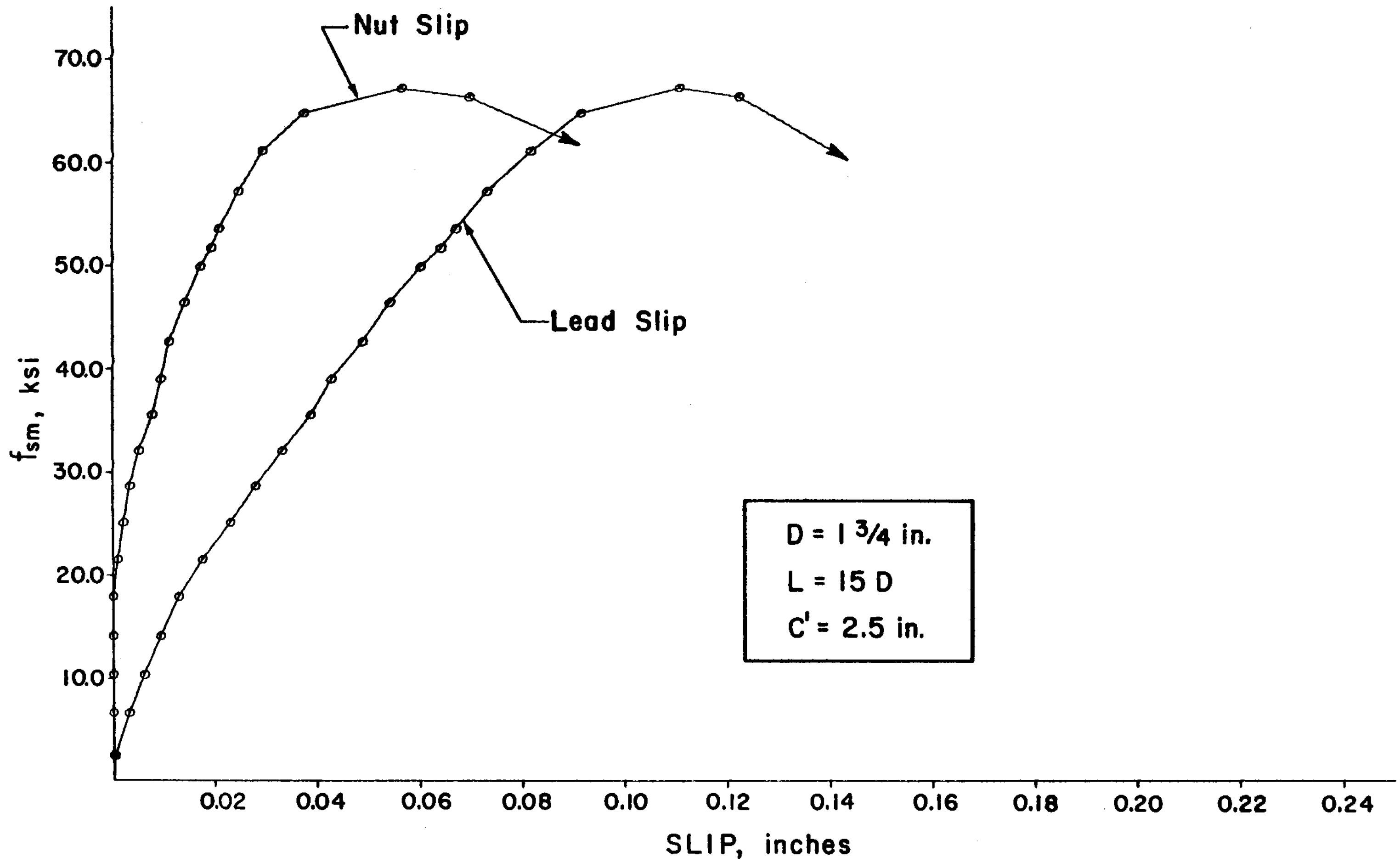


Fig. 4.10 Typical stress-slip curve

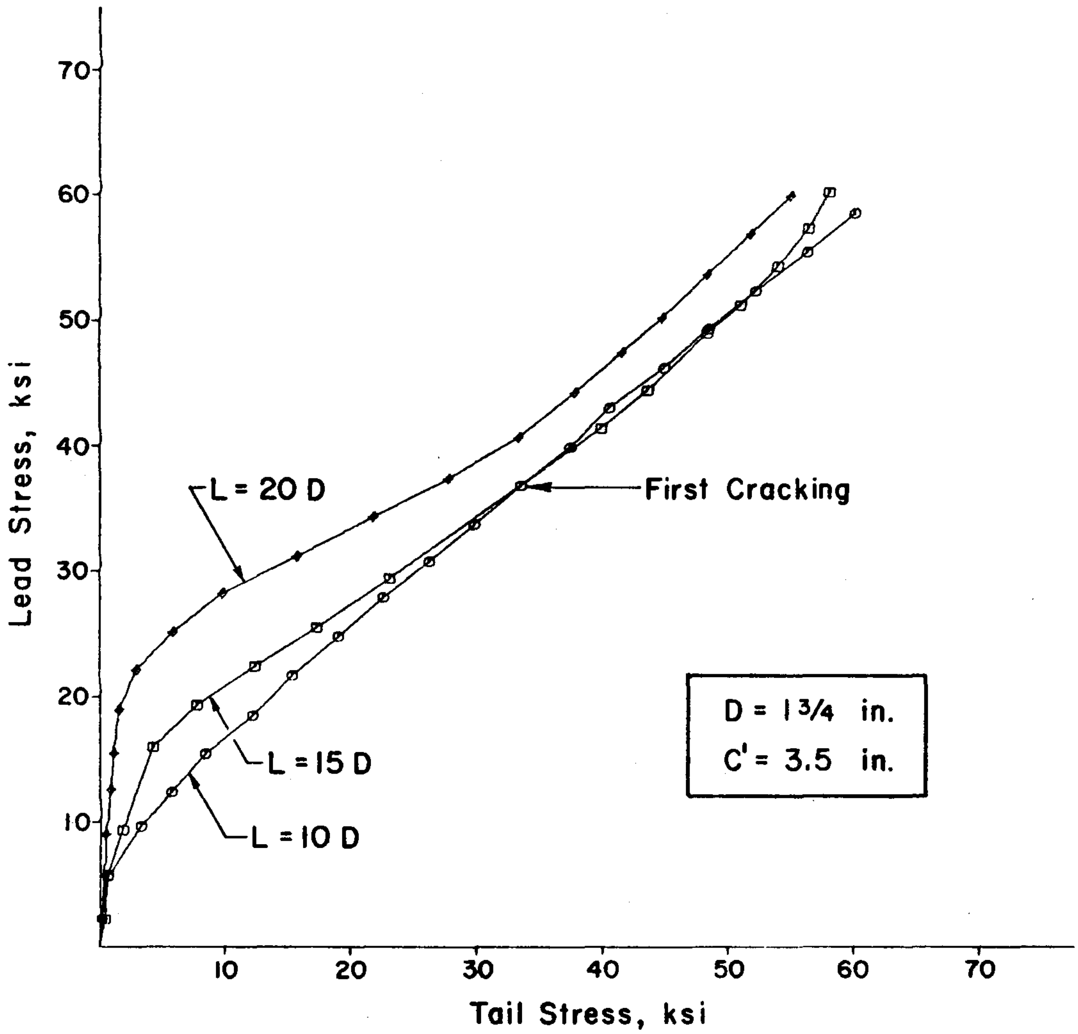


Fig. 4.11 Tail stress vs. lead stress for different embedment lengths

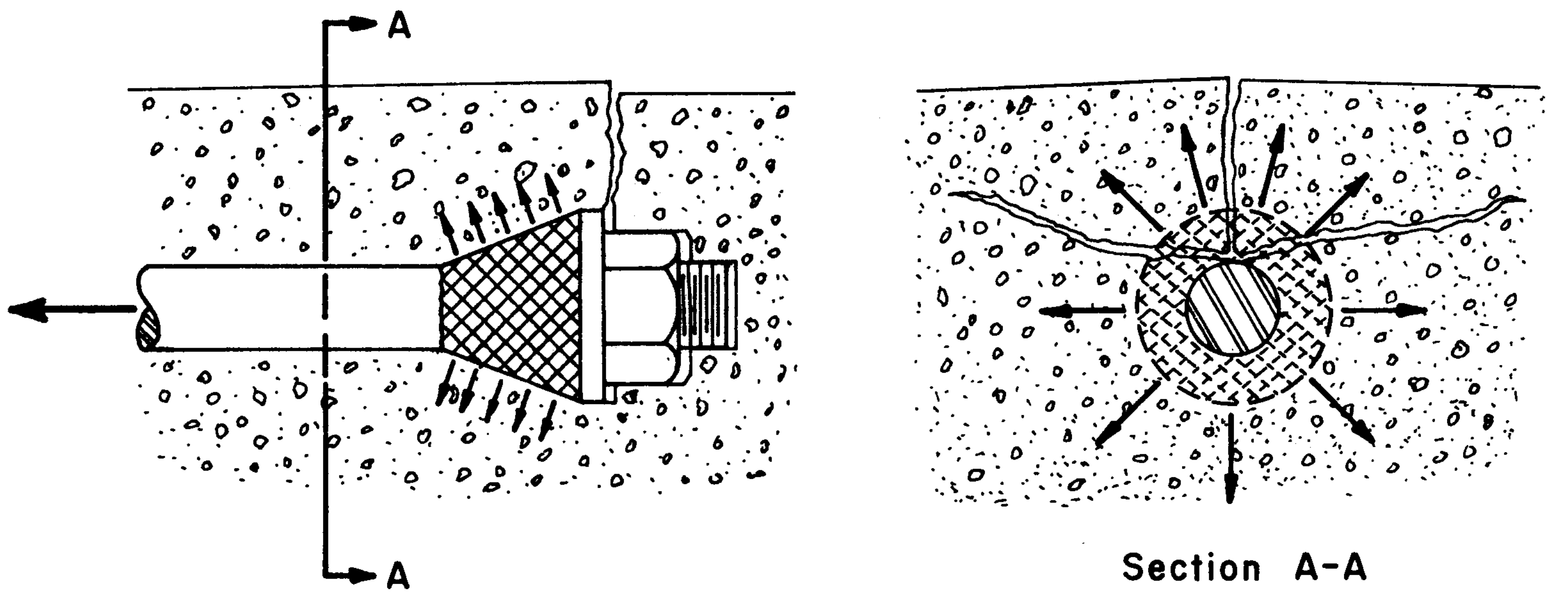


Fig. 4.12 Conditions around the anchorage after formation of the cone of crushed concrete

cracks of the wedge-splitting failure are caused by tensile forces generated by this wedge action. The wedge action also generates circumferential tensile forces which tend to form longitudinal cracks along the bolt axis. The increasing slip of the bolt tends to extend the characteristic first crack to either side of the specimen. Under increasing load, these cracks continue to propagate forming distinct triangular blocks whose apexes meet in front of the anchorage device. Near ultimate these blocks are forced outward by the wedge action of the cone of crushed and compacted concrete in front of the anchorage device. When the cracks that form the boundary of the blocks have extended sufficiently toward the sides and the front of the specimen so that the blocks can no longer resist the wedging action, the specimen fails.

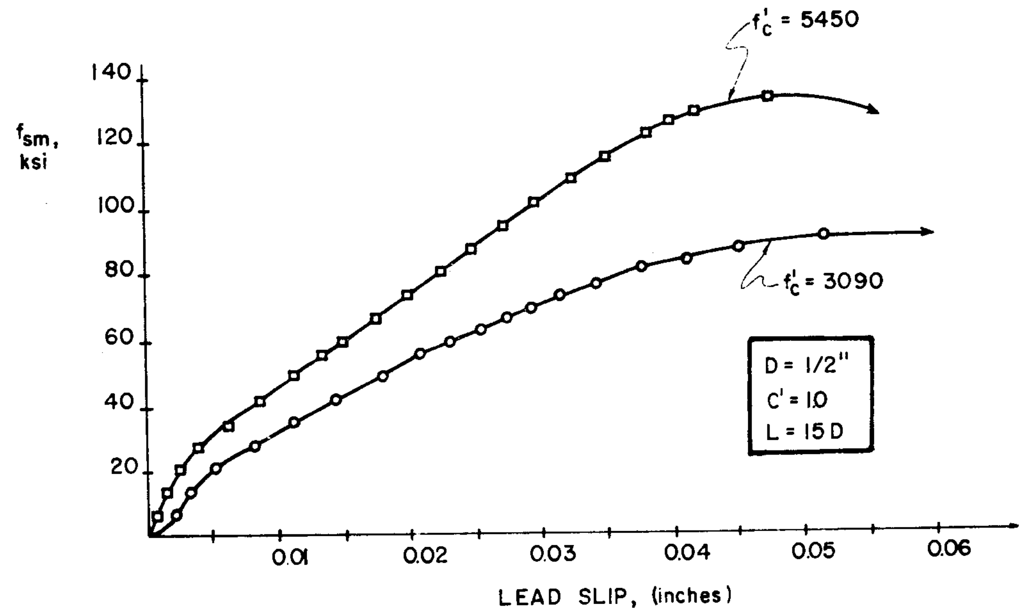
CHAPTER 5

EFFECT OF PRIME VARIABLES ON BOLT BEHAVIOR

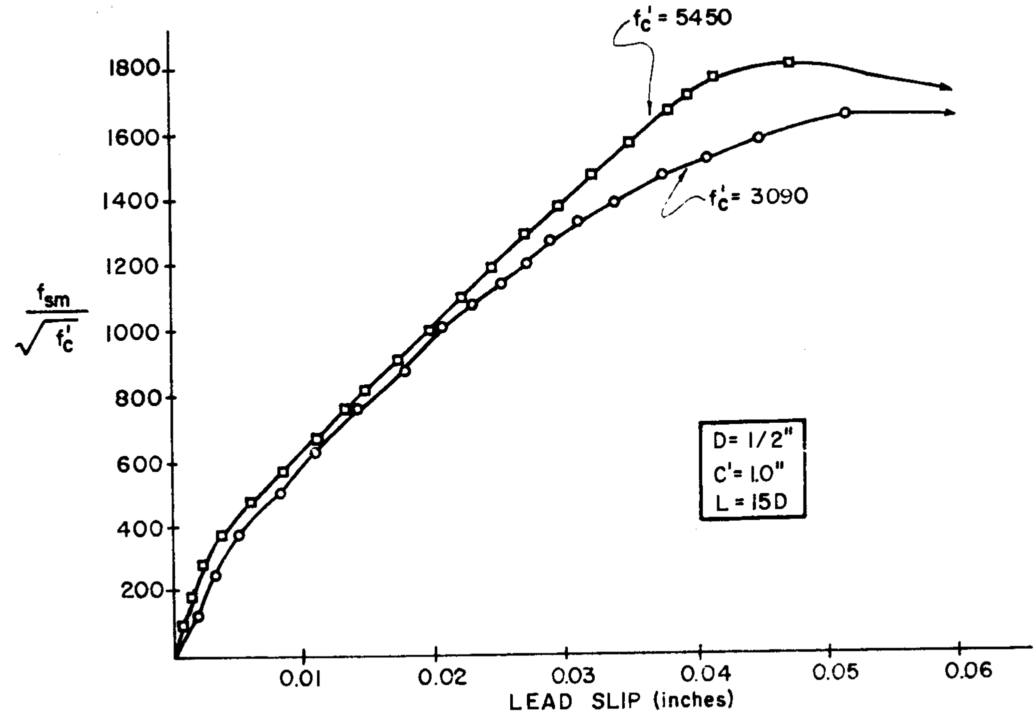
5.1 Introduction

The prime variables studied in the test program included (1) clear cover, (2) embedment length, (3) bolt diameter, and (4) bearing area (washer size). In order to evaluate the effects of these parameters on bolt behavior, the response of anchor bolts will be illustrated by stress-deformation curves in which either mean load bolt stress or bearing stress on the washer is plotted against lead slip. The effects of concrete strength and of confinement from longitudinal reinforcement were also investigated, but these parameters were not considered prime variables.

5.1.1 Effects of Concrete Strength. Several model tests were performed to establish the effect of concrete strength on anchor bolt performance.[5] Figure 5.1(a) shows mean steel stress plotted against lead slip for two model tests with identical geometry but different concrete strengths. Figure 5.1(b) shows that the variation in concrete strength between the two specimens can be approximately accounted for by normalizing the mean steel stress with respect to $\sqrt{f'_c}$. The variation of bolt strength and stiffness is not exactly proportional to $\sqrt{f'_c}$, but for design purposes this method of normalizing is satisfactory and will be used in the discussion of the test results. It was also observed that concrete strength influenced the mode of failure. Figure 5.2 shows the appearance after failure of the two specimens for which data were plotted in Fig. 5.1. With a high concrete strength, a wedge-splitting failure was produced, while for a low concrete strength, a spalling failure resulted.



(a) Nonnormalized lead stress-lead slip curves



(b) Normalized lead stress-lead slip curves

Fig. 5.1 Effect of concrete strength

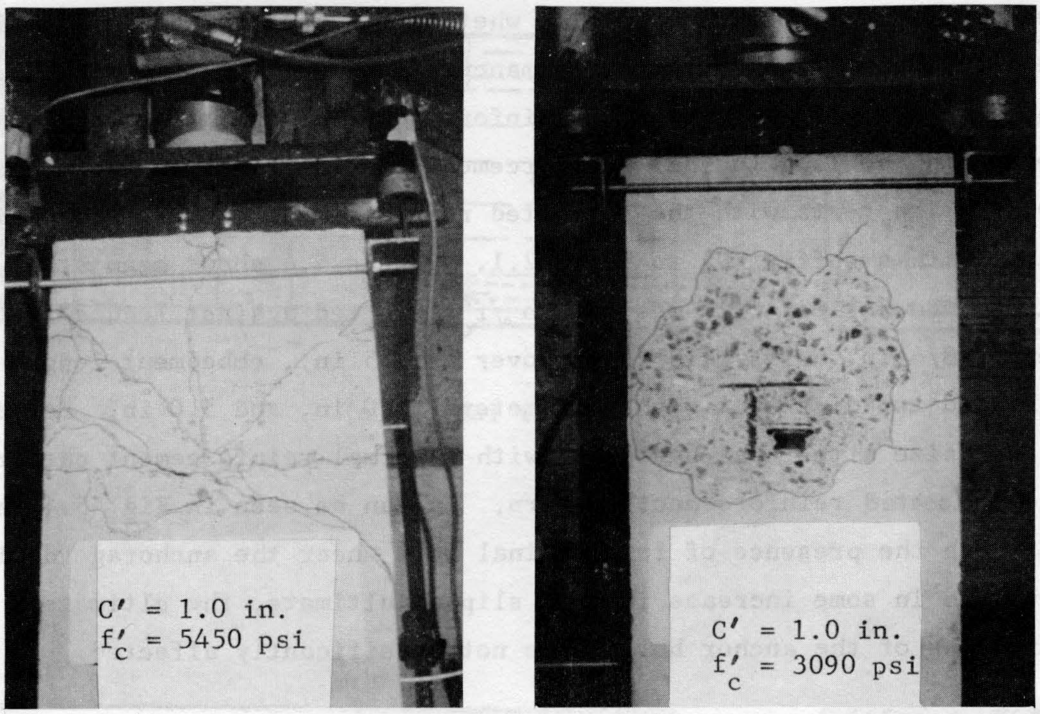


Fig. 5.2 Influence of concrete strength on failure modes--model tests

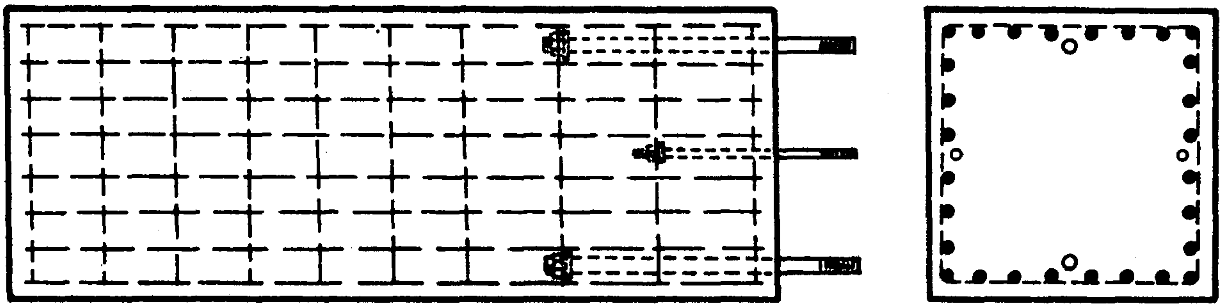
5.1.2 Position of Longitudinal Reinforcement. The normal pattern (uniform spacing on all surfaces) of reinforcement used in the full-scale specimens resulted in longitudinal bars being positioned near the anchorages of the anchor bolts. Several full-scale tests were performed to determine whether the position of longitudinal reinforcement affected the performance of the bolts. Figure 5.3 compares the normal pattern of reinforcing steel and the altered pattern with the longitudinal reinforcement placed away from the anchor bolts. The tests with the relocated reinforcement pattern are designated with a suffix (U) in Table 2.1. Figure 5.4 shows mean steel stress, normalized with respect to $\sqrt{f'_c}$, plotted against lead slip for four 1-3/4 in. bolts with clear cover of 3.5 in., embedment length of 15D, and two different washer diameters, 4.0 in. and 5.0 in. For each washer size a test was performed with a normal reinforcement pattern and a relocated reinforcement pattern. It can be seen in Fig. 5.4 that although the presence of longitudinal bars near the anchorage device resulted in some increase in lead slip at ultimate, the ultimate strengths of the anchor bolts were not significantly affected.

5.2 Effect of Clear Cover

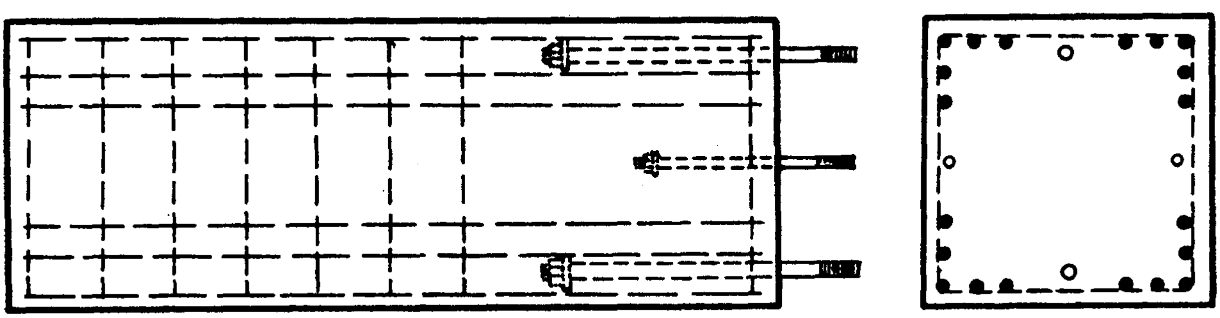
Figure 5.5 illustrates the effect of clear cover on the stress-deformation response of four 1-3/4 in. bolts, each with an embedment length of 15D and an anchorage device consisting of a nut and a standard 4.0 in. diameter, 1/2 in. thick washer. Figure 5.6 shows the effect of clear cover for six 1/2 in. bolts. As seen in the figures, the slopes of the curves are essentially the same until each bolt approaches ultimate. A definite trend of increasing ultimate stress with increasing clear cover is indicated.

5.3 Effect of Embedment Length

Figures 5.7 and 5.8 illustrate the effect of embedment length on the stress-slip relationship of three 1-3/4 in. bolts and three 1 in. bolts. As seen in the figures, the initial slopes of the curves



(a) Normal reinforcement



(b) Relocated reinforcement

Fig. 5.3 Comparison of the normal and the relocated longitudinal reinforcement patterns

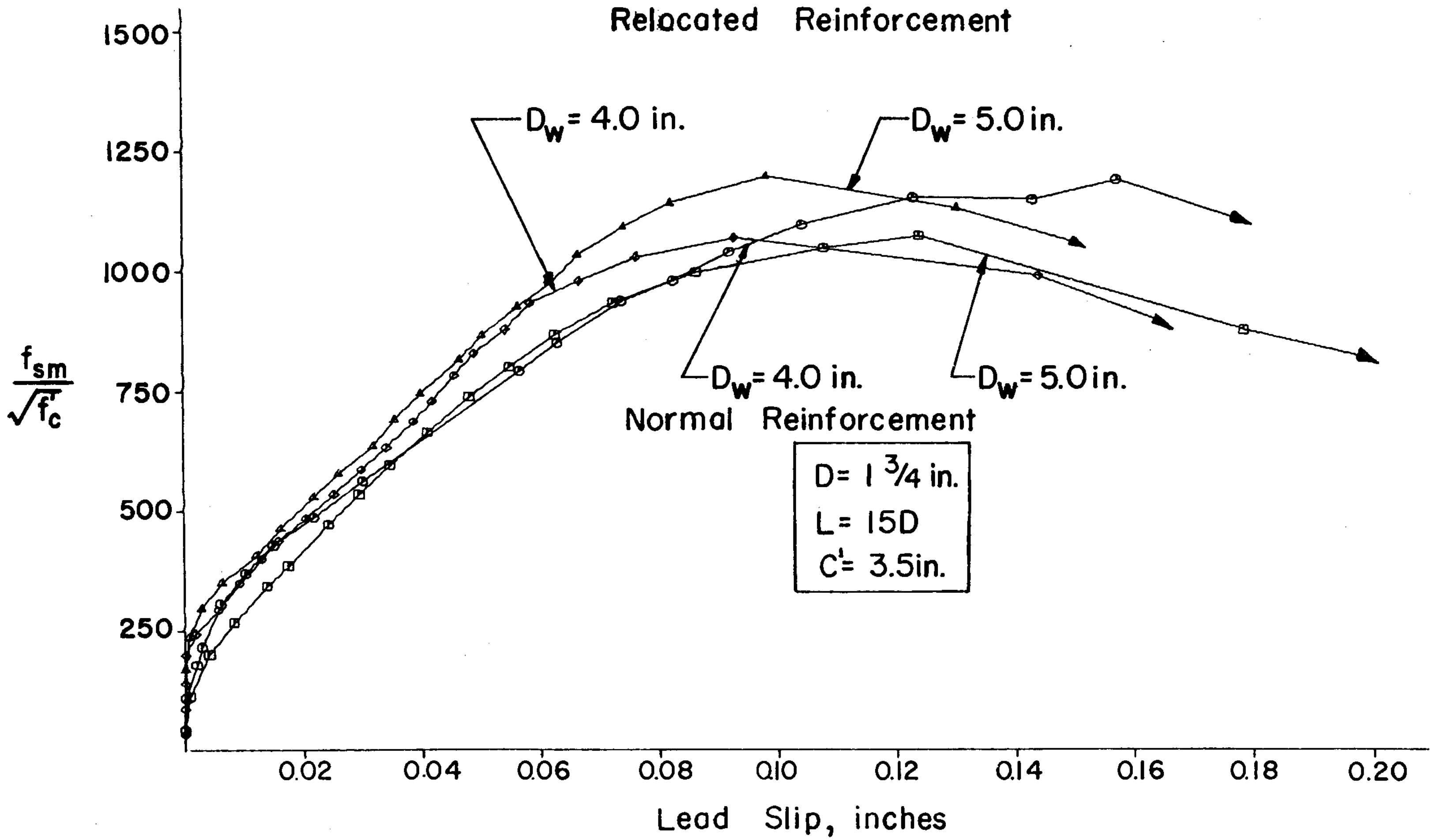


Fig. 5.4 Effect of longitudinal reinforcement

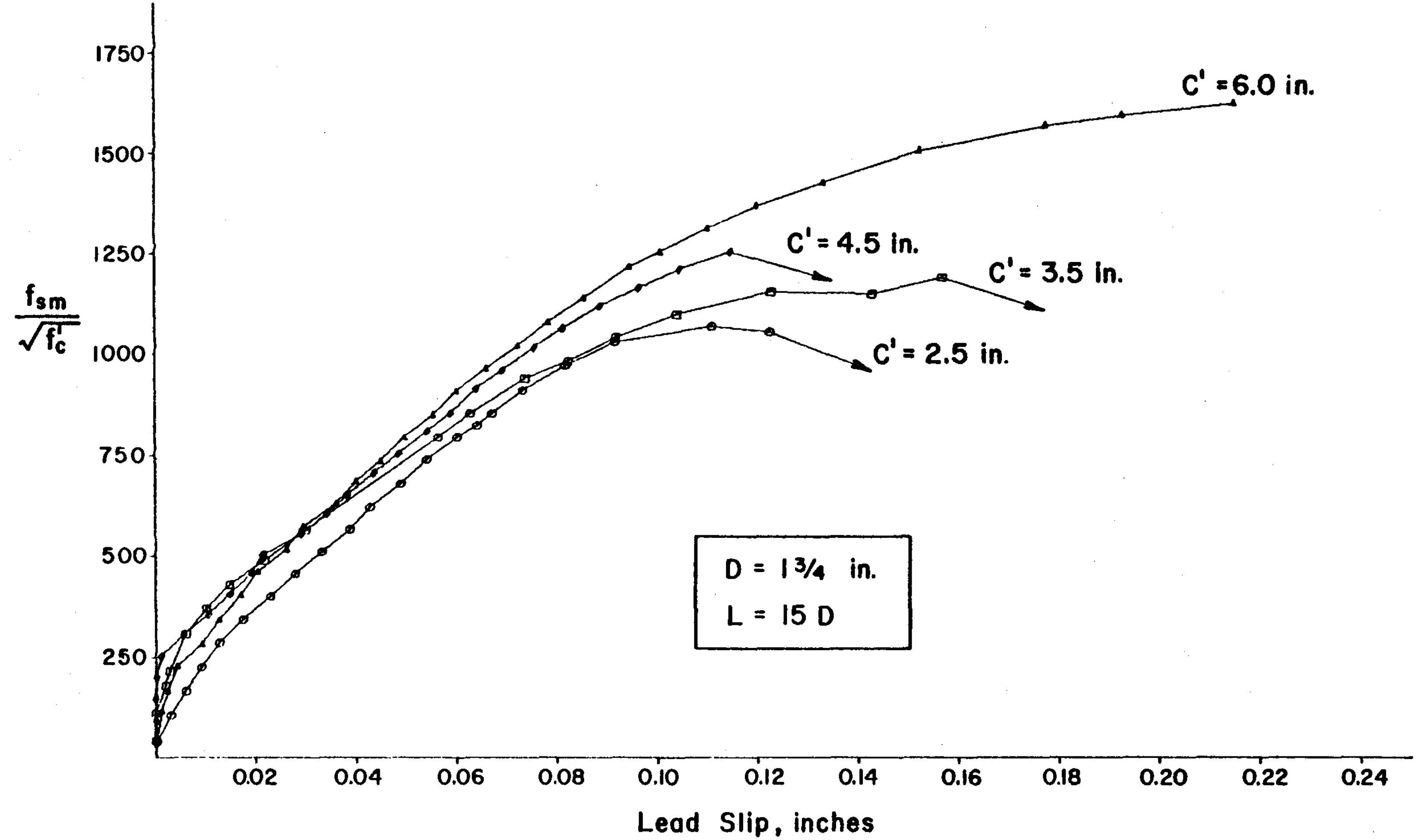


Fig. 5.5 Effect of clear cover--full-scale tests

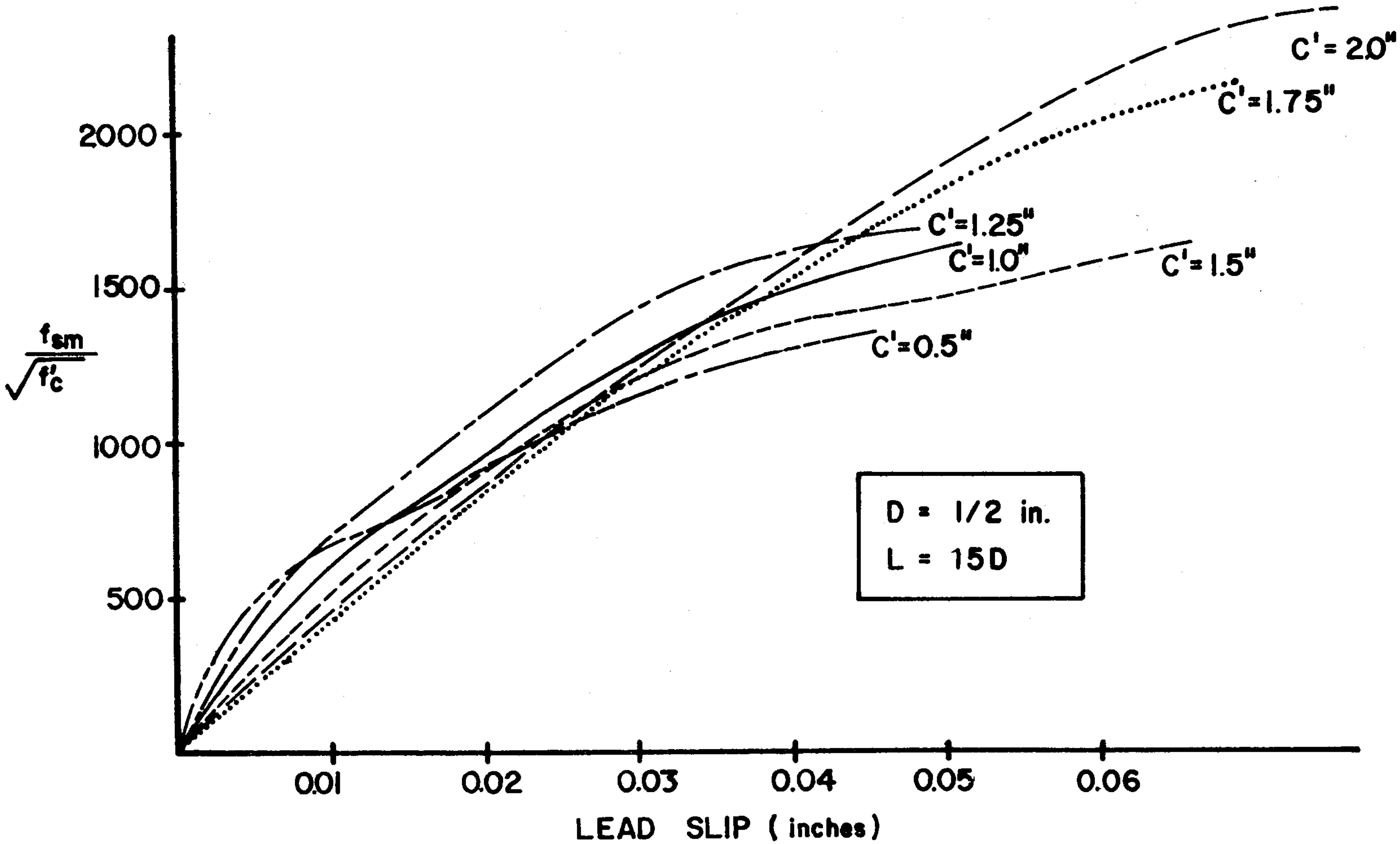


Fig. 5.6 Effect of clear cover--model tests

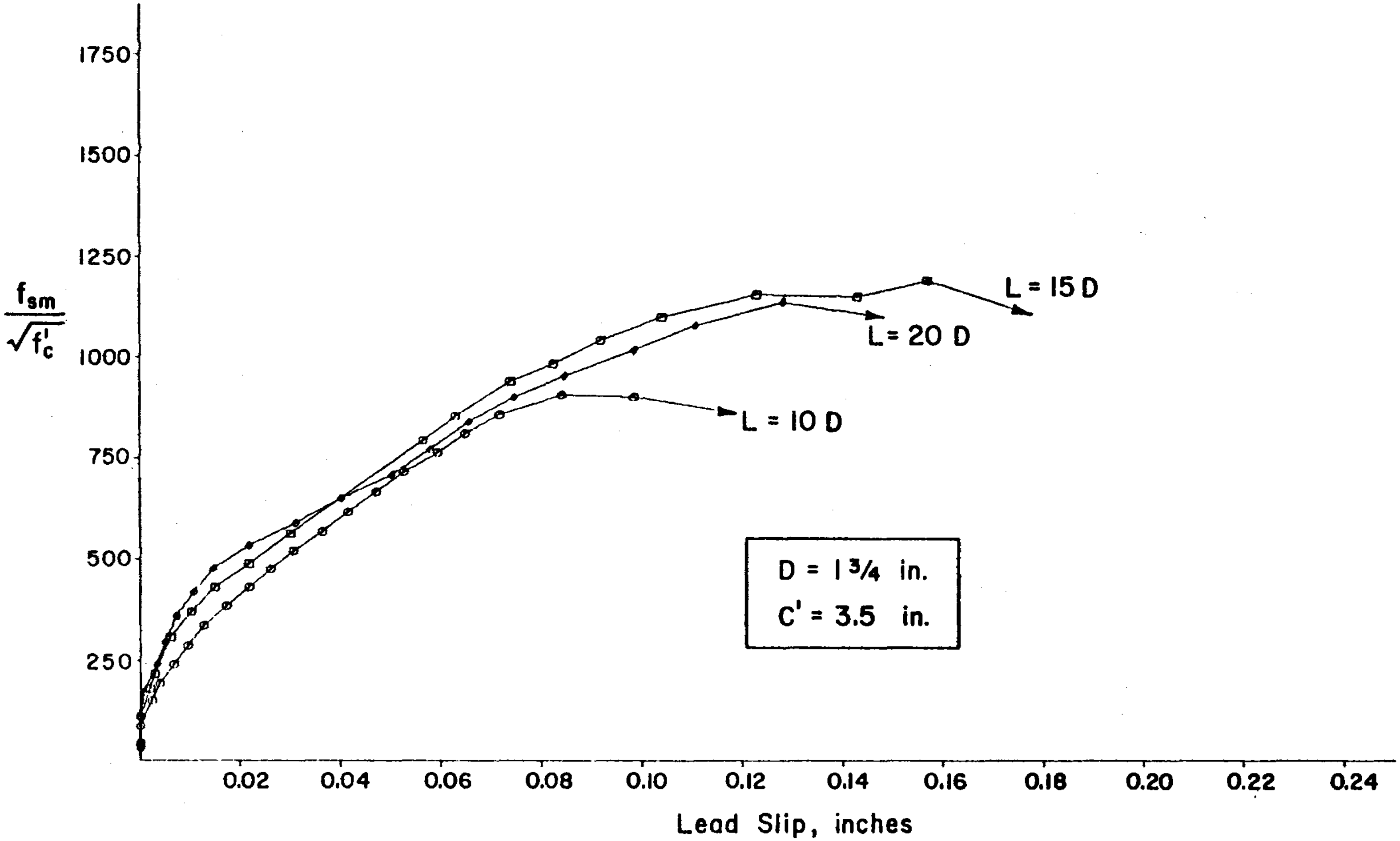


Fig. 5.7 Effect of embedment length (1-3/4 in. bolts)

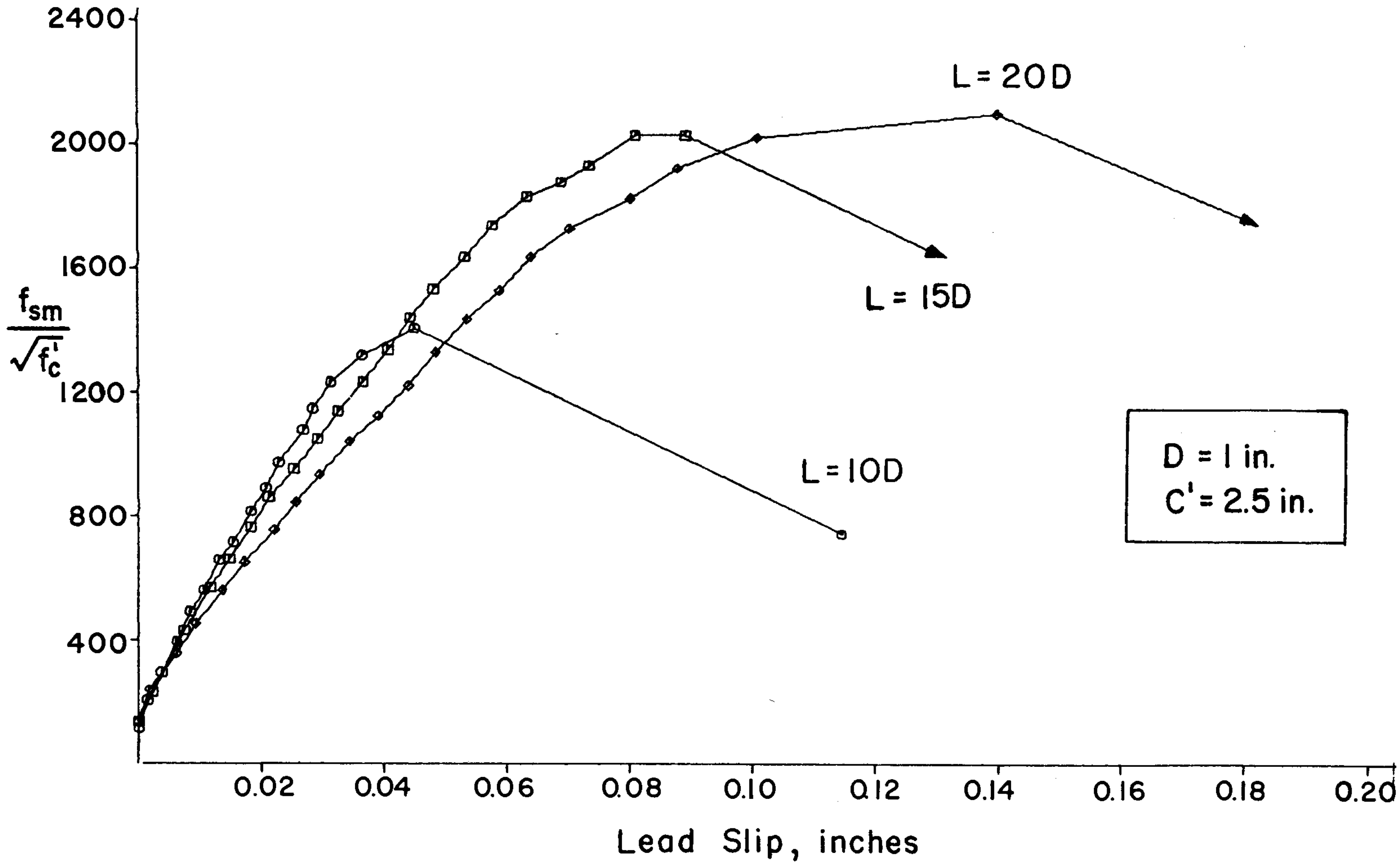
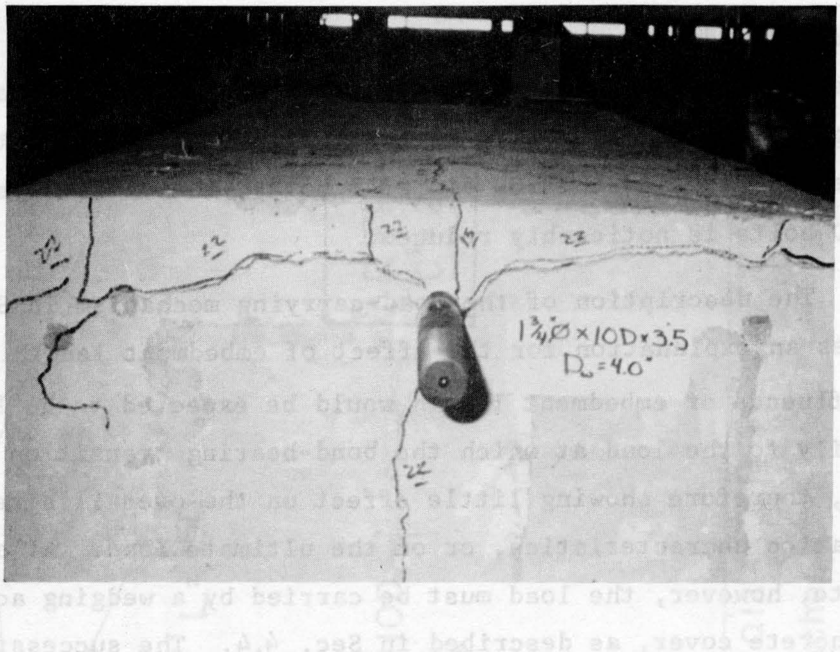


Fig. 5.8 Effect of embedment length (1 in. bolts)

for each bolt diameter are essentially the same. However, some effect of embedment length on the ultimate strength of the bolt can be seen. Although little difference in ultimate strength of the bolt is observed betweeen the 15D and the 20D diameter bolts, the ultimate strength of the 10D bolts is noticeably reduced.

The description of the load-carrying mechanism in Sec. 4.4 provides an explanation for the effect of embedment length. Normally, the influence of embedment length would be expected to be limited primarily to the load at which the bond-bearing transition is completed, therefore showing little effect on the overall stress-deformation characteristics, or on the ultimate load. At or near ultimate, however, the load must be carried by a wedging action against the concrete cover, as described in Sec. 4.4. The successful resistance of the concrete cover to this wedging is dependent on the clear cover, and, to a lesser degree, on the embedment length. Unless there is a sufficient mass of concrete in front of and over the anchorage device, the cracks from the wedge-splitting action of the cone of crushed concrete in front of the washer extend to the free surfaces of the front face and the sides of the specimen and the concrete cover is unable to resist further loading, as indicated by the curve for the 10D bolts.

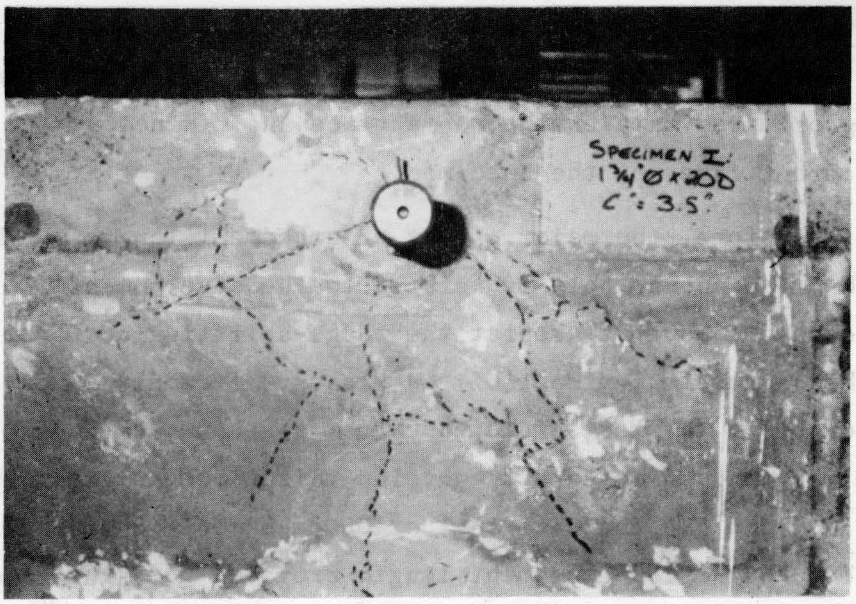
Figure 5.9 shows the 10D bolt for which data are shown in Fig. 5.7 after failure resulted from the complete loss of concrete cover caused by the propagation of cracking to the free surfaces of the specimen. Figure 5.9(a) shows major cracking extending parallel to the test surface across the entire front face of the specimen. In Fig. 5.9(b) the damaged cover has been removed to facilitate examination of the failure surface. In contrast, Fig. 5.10 shows the 20D bolt (data in Fig. 5.7) after failure. It is seen in Fig. 5.10(a) that no major cracking has reached the front face of the specimen. There is a tendency toward shallow spalling of concrete on the front face that is associated with the shear force transferred across the



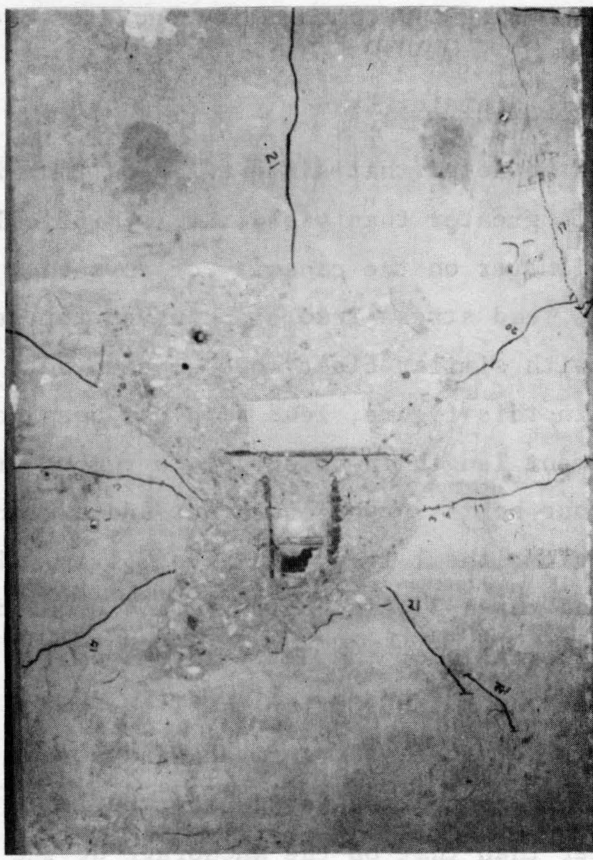
(a) Front face

(b) Damaged cover removed

Fig. 5.9 A 1-3/4 in. bolt with L = 10D after failure



(a) Front face



(b) Damaged cover removed

Fig. 5.10 A 1-3/4 in. bolt with L = 20D after failure bolt at the face of the specimen. In Fig. 5.10(b), the specimen is shown after removal of the damaged cover. It can be seen that cover spalling occurred only on the top surface and did not extend to the side or front faces of the specimen.

Therefore, the major effect of embedment length on the ultimate strength of an anchor bolt is related to the development of the ability of the concrete cover to resist the wedge-splitting action of the cone of crushed concrete in front of the washer. It would appear that in order to obtain the most efficiency from a given clear cover, a certain minimum length is necessary to develop this wedge-splitting resistance. Figures 5.7 and 5.8 indicate that a 10D embedment length is insufficient, but that a 20D embedment length provides no significant improvement over a 15D embedment length. Therefore, in these tests a 15D embedment length can be considered a satisfactory minimum length for the most efficient use of the available concrete cover.

5.4 Effect of Bolt Diameter

It should be noted that since eight of the nine 1 in. bolts failed at stresses greater than yield, it is difficult to evaluate the effect of bolt diameter on the capacity of an anchor bolt installation. Figure 5.11 shows lead stress-lead slip curves for several 1 in. and 1-3/4 in. bolts with similar clear covers; each bolt has an embedment length of 15D. In this figure, lead slip has been normalized with respect to embedment length in an effort to remove its effect from the data. All four bolts shown have a nut and a standard washer for the anchorage device; the 1 in. bolts have 2.5 in. diameter, 1/2 in. thick washers, and the 1-3/4 in. bolts have 4.0 in. diameter, 1/2 in. thick washers. The ratio of the net washer area (or bearing stress) to bolt mean stress area is 6.80 for the 1 in. bolts and 4.88 for the 1-3/4 in. bolts. Therefore, for a given lead stress on the bolt, the bearing stress on the anchorage device for the 1-3/4 in. bolt is about 40 percent greater than that on the anchorage device for the 1 in. bolt.

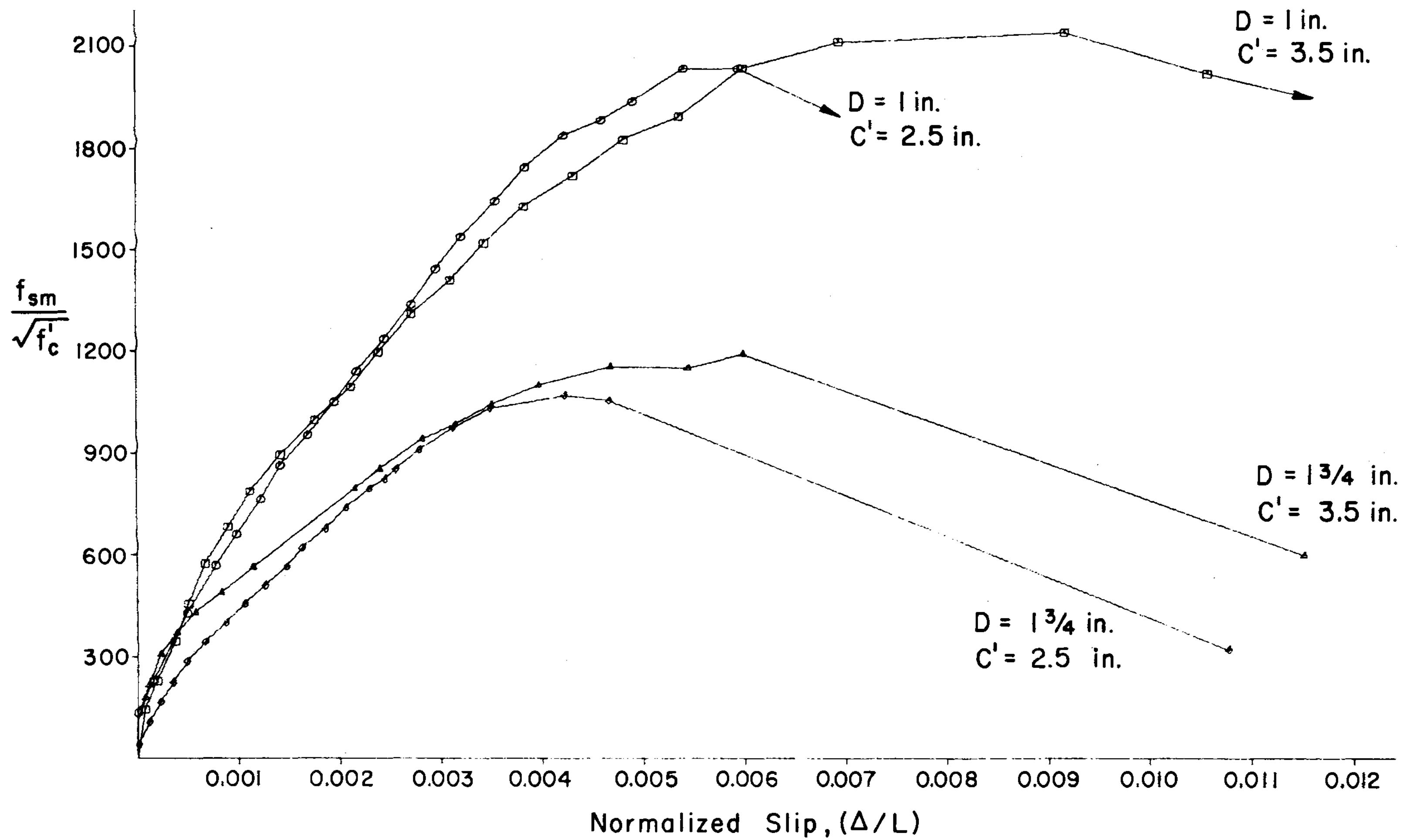


Fig. 5.11 Comparison of lead stress-lead slip curves of bolts with different diameters

Consequently, if a "standard anchor bolt installation" is defined as one with an anchorage device consisting of a nut and a 1/2 in. thick standard diameter washer, Fig. 5.11 compares the performance of two 1 in. and two 1-3/4 in. standard anchor bolt installations. Clearly, the 1 in. standard installations reached a greater ultimate lead stress. Also, for a given lead stress, the 1 in. standard installations showed significantly less lead deformation. A more appropriate measure of serviceability is seen in Fig. 5.12 in which bolt load is plotted against lead slip for the same bolts seen in Fig. 5.11. Note that while the 1 in. standard installations very nearly reached ultimate material strength, the 1-3/4 in. standard installations shown only reached about 50 to 60 percent of ultimate bolt material strength. For the purpose of discussion, let us assume a service load of $(A_m) \times (0.6F_y)$, where F_y is the minimum required yield strength of ASTM A193, Grade B7 material = 105 ksi. The assumed service loads are shown in Fig. 5.12. Clearly, the 1 in. standard installations exhibit significantly less lead deformation at the assumed service loads than the 1-3/4 in. standard installations.

It would be advantageous to evaluate the effect of diameter alone on the behavior of an anchor bolt. In Fig. 5.13, the effect of different washer sizes has been removed by plotting bearing stress on the washer (normalized with respect to square root of concrete strength) against normalized lead slip. It is seen that the curves for both bolt diameters have the same slope. The 1 in. bolts appear to have a higher strength than the 1-3/4 in. bolts. The 1 in. bolts reached essentially the same stress level, while the 1-3/4 in. bolts with clear cover of 3.5 in. reached a higher stress than the 1-3/4 in. bolts with a clear cover of 2.5 in. The differences in ultimate strengths among the bolts shown in Fig. 5.13 can be attributed to the different amounts of relative clear cover. For the same clear cover, the 1 in. bolts have proportionately more concrete over the anchorage than the 1-3/4 in. bolts, as will be discussed more completely below. It is to be expected, therefore, that the 1 in. bolts will reach a higher level of stress than the 1-3/4 in. bolts.

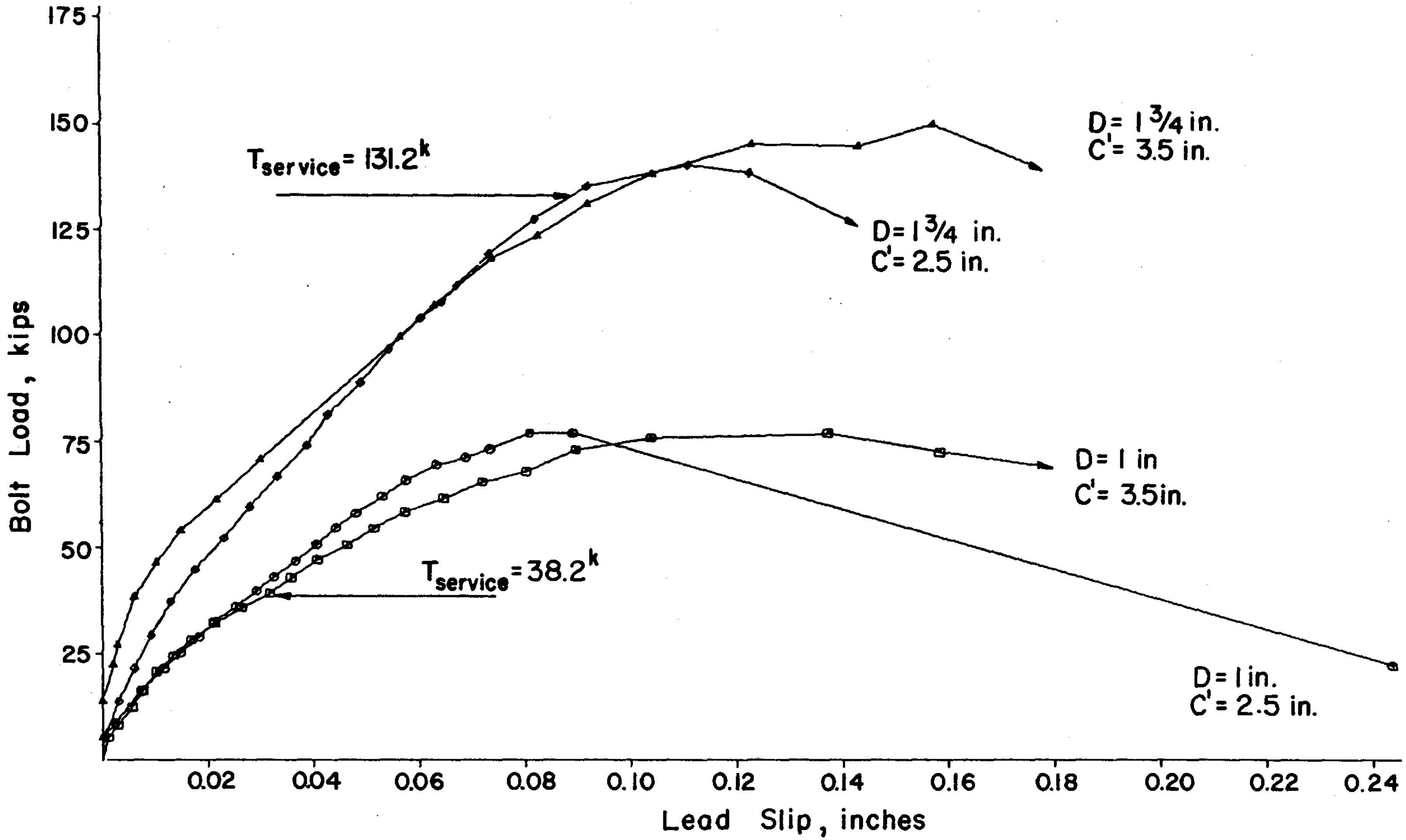


Fig. 5.12 Comparison of load-lead slip curves of bolts with different diameters

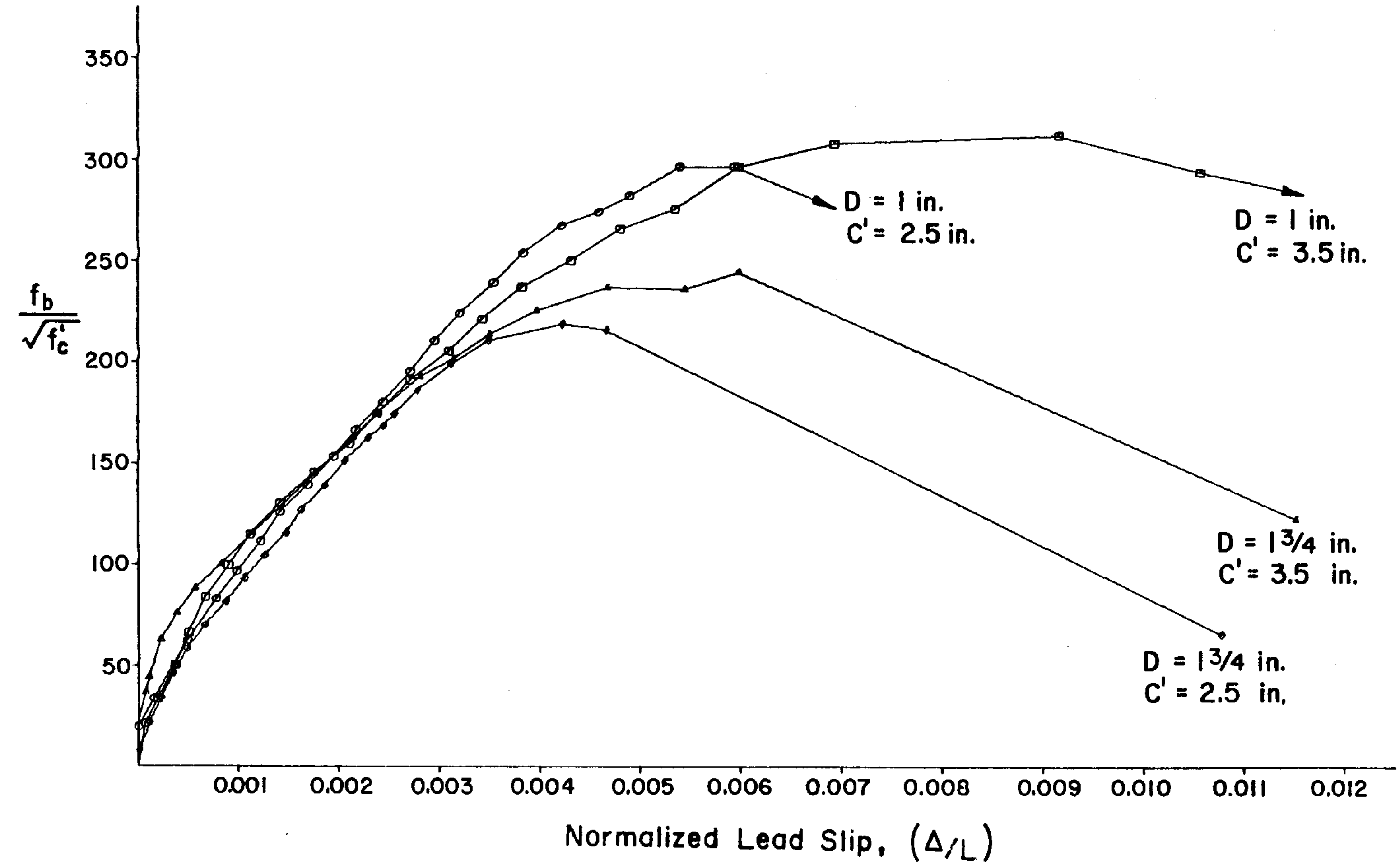


Fig. 5.13 Comparison of bearing stress-lead slip curves of bolts with different diameters

The higher ultimate stress for 1 in. bolts, as indicated in Fig. 5.11, is characteristic of all the 1 in. bolts tested. Figure 5.13, however, indicates that the difference in ultimate strength between the two bolt diameters is not so much a function of bolt diameter but rather some function of washer size and clear cover.

5.5 Effect of Bearing Area

In an attempt to determine the effect of changing bearing area, several model and full-scale tests were performed with different diameter washers. Standard washer diameter for the 1-3/4 in. bolts was 4.0 in. and standard washer diameter for the 1/2 in. bolts was 1.375 in. Figure 5.14 shows stress-slip curves for the four 1-3/4 in. bolts with nonstandard washers and the comparison 1-3/4 in. bolt with a standard diameter washer. Figures 5.15 and 5.16 show similar curves for two series of model tests with varying washer diameters for two different clear covers. It should be noted that the concrete for the 3.0 in. diameter and the 5.0 in. diameter washer tests in Fig. 5.14 was of relatively poor quality, which apparently had a strong influence on the 3.0 in. diameter washer test.

It was found that the large diameter washers generally were not as efficient as smaller diameter washers. It was observed in the model tests that the 1.75 in. diameter washers were bent backwards around the nuts of the anchorage devices. Therefore, it can be concluded that it is necessary to provide an anchorage device that is stiff enough to prevent excessive deformations of the washer which would prevent the washer from being fully effective in bearing. A limit on washer diameter and/or on washer thickness is implied.

It was not possible to quantify the effects of the bearing area using only the tests described above. Varying the washer diameter while maintaining a constant clear cover to the bolt also changed the thickness of cover over the anchorage device. The extent and manner in which the clear cover is mobilized to resist wedge-splitting is apparently dependent on a complex interaction between clear cover,

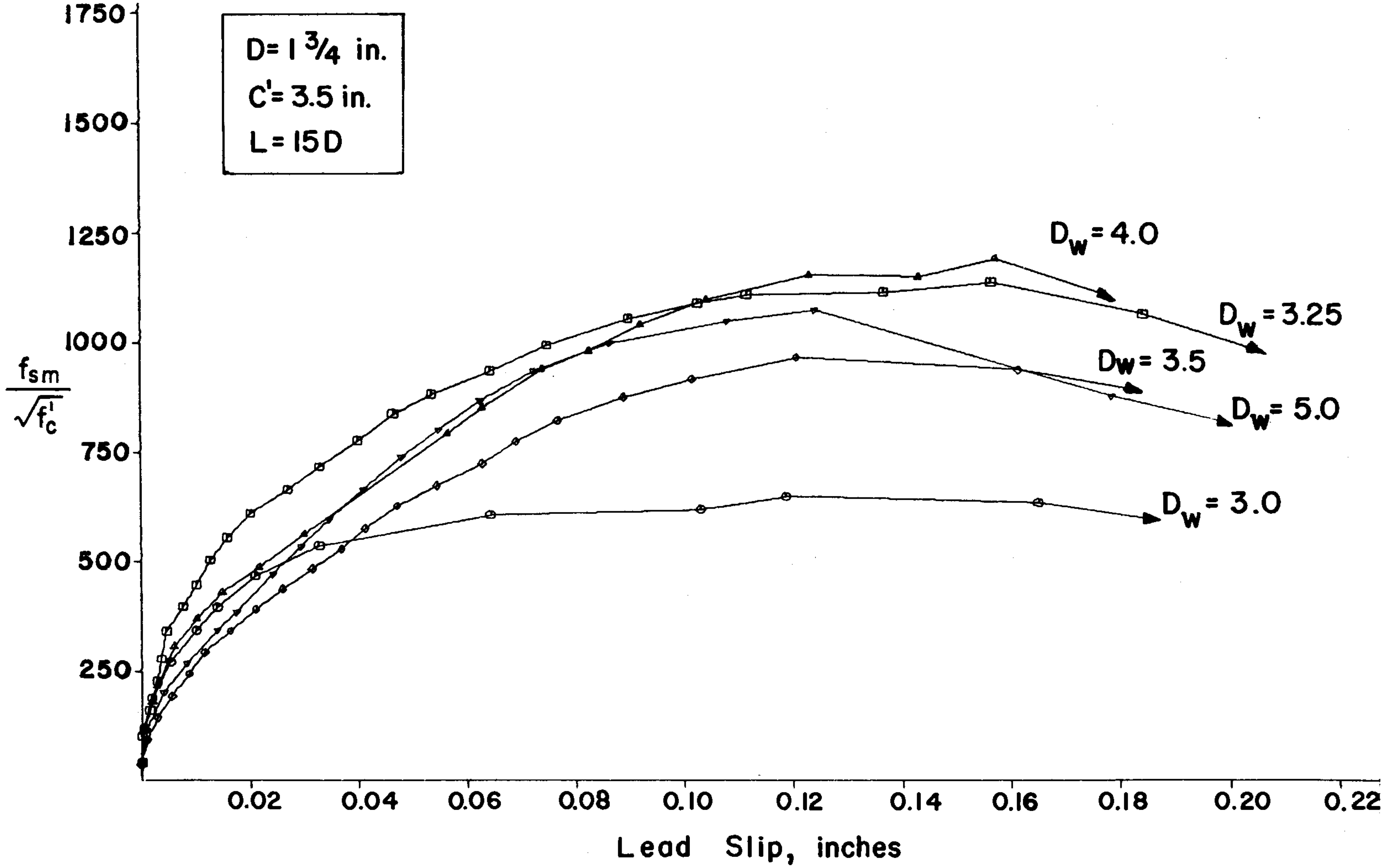


Fig. 5.14 Effect of varying washer diameter--full-scale tests

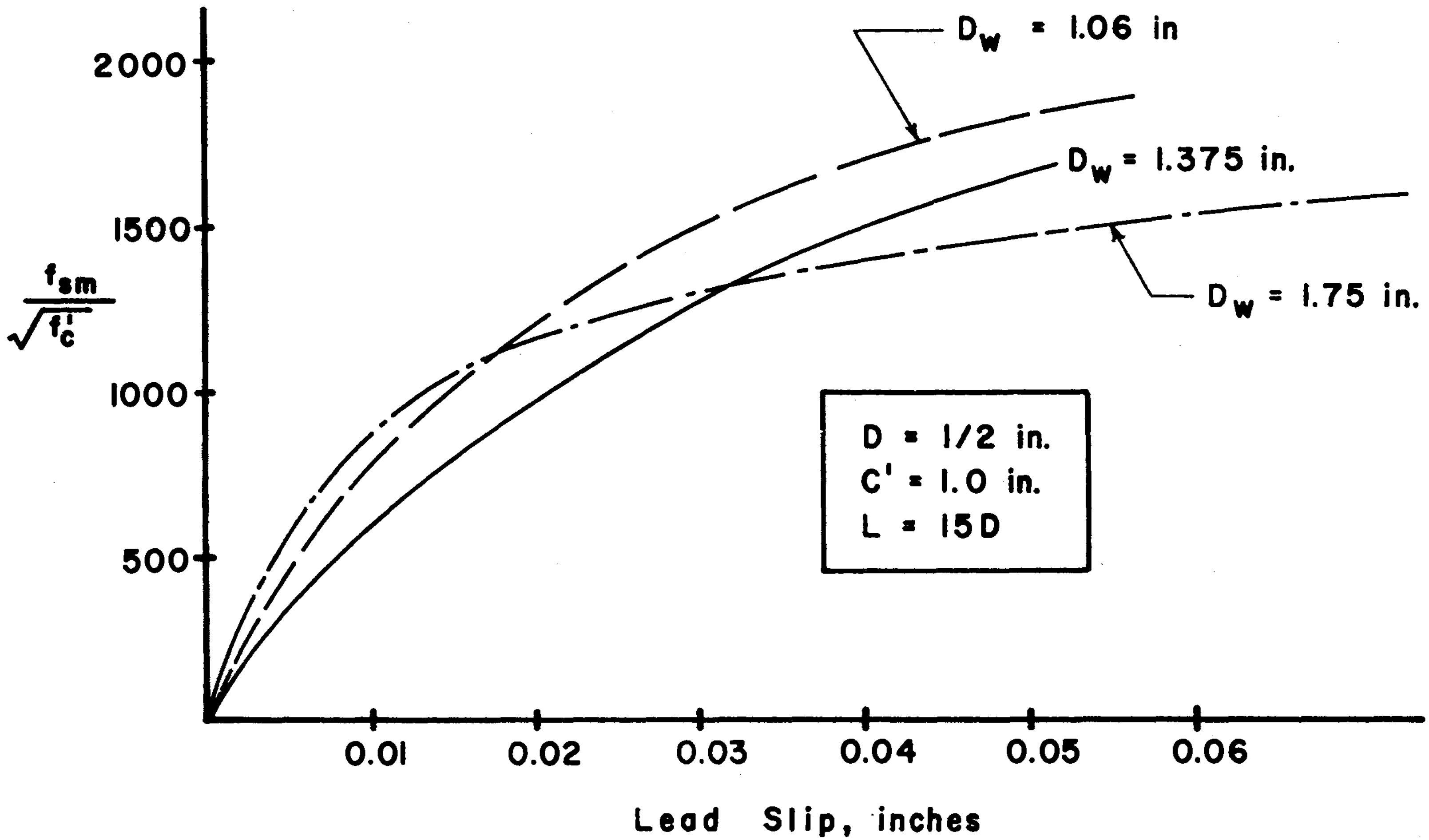


Fig. 5.15 Effect of varying washer diameter--model tests (C′ = 1.0 in.)

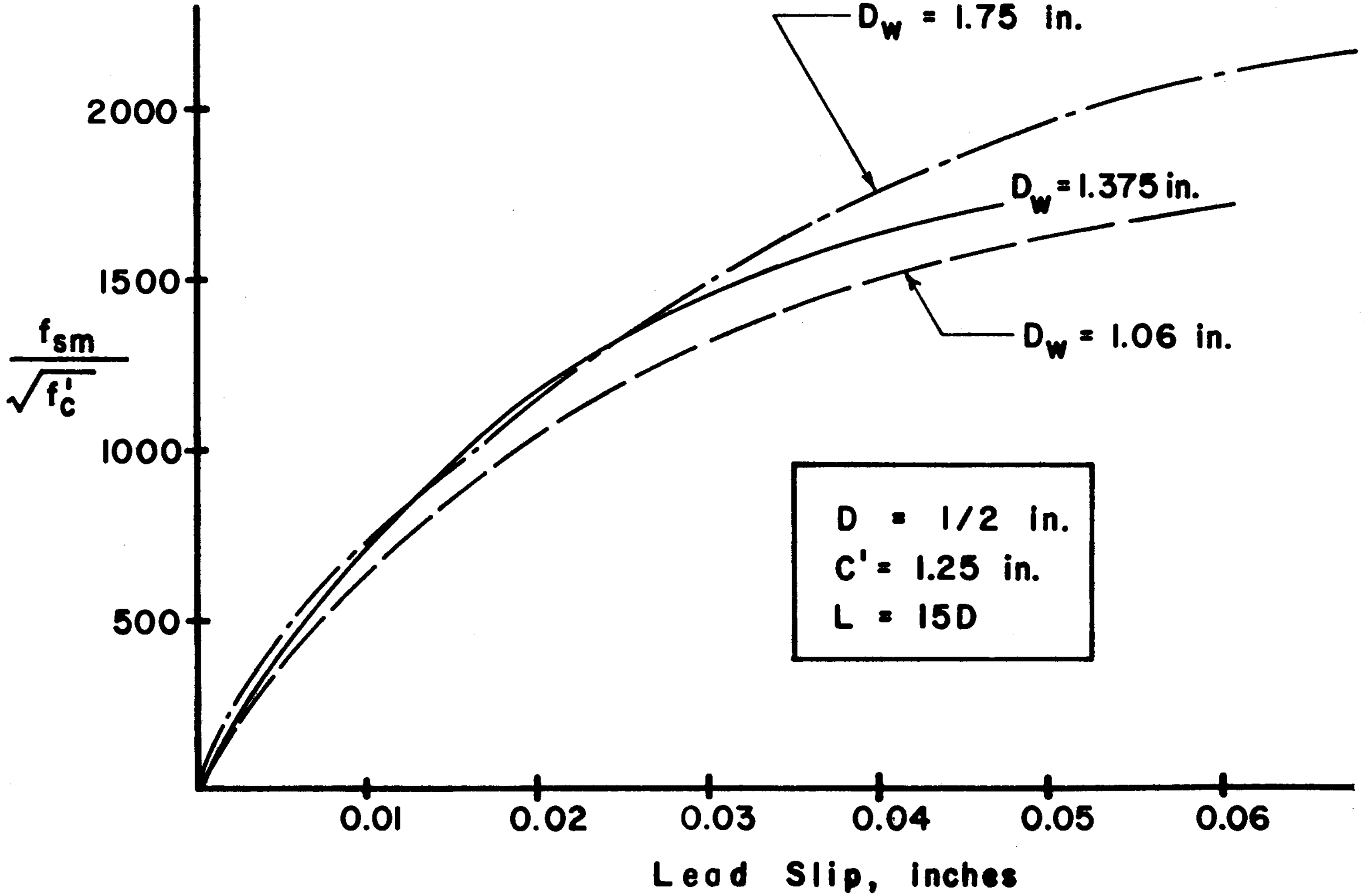


Fig. 5.16 Effect of varying washer diameter--model tests (C′ = 1.25 in.)

washer diameter, and bolt diameter. Only after a statistical analysis was performed on all the available test data, as described in detail in Chapter 7, was it possible to account for this interaction.

5.6 Summary of Effects of Primary Variables

Clear cover was of major importance in the development of the ultimate strength of an anchor bolt installation. If all other factors remain constant, an increase in clear cover generally resulted in an increase in ultimate strength. The ability of the layer of concrete cover to resist wedge-splitting is obviously a function of the thickness of that layer.

Embedment length affected ultimate strength indirectly. An embedment length of 10 bar diameters resulted in a reduced ultimate strength, but no significant difference in ultimate strength was observed between bolts with embedment lengths of 15 and 20 bar diameters. A certain minimum embedment length was required for the concrete cover to develop its full resistance to wedge-splitting, and lengths greater than this minimum did not appear to significantly affect the ultimate strength. For the high-strength bolts tested in this study, an embedment length of 15 bar diameters can be considered a satisfactory minimum.

Comparison of 1 in. and 1-3/4 in. bolts with standard diameter, 1/2 in. thick washers, defined as standard anchor bolt installations, indicated that the 1 in. standard installations consistently reached higher ultimate stresses, and exhibited significantly greater stiffness than the 1-3/4 in. standard installations. It was shown, however, that the difference in stiffness was due to the different relative washer sizes and that the difference in ultimate strength was due to some function of clear cover and washer size.

CHAPTER 6

RESULTS OF EXPLORATORY TESTS

6.1 Introduction

In order to identify specific problem areas, a series of full-scale tests was performed to determine the effects of (1) low-cycle repeated loads, (2) transverse reinforcement, (3) lateral loads, and (4) groups of bolts.

6.2 Effects of Cyclic Loading

One 1-3/4 in. bolt and one 1 in. bolt were subjected to low-cycle repeated loading and performance was compared to similar bolts loaded monotonically to failure.

Figure 6.1 shows mean steel stress, normalized with respect to $\sqrt{f'_c}$, plotted against lead slip for the cyclically loaded 1 in. bolt and a monotonically loaded bolt. Both had clear cover of 2.5 in. and embedment length of 15D. The first twenty cycles of load were applied at a peak mean steel stress of approximately 40 percent of the ultimate material strength of the bolt. During this first set of loading cycles, no concrete cracking occurred. The load was then increased until concrete cracking was observed and another twenty cycles of load were applied. No further concrete cracking occurred beyond that which was observed at the beginning of the second set of loading cycles. The bolt was then loaded to failure. Although cracking characteristic of a wedge-splitting failure began to develop during this last stage of loading, the bolt reached its ultimate material strength. Due to a lower concrete strength, the monotonically loaded 1 in. bolt failed by wedge-splitting at a mean steel stress

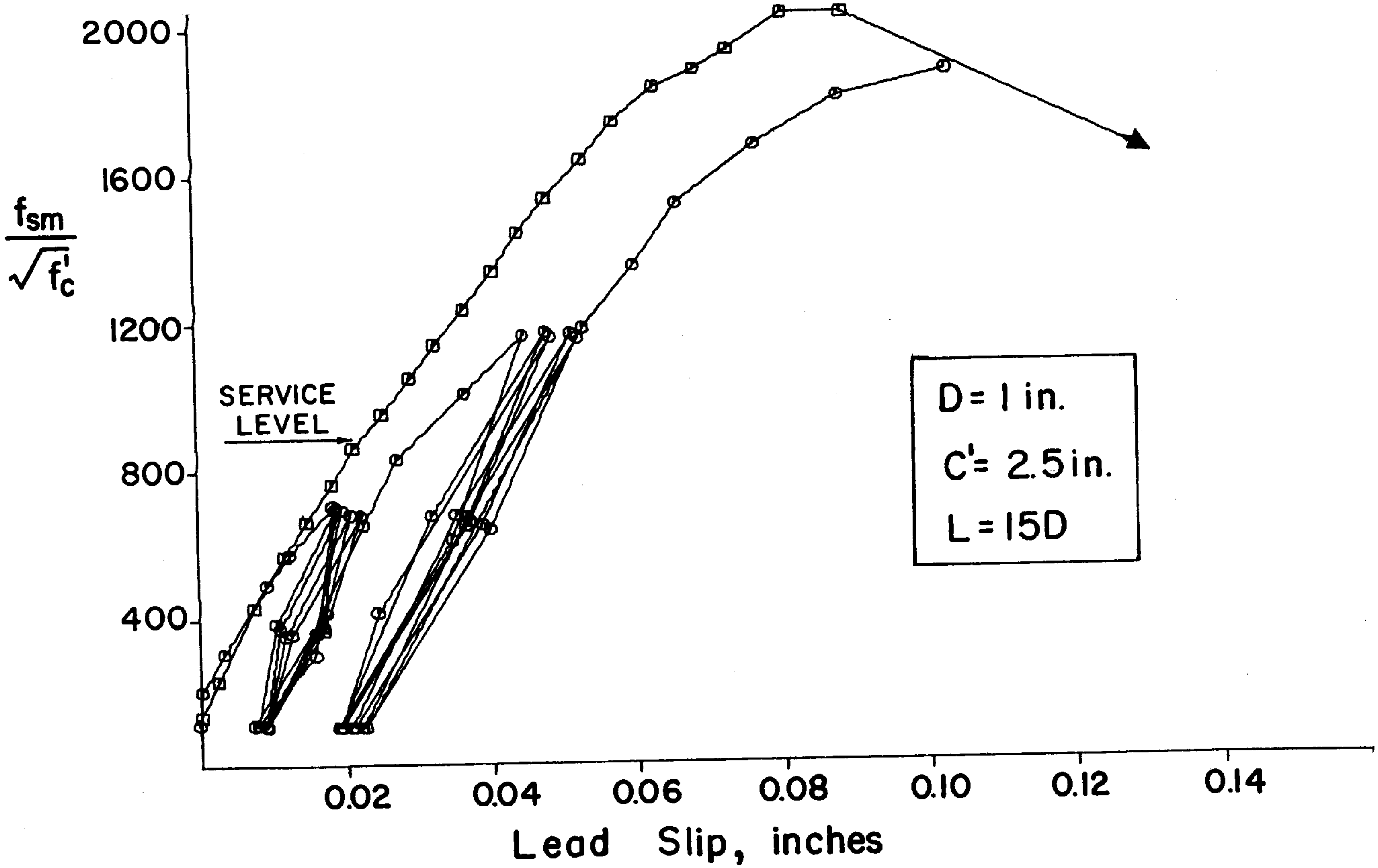


Fig. 6.1 Effect of low-cycle repeated loading on a 1 in. bolt

close to the ultimate material strength. As shown in Fig. 6.1, there was only a slight accumulation of lead slip at the peak of each cycle.

Assuming a service stress of $0.6F_y$, where F_y is the minimum required yield strength of ASTM A193, Grade B7 material = 105 ksi, the service stress for the 1 in. bolt subjected to repeated loading as shown in Fig. 6.1 is between the two sets of loading cycles. It can be seen that cyclic load at a level slightly above assumed service stress did not significantly affect the strength of the anchor bolt installation, although there seems to have been a slight reduction in stiffness near ultimate.

Figure 6.2 shows the stress-slip curves for a cyclically loaded 1-3/4 in. bolt and a monotonically loaded bolt with clear cover of 3.5 in. and embedment length of 15D. The first twenty-five cycles were applied after cracking in the concrete, as shown in Fig. 6.3(a), was first observed. In general, there was some slight accumulation of lead slip at the peak of each cycle, but no further concrete cracking was observed. The load was then increased until cracking characteristic of wedge-splitting, as shown in Fig. 6.3(b), began to develop and another twenty-five cycles of load were applied. Figure 6.3(c) shows that concrete cracking at the end of the second set of load cycles had progressed slightly. In Fig. 6.2 it was seen that the lead slip was accumulating progressively with little recovery at the peak of each cycle. The bolt was then loaded until it failed by wedge-splitting, as shown in Fig. 6.3(d). The comparison 1-3/4 in. bolt also failed by wedge-splitting at a slightly higher normalized mean steel stress.

The assumed service stress of $0.6F_y$ is shown in Fig. 6.2 for the 1-3/4 in. bolt subjected to repeated loading. This service stress was 81 percent of the actual ultimate strength of the anchor bolt installation and corresponds to the peak mean steel stress reached in the second set of loading cycles. It can be seen in Fig. 6.2 that although the ultimate strength of the bolt subjected to fifty cycles

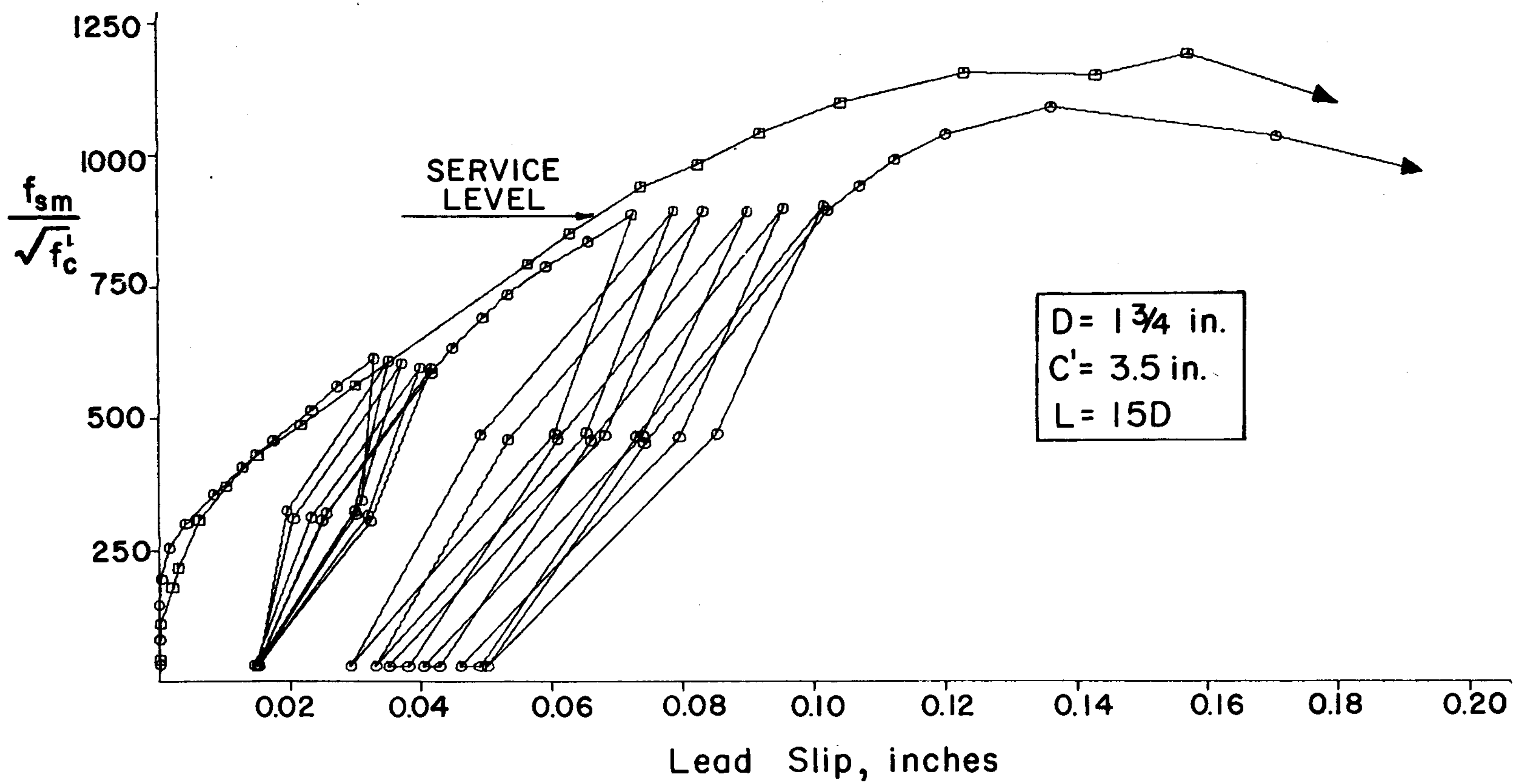
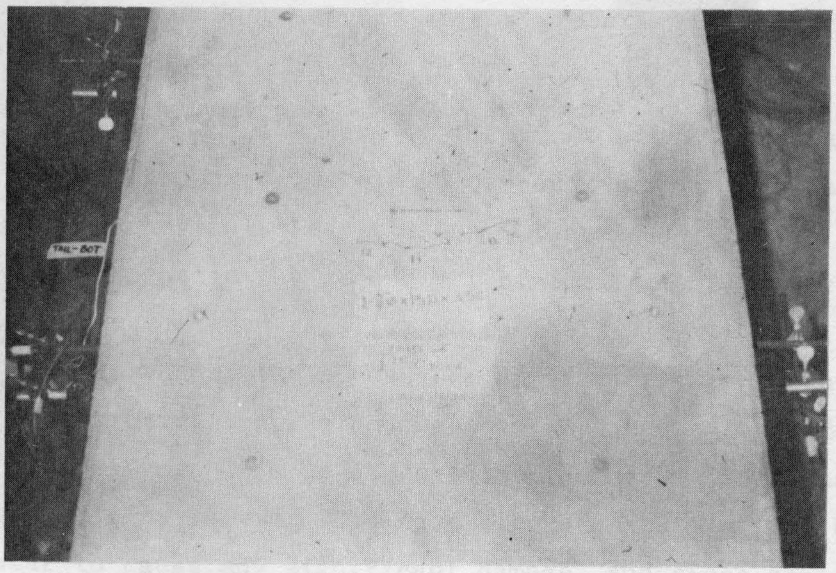
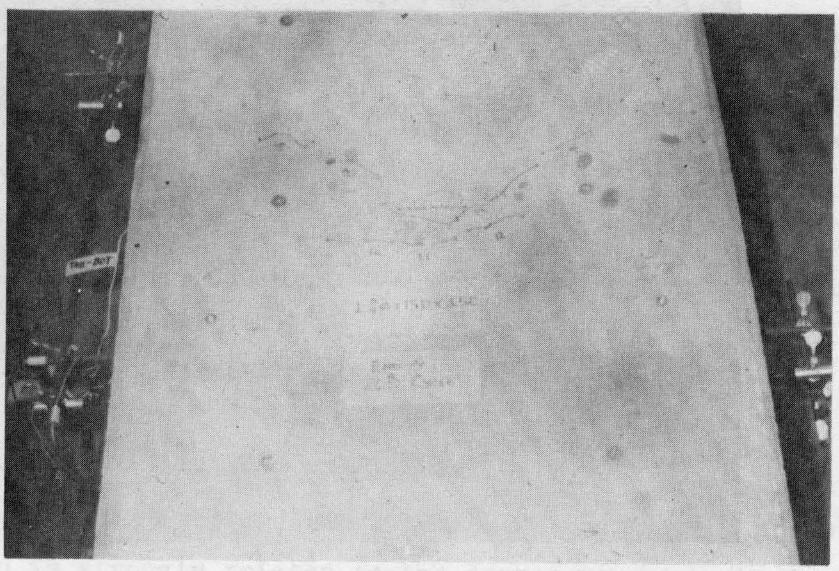


Fig. 6.2 Effect of low-cycle repeated loading on a 1-3/4 in. bolt

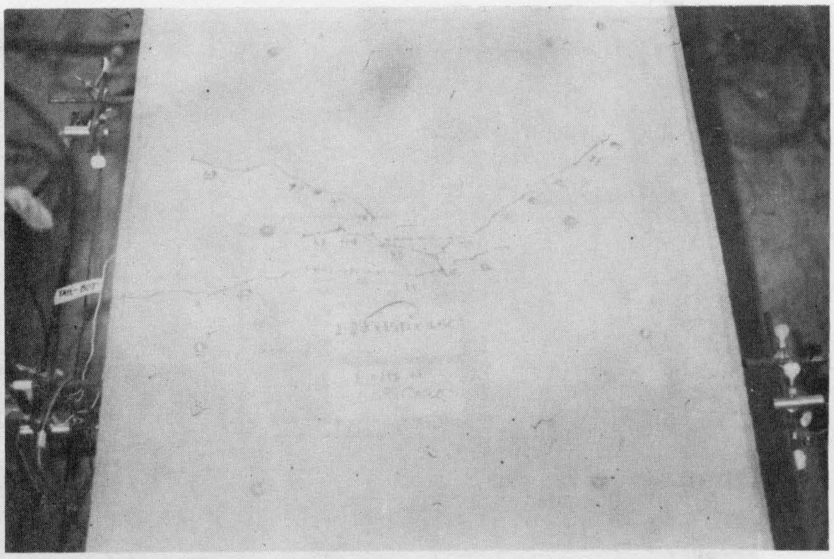


(a) Cracking at start of first set of loading cycles

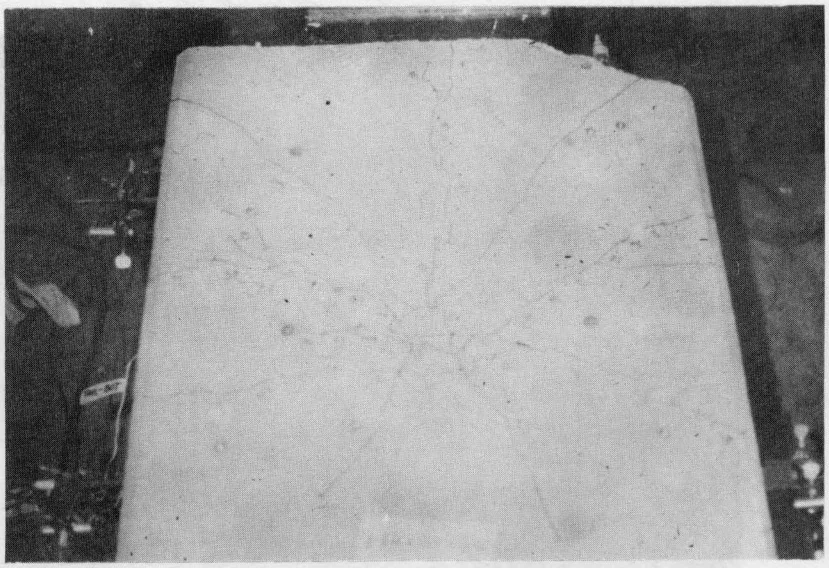


(b) Cracking at end of first set of loading cycles

Fig. 6.3 Sequence of cracking for 1-3/4 in. bolt subjected to cyclic loading



(c) Cracking at end of second set of loading cycles



(d) Cracking at failure

Fig. 6.3 (Continued)

of repeated loading is not significantly different from the ultimate strength of the comparison anchor bolt installation, continued cyclic loading at the assumed bolt service stress probably would have resulted in a premature failure.

The peak mean steel stress of the first set of loading cycles shown in Fig. 6.2 was approximately 56 percent of the actual ultimate strength of the anchor bolt installation. Therefore, just as the peak mean steel stress in the second set of loading cycles corresponds to a service stress defined relative to the bolt yield stress, the peak mean steel stress of the second set of loading cycles can be considered as approximately corresponding to a service stress defined relative to the actual ultimate strength of the anchor bolt installation. Since the first set of loading cycles on the 1-3/4 in. bolt did not appear to cause any significant damage, and since repeated loading at a level slightly above bolt service stress did not significantly reduce the strength of the 1 in. anchor bolt installation, it can be concluded that low-cycle repeated loading at service loads has no significant effect on the performance of an anchor bolt.

It is obvious, however, that service loads must be defined in terms of the actual ultimate strength of the anchor bolt installation and therefore the designer must know whether or not a particular installation can develop the full material strength of the bolt. Clearly, the 1-3/4 in. anchor bolt installation illustrated in Fig. 6.2 would not only have insufficient overload capacity beyond service load, but would be almost certain to experience a premature failure if subjected to repeated loading.

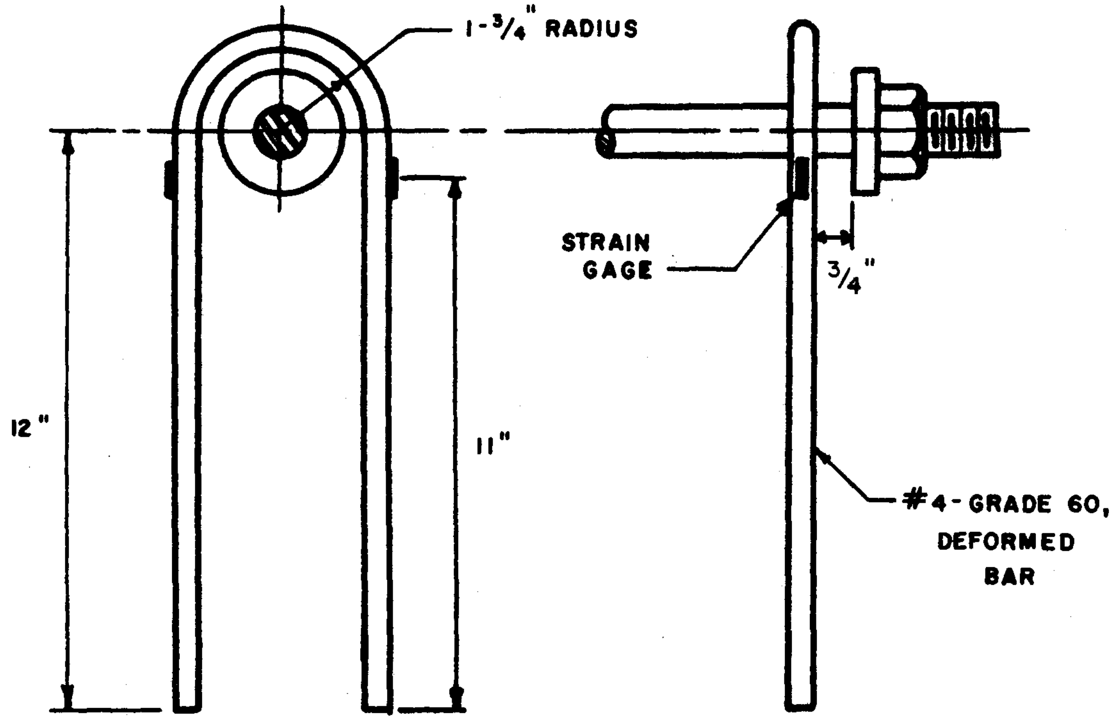
6.3 Effect of Transverse Reinforcement

As discussed earlier, the ultimate strength of an anchor bolt installation is strongly related to the degree to which the layer of clear cover over the bolt is able to resist wedge-splitting. Two tests were performed to explore the feasibility of using transverse reinforcement along the anchor bolt to compensate for relatively shallow layers of clear cover which would otherwise limit the strength of the installation.

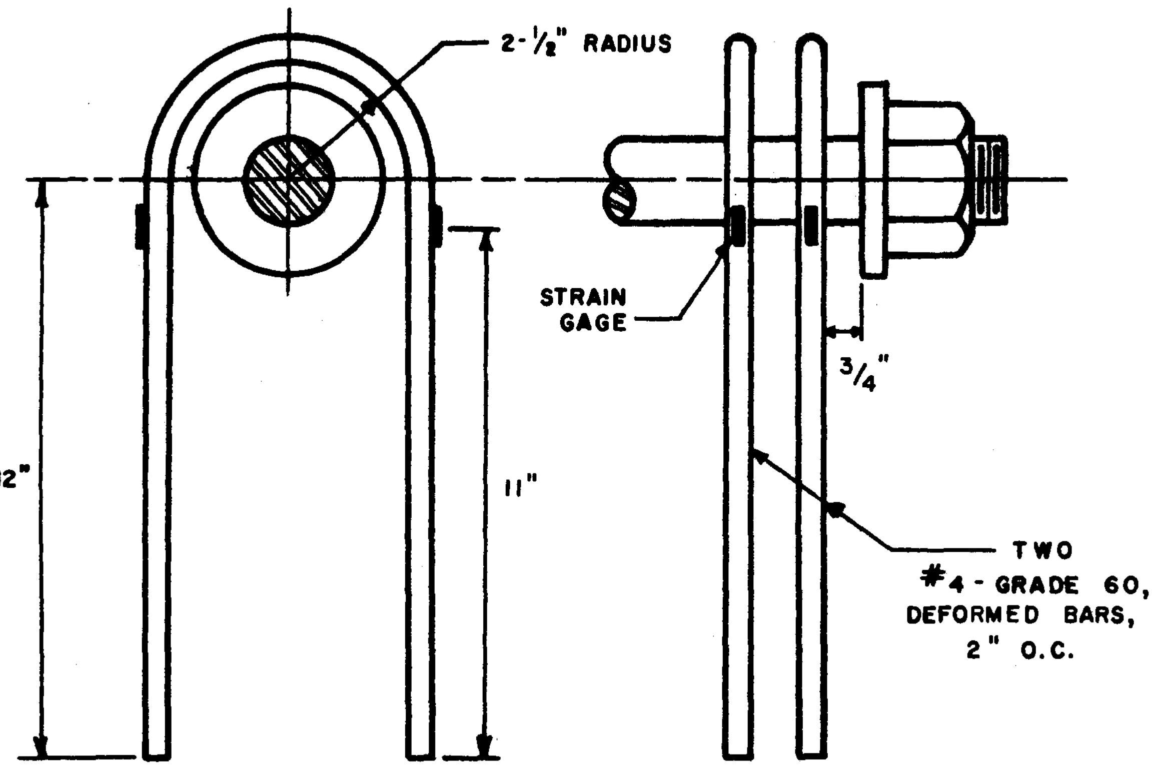
One 1 in. bolt and one 1-3/4 in. bolt were tested with transverse reinforcement along the anchor bolt in front of the anchorage device. The longitudinal reinforcement of the test specimen for these two tests and for the comparison 1 in. and 1-3/4 in. bolts was placed away from the anchor bolts, as shown in Fig. 5.3(b), in order to isolate the effect of the transverse reinforcement. Figure 6.4 shows the details of the transverse reinforcement, which was in the form of #4 Grade 60 reinforcing bars bent into the shape of hairpins. Paper-backed electrical-resistance strain gages with a gage length of 0.64 in. were mounted on the legs of the hairpins, which had sufficient embedment to develop yield. Figure 6.5 shows the hairpin reinforcement in place around the anchor bolts prior to casting. It should be noted that considerable difficulty was encountered in securing the hairpins in the proper position in the forms and special care was required during casting to avoid displacing the hairpins.

The 1 in. bolt with transverse reinforcement and the 1 in. bolt without transverse reinforcement both reached their ultimate material strength. Figure 6.6 shows the crack patterns at failure. As shown in Fig. 6.6(a), the 1 in. bolt without hairpins was beginning to form a characteristic wedge-splitting failure. It can be seen from Fig. 6.6(b) that the presence of transverse reinforcement apparently retarded the development of the wedge-splitting mechanism, but since the anchor bolt installation was able to develop the bolt ultimate material strength without transverse reinforcement, the presence of such reinforcement did not provide any significant improvement.

Figure 6.7 shows the stress-slip curves for the 1-3/4 in. bolts with and without transverse reinforcement. The bolt with hairpins reached about 30 percent higher mean steel stress and an 80 percent larger lead slip at ultimate than that reached by the bolt without the hairpins. Figure 6.8 shows the patterns of



(a) Transverse reinforcement for 1 in. bolt



(b) Transverse reinforcement for 1-3/4 in. bolt

Fig. 6.4 Details of transverse reinforcement

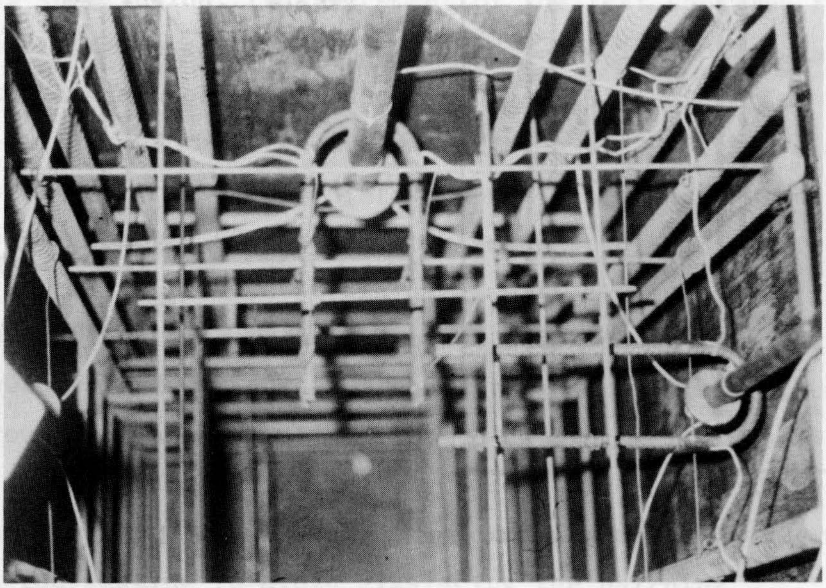
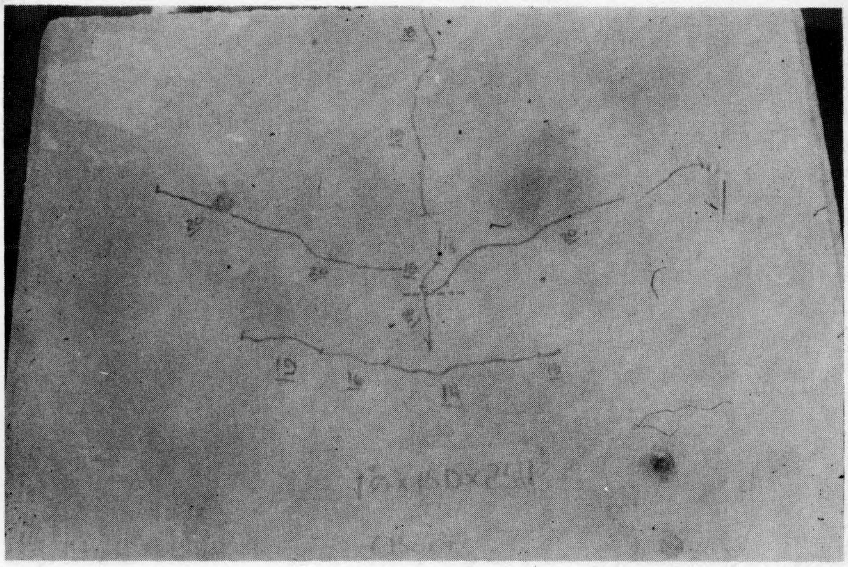
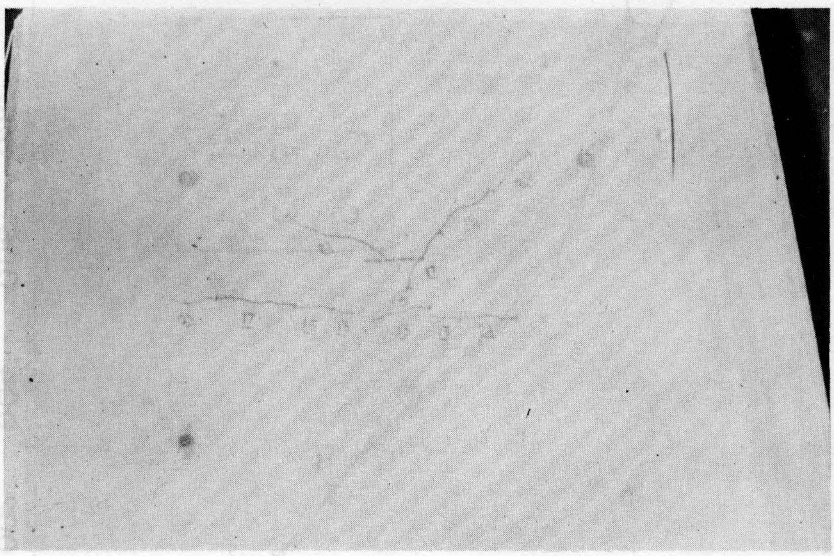


Fig. 6.5 Transverse reinforcement in place prior to casting



(a) Without transverse reinforcement



(b) With transverse reinforcement

Fig. 6.6 Comparison of concrete cracking at failure for 1 in. bolts with and without transverse reinforcement

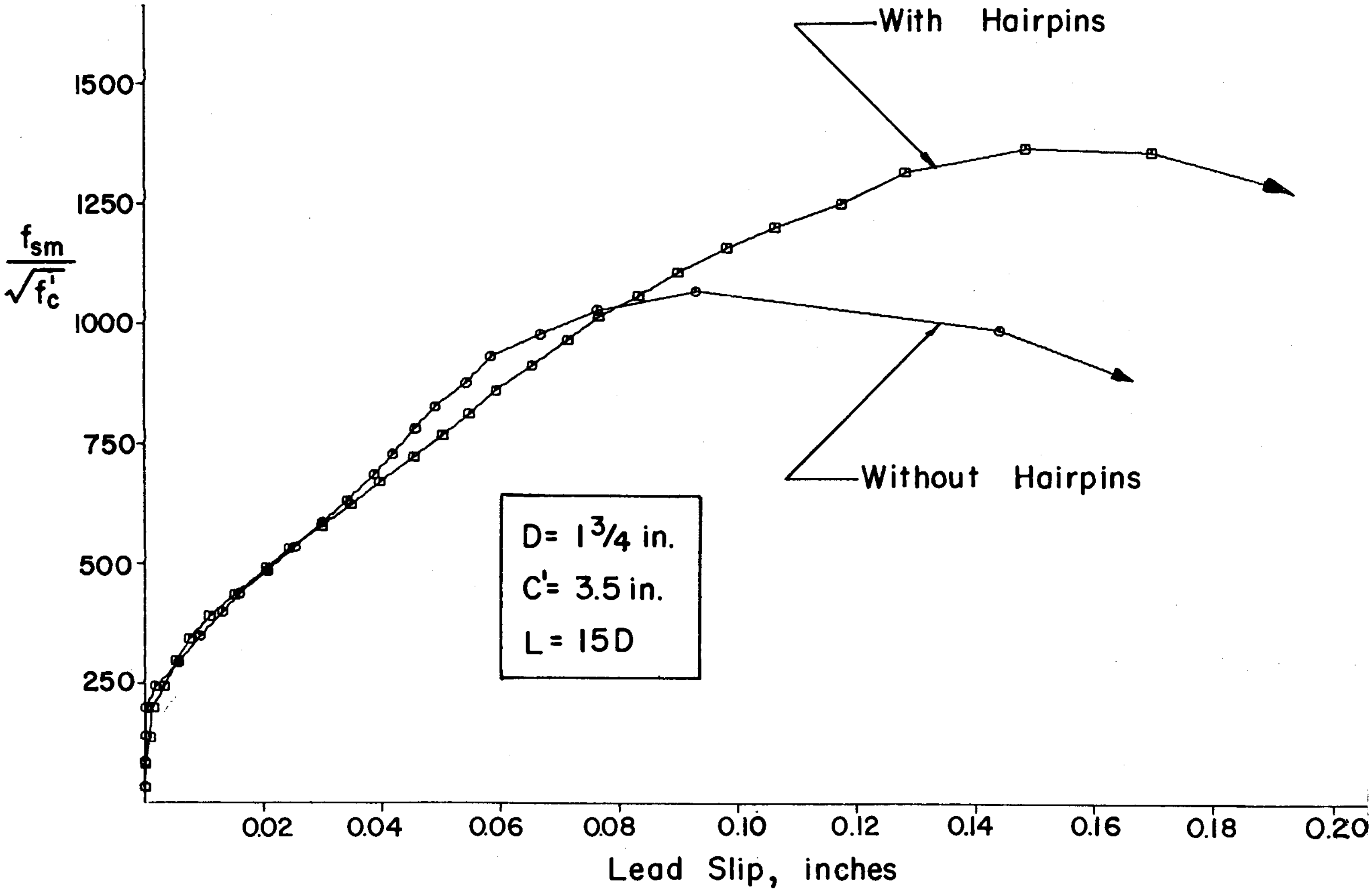
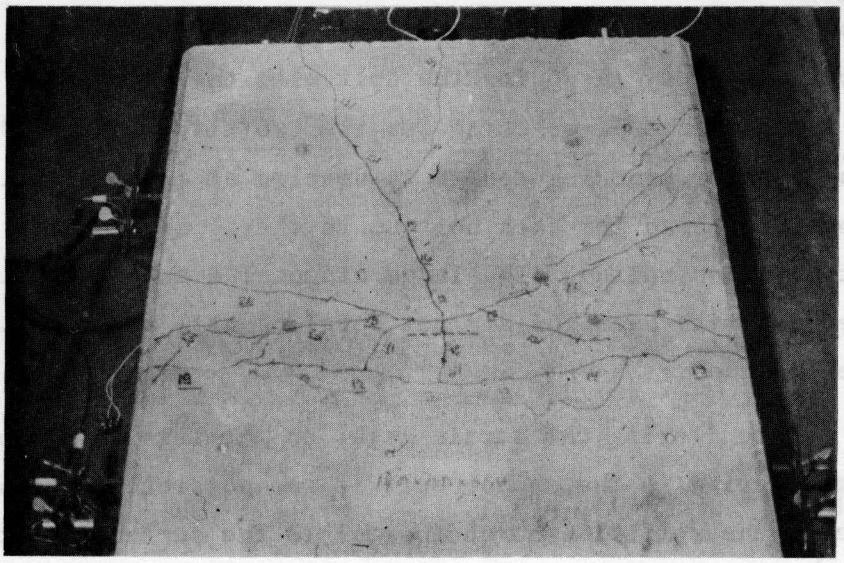
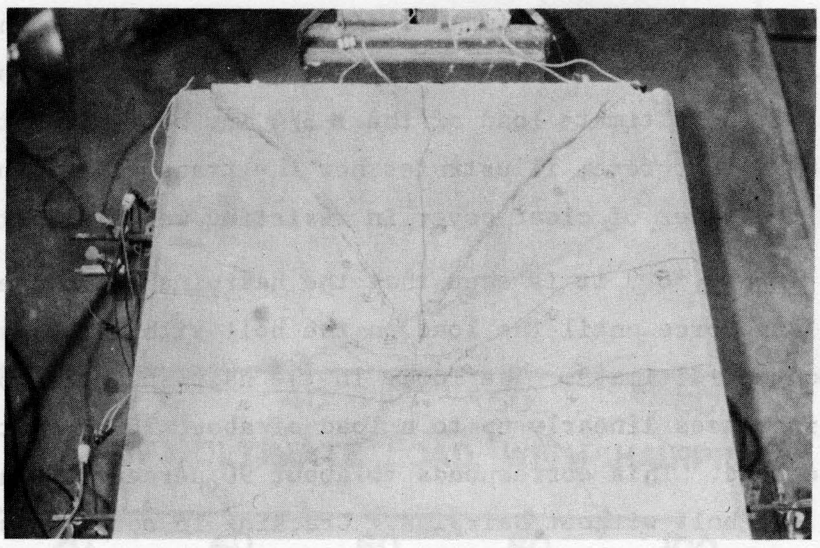


Fig. 6.7 Comparison of lead stress-lead slip curves of bolts with and without transverse reinforcement



(a) Without transverse reinforcement



(b) With transverse reinforcement

Fig. 6.8 Comparison of concrete cracking at failure for 1-3/4 in. bolts with and without transverse reinforcement cracking at failure for the two bolts. Both bolts failed by wedge-splitting. The long diagonal cracks which often accompanied such failures did not form for the bolt without transverse reinforcement, but were very large for the bolt with the hairpins. A comparison of the sequence of cracking for the two tests indicated that the various types of cracking tended to develop at approximately the same levels of load for each bolt up to the load at which the bolt without hairpins failed. The large diagonal cracks shown in Fig. 6.8(b) for the bolt with hairpins did not develop until after ultimate load had been reached.

By monitoring the strain gages on the legs of the hairpins as load was applied to the anchor bolt it was possible to obtain a measure of the splitting force (normal to the surface) at the anchorage device. Figure 6.9 shows the total force in the hairpins (as determined from the strain gages) plotted against bolt load on the 1-3/4 in. bolt with transverse reinforcement. The load is expressed as a percentage of the ultimate load of the bolt. Also shown in Fig. 6.9 is a supplemental axis depicting bolt load expressed as a percent of the ultimate load of the 1-3/4 in. bolt without hairpins. Figure 6.9, therefore, illustrates how the transverse reinforcement assists the layer of clear cover in resisting wedge-splitting.

In Fig. 6.9 it is seen that the hairpins do not begin to develop any force until the load on the bolt with hairpins is about 35 percent of ultimate. The force in the hairpins (the splitting force) increases linearly up to a load of about 70 percent of the ultimate load. This corresponds to about 90 percent of the ultimate load on the bolt without hairpins. Cracking in both tests at this load level, which corresponds to a value of $f_{sm}/\sqrt{f'_c}$ in Fig. 6.7 of about 940, has begun to develop into the characteristic wedge-splitting pattern but is not extensive. It can be seen in Fig. 6.7 that beyond this level of load the lead slip of the bolt without transverse reinforcement begins to increase rapidly with small increases in load. It is apparent that the layer of clear cover has begun to lose its

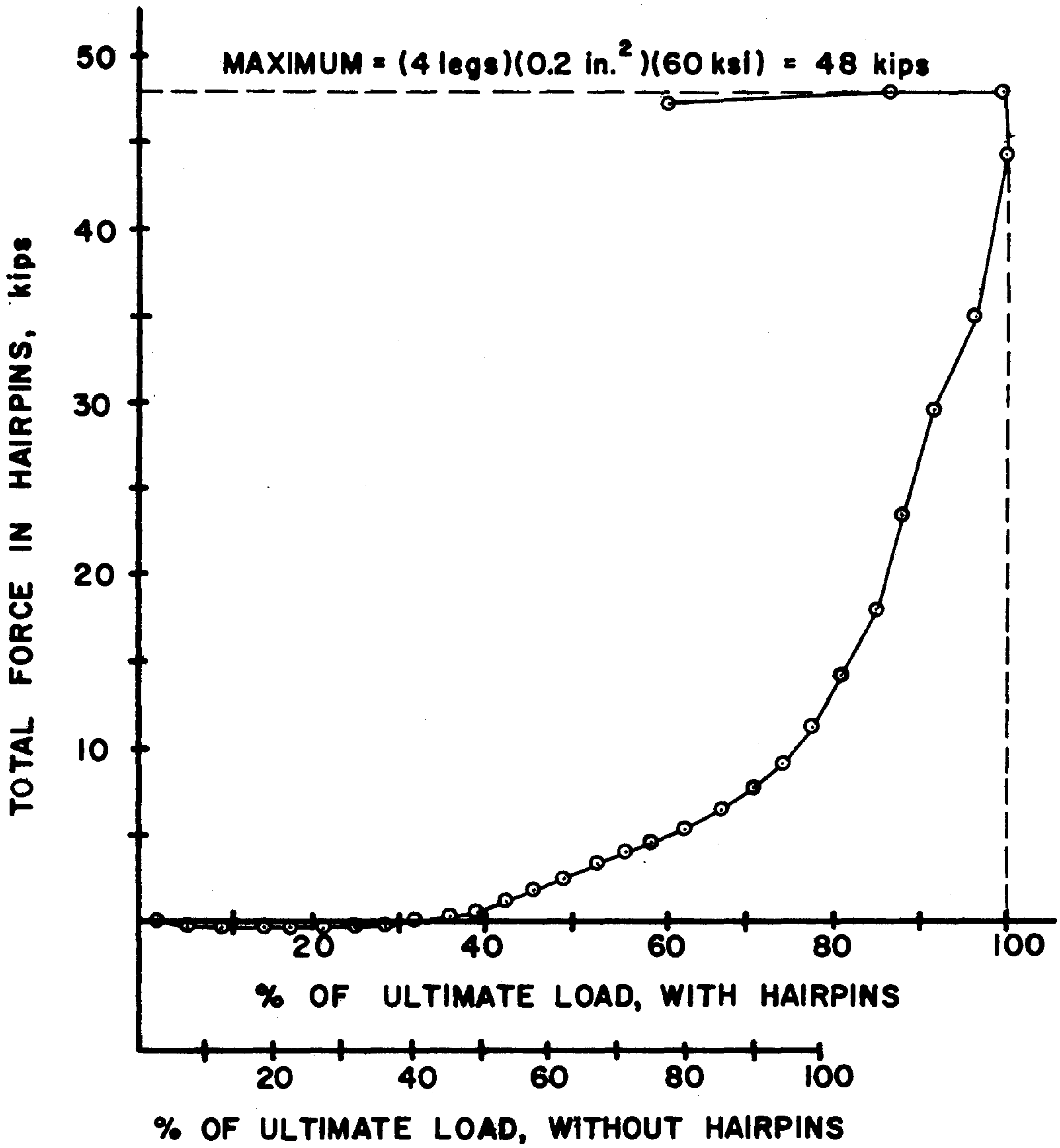


Fig. 6.9 Variation of force in hairpin reinforcement with bolt load

ability to resist wedge-splitting. With further loading of the bolt without transverse reinforcement, cracking develops rapidly and the anchor bolt installation fails. With further loading of the bolt with transverse reinforcement, the rate of change of the force in the hairpins increases rapidly, as shown in Fig. 6.9, indicating that the splitting forces around the anchorage device are being transferred by the legs of the hairpins to the concrete below the anchor bolt. In effect, the hairpins are tying the layer of clear cover to the main body of concrete. Cracking developed slowly until the ultimate load was reached when the hairpins yielded. This phenomenon may not be readily apparent from Fig. 6.9 because the hairpins yielded abruptly between load stages. Continued application of load resulted in the development of extensive cracking and a progressive reduction in actual bolt load. When the large diagonal cracks shown in Fig. 6.8(b) formed, the bolt load continued to decrease rapidly and the force in the hairpins also decreased slightly, as shown in Fig. 6.9.

It can be concluded that the presence of transverse reinforcement can result in a significantly higher ultimate strength and ductility than would otherwise be available with a relatively shallow clear cover. It can be seen that once clear cover splits away from the core the hairpins can be very effective in transmitting the large forces normal to the surface which are developed as the anchor bolt fails.

Obviously, additional research is required before definite recommendations can be made concerning the manner in which transverse reinforcement should be designed, but it seems reasonable to assume that sufficient amounts of transverse reinforcement can adequately compensate for shallow clear cover.

6.4 Effect of Lateral Load

Two 1-3/4 in. bolts were subjected to tension and to different fixed levels of lateral load applied normal to and towards the top surface (edge) of the specimen. The results were compared to a 1-3/4 in. bolt which was subjected to tension only. In all three tests, clear cover was 3.5 in. and the embedment length was 15D. In the first lateral load test, designated with the suffix V1 in Table 2.1, the lateral load corresponded to the load causing first cracking on the front face of the specimen where the bolt protrudes from the concrete. In the second test, designated with the suffix V2 in Table 2.1, the applied lateral load was the maximum force which the specimen would hold and corresponded to an approximate ultimate lateral shear strength of the anchor bolt installation. The average lateral load in the V1 test was 10.6 kips and the average lateral load in the V2 test was 13.3 kips.

In both cases the lateral load initiated cracking at the front face of the specimen, as shown in Fig. 6.10. As bolt tension was increased and the lateral load maintained, the first crack extended toward the sides and top surface of the specimen. Figure 6.11 shows the crack pattern on the front face of the specimen after failure. In both lateral load tests, the cracking on the front face extended along the top surface of the specimen and a longitudinal crack developed at the lead end of the anchor bolt before any cracks were observed near the anchorage device.

The magnitude of the lateral load influenced both the failure mode and the amount of top cover that was damaged by the lateral deformation of the bolt at the front of the specimen. As shown in Fig. 6.12(a), the bolt with the lower lateral load (V1) failed primarily by wedge-splitting. With a higher lateral load (V2), failure was initiated primarily by splitting along the large longitudinal crack over the bolt axis as shown in Fig. 6.12(b). The area of top cover damaged by the lateral load in the V1 test, as shown in

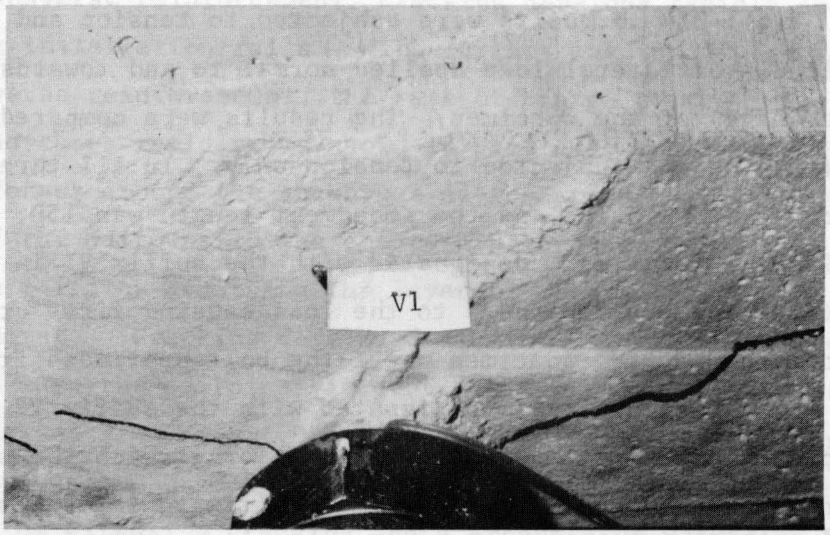


Fig. 6.10 First cracking on front face of specimen due to lateral load

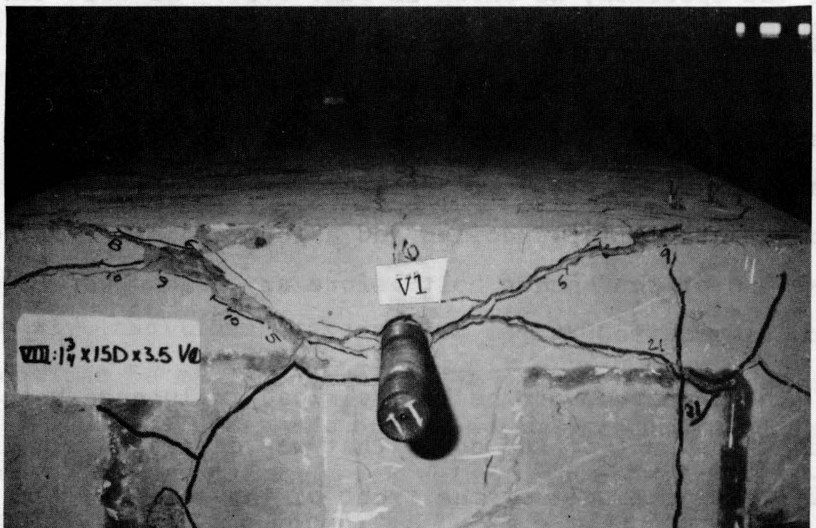
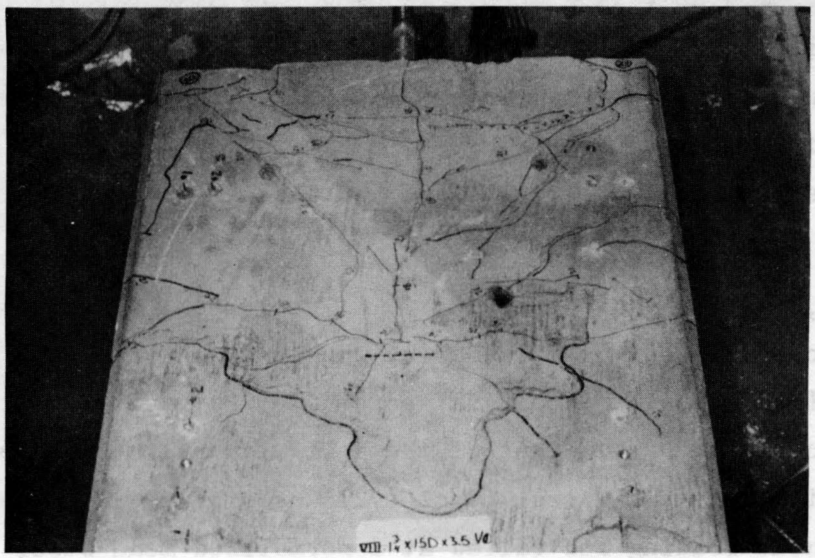
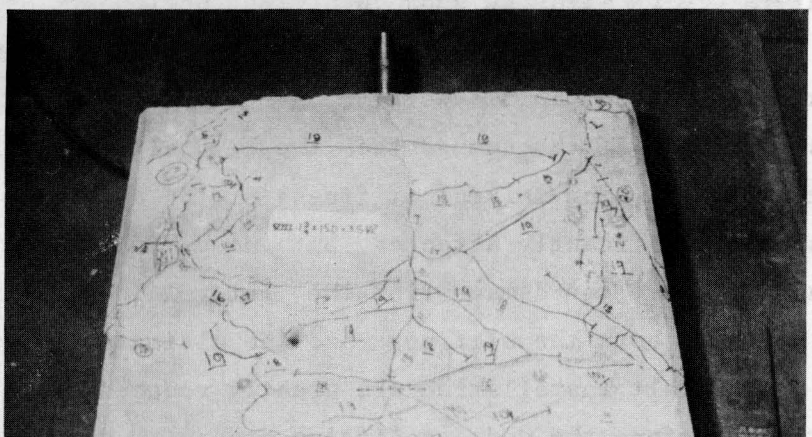


Fig. 6.11 Crack pattern at failure, V1 test



(a) Specimen V1



(b) Specimen V2

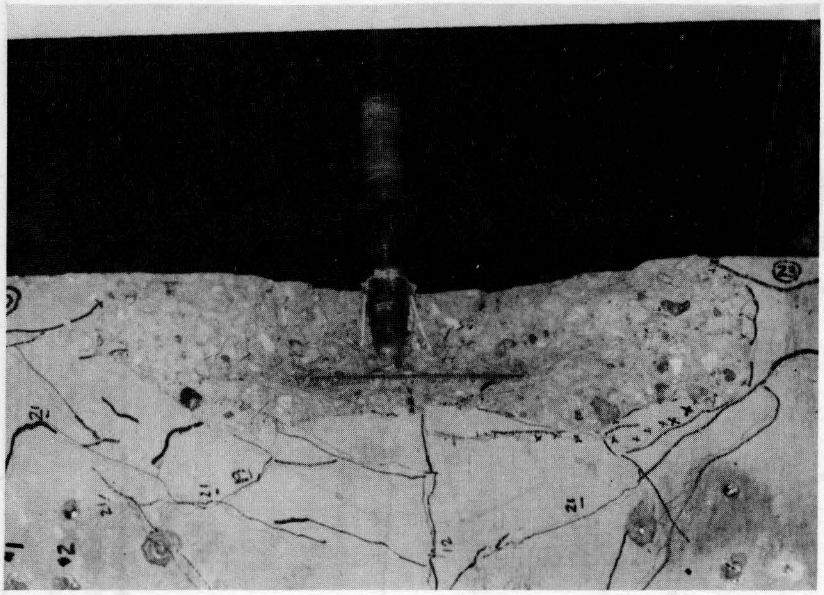
Fig. 6.12 Failure patterns--specimens with lateral load

Fig. 6.13(a), extended along about 30 percent of the bolt embedment length, with some additional cracking extending back along the bolt. The damaged area in the V2 test, shown in Fig. 6.13(b), extended along nearly 70 percent of the bolt embedment length. There was a significant difference between failure modes and between the amounts of damaged cover for a relatively small difference in lateral load.

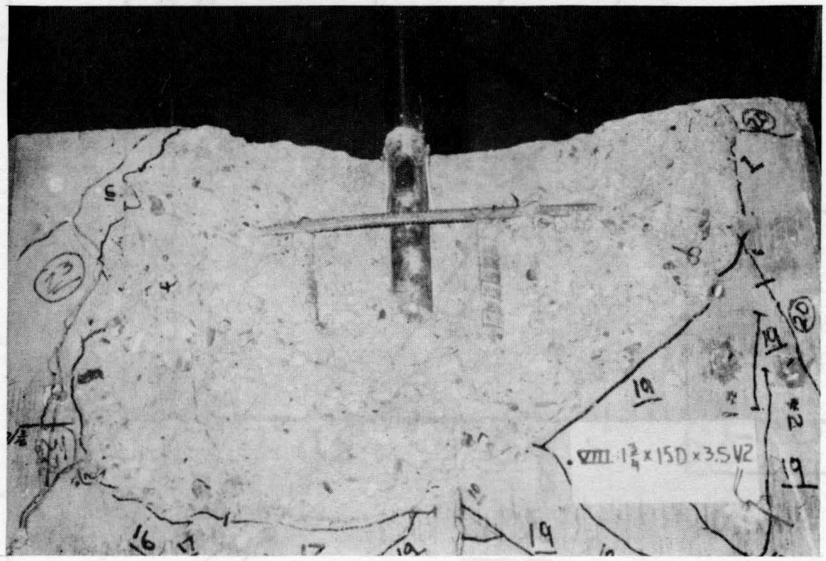
Figure 6.14 shows mean steel stress, normalized with respect to $\sqrt{f'_c}$, plotted against lead slip. The initial stiffness of the bolt with lower lateral load was about the same as for a bolt with no lateral load, but the bolt failed at a normalized mean steel stress 30 percent lower than that of the bolt with no lateral load. The bolt with maximum lateral force (V2) was significantly less stiff throughout loading and failed at a normalized mean steel stress 50 percent less than the bolt with no lateral load.

A comparison of the three tests indicates that the application of a lateral force with the subsequent destruction of the top cover at the front of the specimen and the longitudinal splitting along the bolt axis reduces the ability of the layer of top cover to resist wedge-splitting. If the damage from the lateral load is not extensive, as in the V1 test, the initial stiffness of the anchor bolt installation is not significantly affected, but the ultimate tensile strength is reduced. Where the damage from the lateral load is extensive, as in the V2 test, both the stiffness and the ultimate tensile strength of the anchor bolt installation are greatly reduced as the top cover is destroyed before the wedge-splitting mechanism develops.

It has been shown[11] that the interaction between tension and shear on high-strength bolts in steel joints and connections can be adequately described by an elliptical curve such as that shown in Fig. 6.15. The tensile mean steel stress and the nominal shear stress on the root area of the bolt have been normalized with respect to the ultimate tensile strength of ASTM A193, Grade B7 material = 125 ksi.



(a) V1 test



(b) V2 test

Fig. 6.13 Damage to top cover due to lateral load

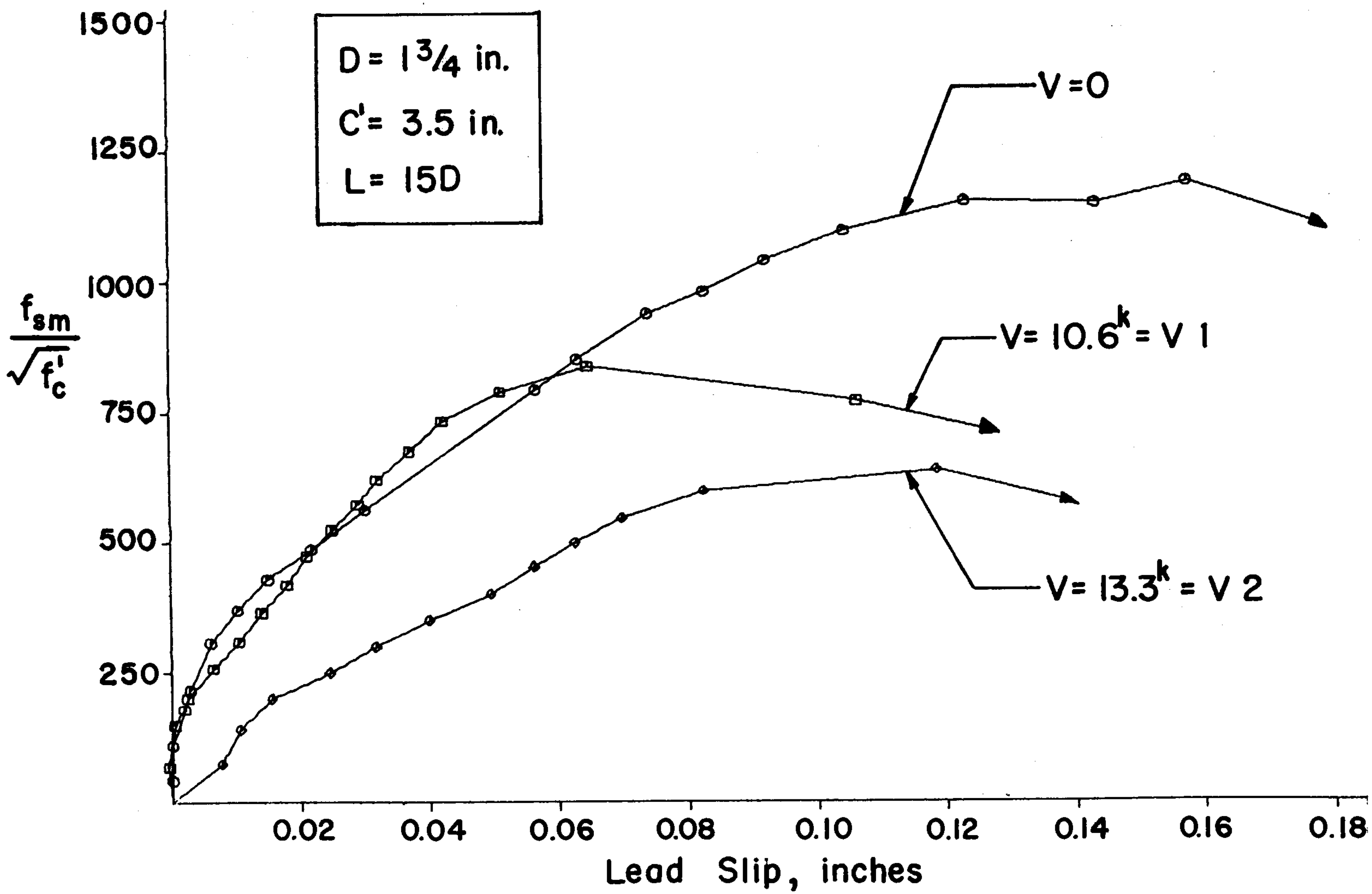


Fig. 6.14 Comparison of lead slip-lead stress curves of bolts subjected to different levels of lateral load

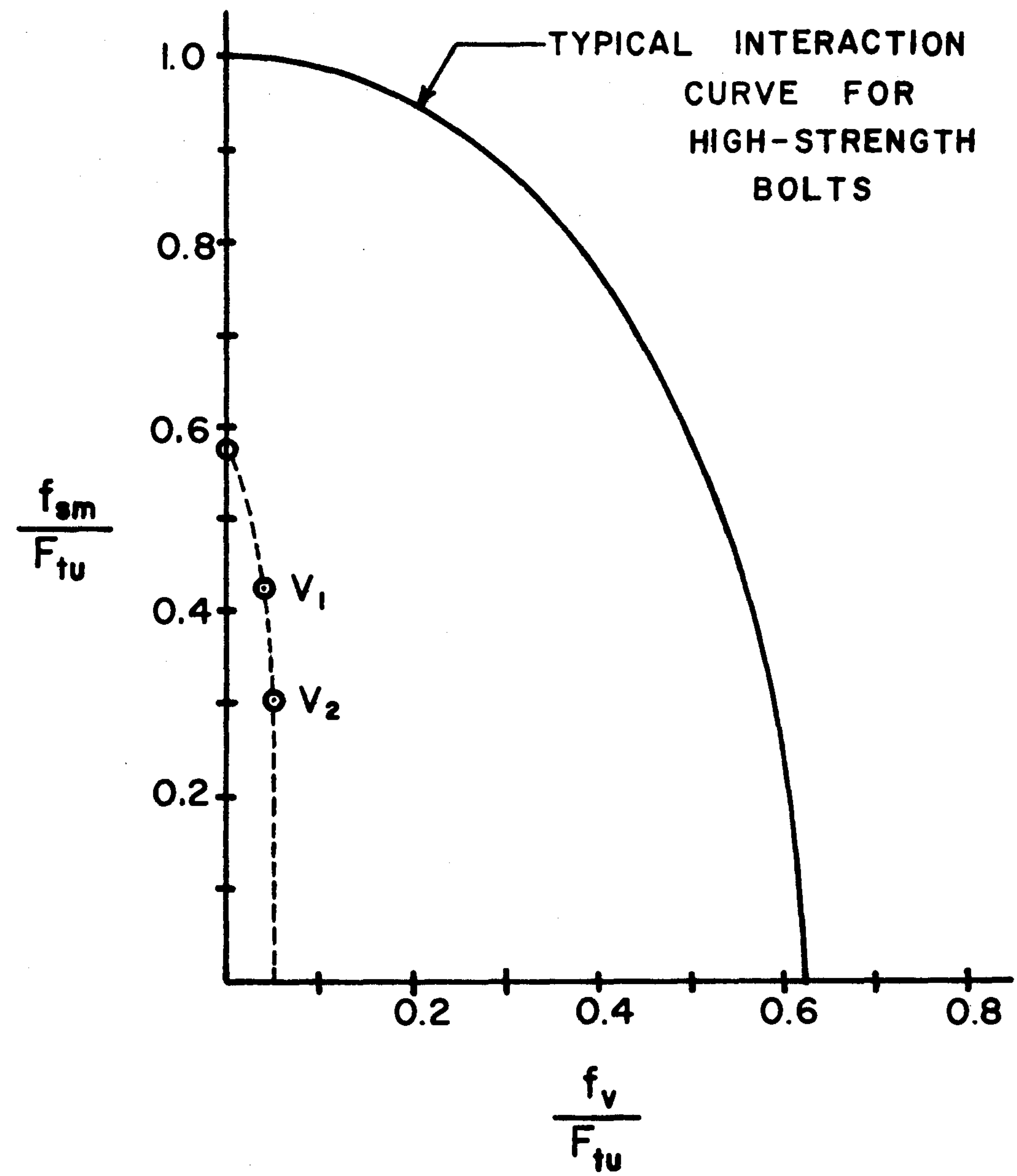


Fig. 6.15 Shear-tension interaction for 1-3/4 in. diameter anchor bolt installation

This type of interaction curve is commonly used to design anchor bolts subjected to combined shear and tension. Such an approach is satisfactory for determining the ultimate strength of the bolt itself, but it is clear that it may not reflect the actual strength of the anchor bolt installation since the interaction curve shown in Fig. 6.15 does not take into account a failure of the concrete over the bolt.

Also shown in Fig. 6.15 are the results of the two lateral load tests and the comparison test with no lateral load. All three bolts experienced a failure of the concrete over the bolt. Therefore, Fig. 6.15 compares the actual ultimate strengths of an anchor bolt installation with different values of lateral load, or bolt shear, with the ultimate strength of the threaded bolt stock as predicted by the elliptical interaction curve.

As shown in Fig. 6.15, the anchor bolt installation could not develop the ultimate tensile strength of the bolt material even with no lateral load. As expected, the application of a lateral load to the bolt caused a further reduction in the ultimate strength of the anchor bolt installation, but this reduction is not proportionally the same as the reduction in the ultimate tensile strength of a high-strength bolt caused by the application of a shear force. It is clear that the lateral force (shear) had a far more significant effect on the strength of an anchor bolt installation than on the strength of the bolt stock.

It can be concluded that the application of a lateral force normal to and in the direction of the outer face of the specimen results in a significant reduction of the ultimate tensile strength and, if the lateral force is large enough, of the stiffness of an anchor bolt installation. Although more research is necessary to determine the effect of clear cover on the ultimate shear strength of an anchor bolt installation, it is clear that lateral loads which cause longitudinal splitting can have a detrimental effect on the strength of an anchor bolt installation which cannot be adequately accounted for with the typical elliptical interaction curves used for the design of high-strength bolts subjected to combined shear and tension.

6.5 Behavior of Bolts in a Group

While nearly all of the past research on anchor bolts has involved the testing of single bolts, most practical applications involve groups of bolts. It can be assumed that if bolts are spaced closely together, there is likely to be some interaction between the individual bolts in the group. Likewise, it can be assumed that there is some critical spacing beyond which the bolts in a group behave like single bolts with no interaction. In an effort to determine the nature of the interaction between bolts in a group, a series of two-bolt groups with 1 in. bolts on 5 in., 10 in., and 15 in. center-to-center spacings were tested. All three groups had a clear cover of 2.5 in., an embedment length of 15D, and a standard end anchorage consisting of a nut and a 1/2-in. thick, standard diameter washer.

Figure 6.16 shows mean steel stress, normalized with respect to $\sqrt{f'_c}$, plotted against lead slip for a single 1 in. bolt with a clear cover of 2.5 in. and an embedment length of 15D which experienced a wedge-splitting failure (see Fig. 4.9) at about 94 percent of the bolt ultimate material strength. The three bolt group tests are shown in Fig. 6.16 in terms of average values of normalized mean stress and lead slip.

It can be seen in Fig. 6.16 that the bolts in a group have essentially the same stiffness as a single bolt until ultimate load is approached. In all three groups, the bolts failed at a normalized mean steel stress significantly lower than that reached by the single bolt. It is clear that for the bolt spacings tested, the total load capacity of a two-bolt group was not twice the load capacity of the single bolt.

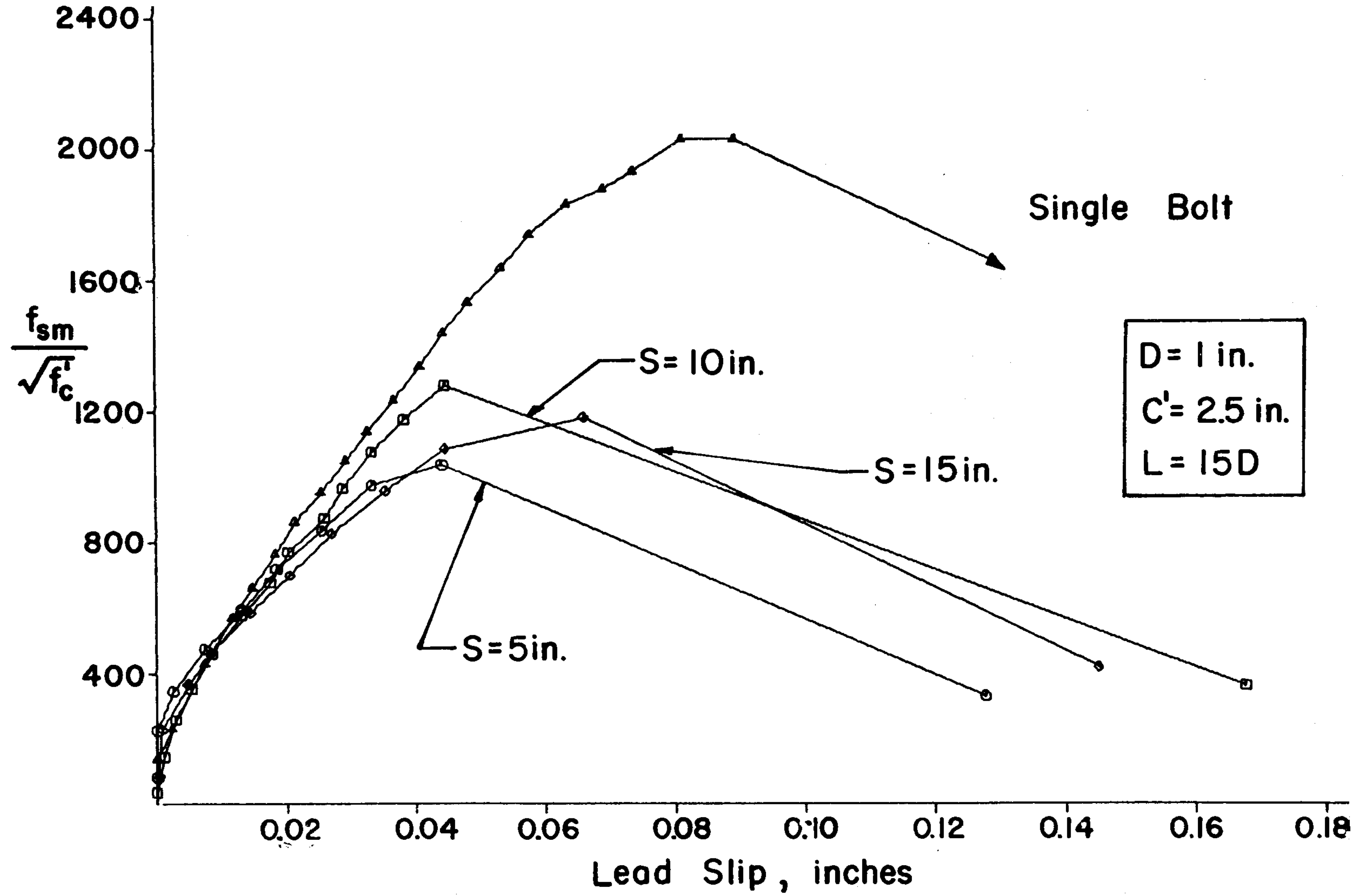


Fig. 6.16 Comparison of lead slip-lead stress curves of bolts in a group

Specifically, the total strengths of the groups with two 1 in. bolts on 5 in., 10 in., and 15 in. centers was only 1.02, 1.29, and 1.16 times the strength of the single 1 in. bolt, respectively. In addition, it should be emphasized that all of the bolt groups failed very abruptly with little previous cracking. In general, the single bolts failed by wedge-splitting, with considerable cracking prior to failure.

As discussed earlier, the load-carrying mechanism of an anchor bolt consists of three stages involving steel-to-concrete bond, bearing on the washer, and wedge-splitting by the cone of crushed and compacted concrete in front of the anchorage device. It seems reasonable to assume that placing bolts in a group could not have a significant effect on the steel-to-concrete bond characteristics of the bolts. In addition, the tests seemed to indicate that placing bolts in a group had no effect on the formation of the characteristic cone of crushed and compacted concrete in front of the anchorage devices. Figure 6.17 shows these cones fully formed for the group with the 5 in. bolt spacing. However, the presence of two bolts near one another definitely interfered with the formation of the individual wedge-splitting mechanisms of each bolt, as the splitting forces caused by the two cones of crushed and compacted concrete in front of the anchorage devices interacted with one another in the region between the bolts.

Figure 6.18 shows the crack pattern after failure of the group of two bolts on 5 in. centers. The locations of the washers in the anchorage devices are marked on the specimen with straight, dashed lines. As discussed earlier, a typical wedge-splitting failure for a single bolt was characterized by diagonal cracks originating near the anchorage device on the bolt axis and extending toward the front and sides of the specimen. In Fig. 6.18 it is seen that the wedge-splitting failure formed for the group with a 5 in. bolt spacing, but the anchorage devices are connected with a crack parallel to the washers, which indicates that the individual wedge-splitting mechanism of each

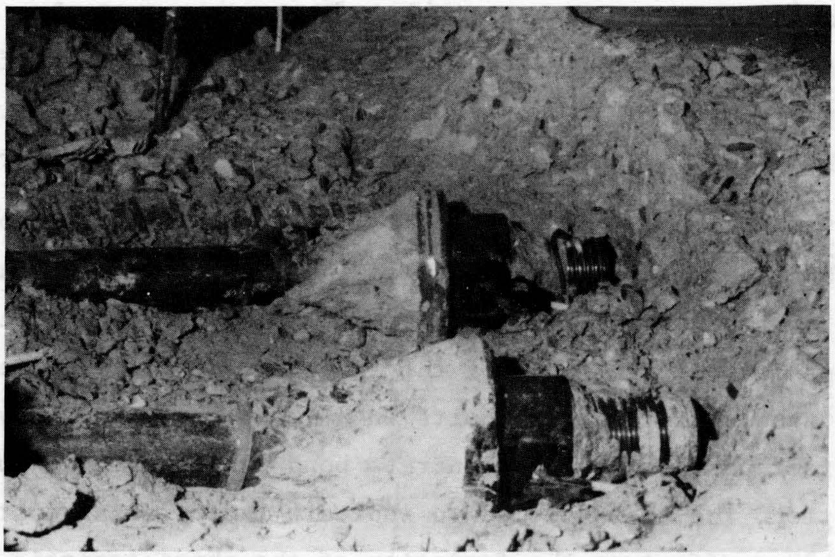


Fig. 6.17 Individual cones of crushed concrete in front of the anchorage devices--bolt spacing = 5 in.

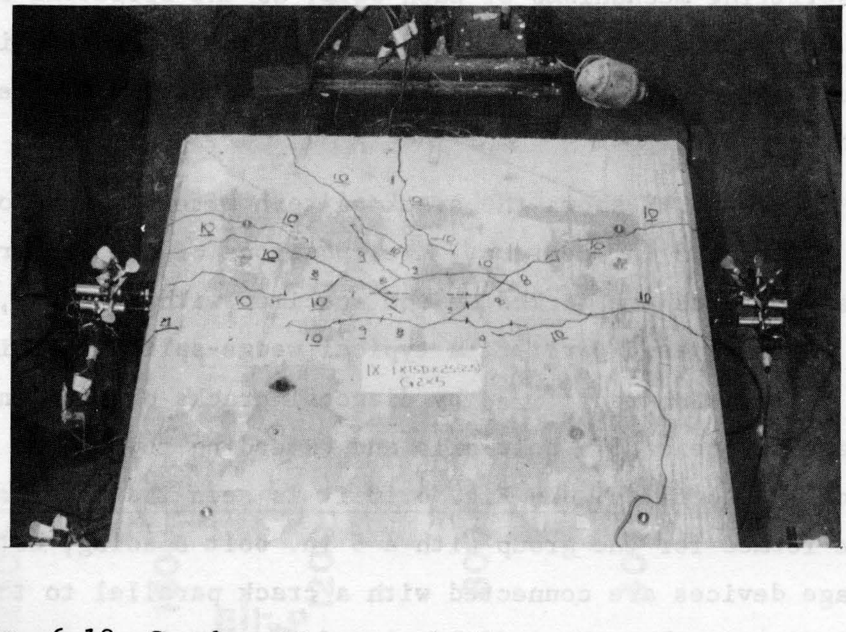


Fig. 6.18 Crack pattern at failure--bolt spacing = 5 in.

bolt could not form completely. In Fig. 6.19 this phenomenon is more clearly seen for the group with a 10 in. bolt spacing. Again, a wedge-splitting failure has occurred with a crack parallel to the washers linking the incomplete individual wedge-splitting mechanisms. Figure 6.20 shows the crack pattern at failure for the group with a 15 in. bolt spacing. Again, the wedge-splitting failure has formed, but the cracks between the bolts began to branch forward before meeting at the center, which indicates that with increasing bolt spacing the tendency toward formation of the individual wedge-splitting mechanisms also increases.

It can be concluded that bolts in a group may be subject to a very abrupt, nonductile failure at loads corresponding to individual bolt loads significantly less than the strength of a single bolt with similar geometry. The drastic reduction in individual bolt strength is apparently due to the interaction between splitting forces around the anchorage devices which presents the individual wedge-splitting mechanisms from forming completely. The tests on bolt groups indicate that there is a critical spacing beyond which the bolts behave as single bolts, but this spacing is apparently much larger than that which would normally be encountered in a typical highway structure. The tests also indicated that any interaction between bolts in a group was sufficient to cause a significant reduction in strength. Although much research is needed to completely define all the factors which affect the strength of bolts in a group, it is clear that great care must be exercised in defining the ultimate strength of a bolt group on the basis of the strength of a single bolt with geometry similar to that of the individual bolts in the group.

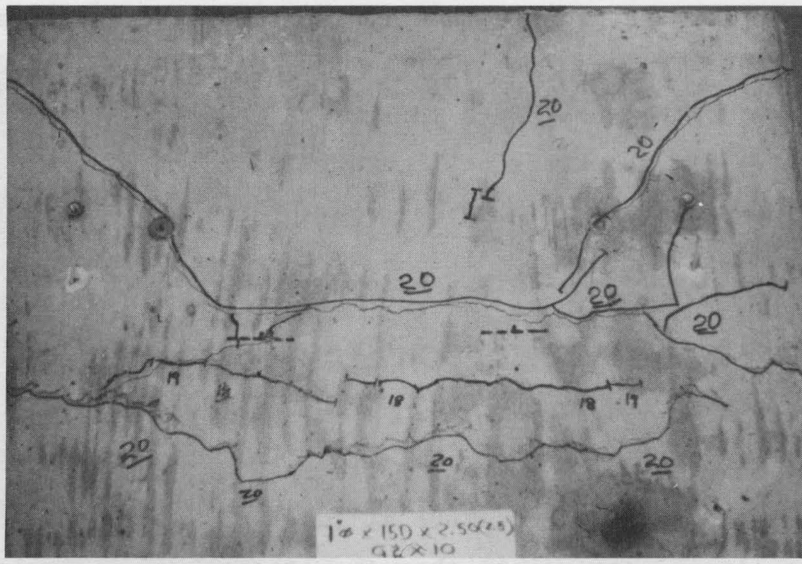


Fig. 6.19 Crack pattern at failure--bolt spacing = 10 in.

Fig. 6.20 Crack pattern at failure--bolt spacing = 15 in.

CHAPTER 7

DESIGN EQUATION FOR ANCHOR BOLTS

7.1 Introduction

In order to incorporate the variables studied into an equation for predicting the strength of anchor bolts, a regression analysis of the data was undertaken. The objective was to develop an empirical equation that would provide a reasonable estimate of the strength of a single anchor bolt subjected to tension. Because the mode of failure influenced the strength, it was decided to limit the analysis to those specimens failing in a wedge-splitting mode; that is, those specimens with embedment lengths of 15 or 20D which did not reach yield. Using the empirical equation and design parameters properly, the anchor bolt installation could be proportioned to have a strength of wedge-splitting failure in excess of the yield strength of the bolt, thereby ensuring a ductile behavior in the installation.

7.2 Statistical Analysis

The data were analyzed with the aid of the computer program STEP01, which is a version of the Stepwise Regression Program BMD02R of the Biomedical Computer Program Series developed at the University of California.[12] The STEP01 program is basically a forward stepwise multiple regression algorithm and is, therefore, a powerful aid in developing likely empirical models.

The data base used in the statistical analysis consisted of 48 tests with the following variables: bolt diameter, clear cover, size of the anchorage device, and concrete strength. Twelve of these tests were from an earlier anchor bolt study, conducted at The University of Texas at Austin,[2] using bolts with a nut and no washer for an anchorage device. The main variables and combinations of variables were used in different empirical models which were investigated with the stepwise regression techniques.

Embedment length was not normally included as a main variable in the statistical analysis, since the present test program indicated that embedment length did not affect the ultimate strength of an anchor bolt installation if a certain minimum length was provided such that a wedge-splitting failure could develop.

It was confirmed that variations in concrete strength could conveniently be accounted for using the $\sqrt{f'_c}$ as measure of tensile strength. It was also established that clear cover was a highly significant parameter, but that as the value of clear cover increased, its effect did not increase linearly. A large number of equations were examined and the equation which best fit the available data took the form

$$T_n = A_b\sqrt{f'_c}\left[96 + 142\ \ln\left(\frac{1}{1 - \frac{C_w}{C'}}\right)\right] \tag{7.1}$$

where T_n = nominal load capacity of a single anchor bolt under simple tension, lbs.

A_b = net bearing area of anchorage device

$= \frac{\pi}{4}(D_w^2 - D^2)$

D = bolt diameter, in.

D_w = washer diameter, in.

C' = clear cover to bolt, in.

C_w = cover over the washer

The statistical analysis indicated that large washers in the anchorage device were not fully effective in bearing. It was found that this relative inefficiency could be accounted for by limiting the value of the net bearing area used in the design equation to about $4D^2$. It was also found that, regardless of diameter, the washer had to be sufficiently thick to prevent excessive bending which would reduce the efficiency of the washer in bearing; a minimum washer thickness of $D_w/8$ is suggested.

As noted earlier, it was found necessary to provide a certain minimum embedment length in order to allow the wedge-splitting failure mechanism to form. The required minimum length can be assumed to be more of a function of the ultimate tensile strength of the bolt than merely the bolt diameter. Since the strength of an anchor bolt installation is related to the net bearing area of the anchorage device, the required minimum embedment length can be related to some measure of the net bearing area. Therefore, it is suggested that an embedment length of at least $12(D_w - D)$ be provided.

7.3 Design Equation

In order to produce an acceptable design equation, the constants in Eq. (7.1) were rounded and the limits described in Sec. 7.2 were included with the following result:

$$T = \varphi 140 A_b \sqrt{f'_c}\left[0.7 + \ln\left(\frac{2C'}{D_w - D}\right)\right] \leq A_{sm} f_y \tag{7.2}$$

where T = design tensile capacity (φT_n) of a single anchor bolt, lbs., with embedment length not less than $12(D_w - D)$

A_b = net bearing area, in.^{2}, $\frac{\pi}{4}(D_w^2 - D^2)$, but not greater than $4D^2$

D = bolt diameter, in.

D_w = diameter of washer, in., with thickness not less than $D_w/8$

C' = clear cover to bolt, in.

φ = a capacity reduction factor to account for the scatter in the test results and for variations in material properties and construction tolerances (0.75 for anchor bolts)

A_{sm} = mean tensile area of the anchor bolt

It is emphasized that Eq. (7.2) is based on tests of isolated anchor bolts loaded in pure tension which failed by wedge-splitting. The limits on embedment length, bearing area, and washer thickness are intended to ensure that any failure in the concrete is of the wedge-splitting type. Anchor bolts with relatively short embedment lengths (or embedment lengths approximately equal to clear cover) or very shallow clear covers are likely to fail in a different mode. In addition, great care must be taken in applying Eq. (7.2) to bolts in a group or to bolts subjected to combined lateral load and tension. Testing has shown that bolts subjected to combined lateral loading and tension and bolts in a group experience a significant reduction in strength compared to the strength of a single-bolt subjected to tension only. Additional research is needed before specific design recommendations can be made.

Table 7.1 lists the ratio of the measured bolt ultimate strength to that predicted using Eq. (7.2) for the 48 tests studied. For the 29 full-scale tests the average ratio is 1.03, with a standard deviation of 0.16 and for the 19 model tests, the average ratio is 1.07 with a standard deviation of 0.23.

It can be seen that the largest underestimate of the strength is calculated for cases in which the cover is small. However, that is probably desirable because constructional tolerances are likely to be more significant in cases where the cover is small.

Figure 7.1 shows graphically the proposed design equation and the data from the test program plotted to indicate the accuracy of the equation. As can be seen the equation provides a reasonable estimate of strength, yet is simple to use and reflects the critical parameters observed in the test program.

TABLE 7.1 COMPARISON OF TEST RESULTS WITH SUGGESTED DESIGN EQUATION

Diameter (in.)	Embedment Length (in.)	Clear Cover (in.)	Washer Diameter (in.)	Net Bearing Area (1) (in.^{2})	f'_c (psi)	T_{test} (kips)	T_{calc} (kips)	$\frac{T_{test}}{T_{calc}}$
Full-Scale Tests								
1.0	15.0	1.0	2.5	4.12	5500	62.00 (2)	42.28	1.47
1.0	15.0	2.5	2.5	4.12	3910	77.00	68.73	1.12
1.0	15.0	3.5	2.5	4.12	3520	76.80	76.73	1.00
1.0	20.0	2.5	2.5	4.12	3880	79.30	68.46	1.16
1.75	26.25	2.5	4.0	10.16	3950	139.8	133.9	1.04
1.75	26.25	3.5	4.0	10.16	3630	149.4	157.3	0.95
1.75	26.25	4.5	4.0	10.16	4680	178.3	203.0	0.88
1.75	26.25	4.5	4.0	10.16	4310	168.0	194.8	0.86
1.75	26.25	6.0	4.0	10.16	3980	212.9	213.1	1.00
1.75	35.0	3.5	4.0	10.16	3680	143.4	158.3	0.91
1.75	35.0	4.5	4.0	10.16	4910	188.3	207.9	0.91
1.75	26.25	3.5	3.0	4.66	2640	68.00	81.27	0.84
1.75	26.25	3.5	3.25	5.89	4300	155.4	121.2	1.28
1.75	26.25	3.5	3.50	7.22	5470	148.9	155.9	0.96
1.75	26.25	3.5	5.0	12.25 (3)	2770	117.8	132.4	0.89
1.75	26.25	3.5	4.0	10.16	5380	163.5	191.5	0.85
1.75	26.25	3.5	5.0	12.25 (3)	3960	157.0	158.3	0.99
1.25	12.5	1.25	2.16	2.44	4580	51.26	39.60	1.29
1.25	12.5	2.38	2.16	2.44	4610	58.24	54.40	1.07
1.25	12.5	3.75	2.16	2.44	4610	74.13	65.18	1.14
1.25	12.5	5.00	2.16	2.44	4580	68.51	71.61	0.96
1.25	12.5	2.38	2.16	2.44	2460	48.35	39.73	1.22
1.25	12.5	2.38	2.16	2.44	2460	45.83	39.73	1.15
2.0	20.0	2.0	3.38	5.83	4780	124.3	99.82	1.25
2.0	20.0	3.0	3.38	5.83	5240	107.5	128.5	0.84
2.0	20.0	5.0	3.38	5.83	5240	155.3	158.7	0.98
2.0	20.0	7.0	3.38	5.83	4780	175.0	170.5	1.03
2.0	20.0	3.0	3.38	5.83	2240	72.00	84.00	0.86
2.0	20.0	3.0	3.38	5.83	2240	76.00	84.00	0.90
Model Tests								
0.5	7.5	0.50	1.375	1.29	3460	11.50	8.78	1.31
0.5	7.5	0.75	1.375	1.29	3460	16.00	13.05	1.23
0.5	7.5	0.75	1.375	1.29	3260	16.82	12.67	1.33
0.5	7.5	1.00	1.375	1.29	5450	19.00	20.18	0.94
0.5	7.5	1.00	1.375	1.29	3090	13.00	15.19	0.86
0.5	7.5	1.00	1.06	0.69	3260	15.33	10.80	1.42
0.5	7.5	1.00	1.75	1.29 (4)	3260	13.00	12.06	1.08
0.5	7.5	1.25	1.375	1.29	3660	15.05	18.95	0.79
0.5	7.5	1.25	1.06	0.69	3450	14.20	12.37	1.15
0.5	7.5	1.25	1.75	1.29 (4)	3260	17.84	14.37	1.24
0.5	7.5	1.50	1.375	1.29	3950	15.48	21.74	0.71
0.5	7.5	1.75	1.375	1.29	3400	18.00	21.77	0.83
0.5	7.5	1.75	1.06	0.69	3090	10.48	13.50	0.78
0.5	7.5	2.00	1.375	1.29	3090	18.75	22.09	0.85
0.5	7.5	2.25	1.06	0.69	3500	21.00	15.80	1.33
0.5	10.0	0.75	1.375	1.29	2970	14.95	12.09	1.24
0.5	10.0	1.00	1.375	1.29	3090	13.96	15.19	0.92
0.5	10.0	1.25	1.375	1.29	3460	17.50	18.42	0.95
0.5	10.0	1.25	1.06	0.69	3500	16.54	12.46	1.33

(1) In the model tests the net bearing area of the standard diameter washer, D_w = 1.375 in., is considered fully effective, although it slightly exceeds the suggested limit $A_b \le 4D^2$.

(2) This bolt failed by local spalling of cover over anchorage device.

(3) Effective net bearing area limited to $4D^2$.

(4) Effective net bearing area limited to that of a standard diameter washer.

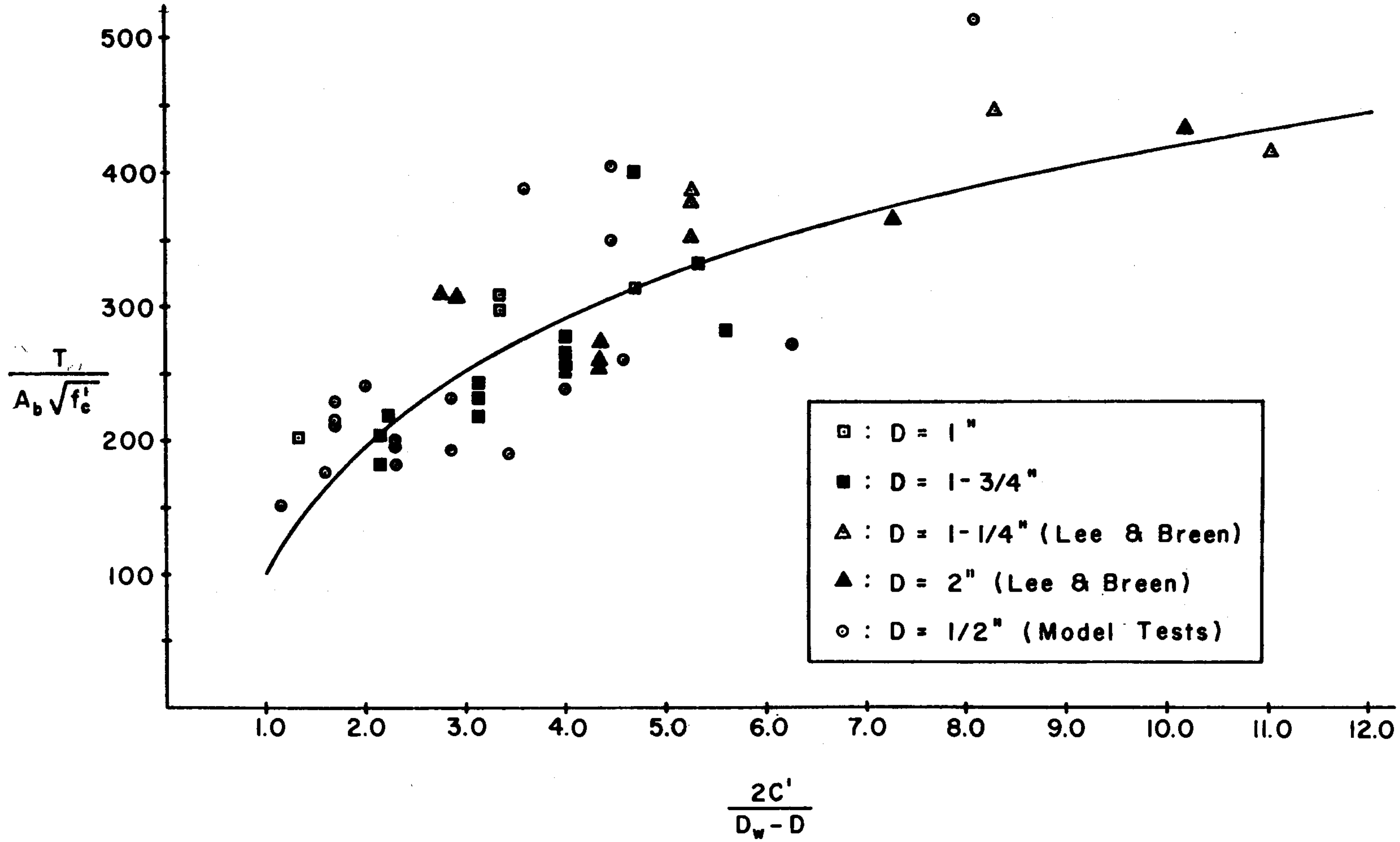


Fig. 7.1 Comparison of predicted and observed test values

CHAPTER 8

SUMMARY AND CONCLUSIONS

8.1 Summary

The primary objective of this investigation was to evaluate the effects of bolt diameter, embedment length, clear cover and bearing area on the behavior of high-strength anchor bolts. Both model and full-scale tests were conducted. In addition, a series of exploratory full-scale tests were run to determine the influence of cyclic load, lateral load, bolt groups, and transverse reinforcement on the behavior.

An examination of the test results indicated that three distinct modes of failure could be identified:

1) Bolt yielding, generally in the threaded region
2) Cover spalling, relatively sudden localized spalling of the cover over the anchorage device in cases where the clear cover was small
3) Wedge splitting, formation of a cone of crushed and compacted concrete in front of the washer which split the concrete into blocks and forced the spalling of the large portion of the cover in cases with large clear cover.

It should be noted that it is possible that an anchor bolt installation with a value of clear cover on the order of the value of embedment length may exhibit an entirely different mode of failure than those listed above. The test program did not examine anchor bolt installations with such large values of clear cover.

The mechanism by which the bolt transfers load to the concrete is a sequence involving steel to concrete bond, bearing against the washer, and wedging action by the cone of concrete ahead of the washer.

Steel to concrete bond is present only in very early stages of loading and as bond along the bolt is lost, the load is transferred entirely by bearing against the washer. As bearing stresses increase the concrete crushes and becomes compacted forming a cone which acts as a wedge to split the cover and cause spalling and failure. Depending on the amount of cover and length of embedment, the loading terminates in failures of the type described above.

The exploratory tests indicated that cyclic loads at or below the service level of the anchorage did not detrimentally influence the strength or behavior of the anchor bolt. Transverse reinforcement significantly increased the strength and ductility of anchor bolts with relatively shallow cover. The transverse reinforcement provided lateral restraint once the cover split away from the bolt. Lateral forces normal to the edges resulted in a significant reduction in strength. The lateral loads induced additional cracks which tended to split the cover from the bolt at the lead end. The strength of bolts in groups was drastically reduced over that for an isolated bolt. For the spacings considered, the capacity of a two bolt group was about the same as that for a single bolt with each bolt carrying half the force of an isolated bolt. It should be emphasized that the exploratory tests were not extensive enough to permit quantitative evaluations of the parameters considered.

8.2 Conclusions

The test results indicated that the clear cover and bearing area were the prime variables influencing the strength of anchor bolts. In order to incorporate the variables into an equation for predicting the strength of isolated anchor bolts subjected to tension only, a regression analysis of the data was carried out. Because the mode of failure influenced strength, only the results of specimens failing in a wedge-splitting mode were considered. The analysis produced the following equation for the nominal tensile capacity (in lbs.) of an anchor bolt:

$$T_n = 140A_b\sqrt{f'_c}\left[0.7 + \ln\left(\frac{2C'}{D_w - D}\right)\right] \tag{7.2}$$

where A_b is the net bearing area (in.^{2}), D and D_w are the bolt and washer diameter (in.) and C' is the clear cover to the bolt (in.). The design tensile strength, T, can be determined as:

$$T \le \varphi T_n \quad \text{but} \le A_{sm} f_y$$

where φ = a capacity reduction factor of 0.75, A_{sm} = mean tensile area of the anchor bolt, and f_y = yield strength of the anchor bolt material. The equation provides a reasonable estimate of strength when compared with the test results and reflects the critical parameters observed in the test program.

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