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FATIGUE AND STATIC STRENGTH OF STUD SHEAR CONNECTORS

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I. INTRODUCTION

In an earlier paper, "Composite Beams with Stud Shear Connectors" (1)* the results of a laboratory investigation on a 32 ft. span composite beam bridge were reported. After completion of these tests it was realized that additional information--particularly on the fatigue behavior of stud shear connectors--was highly desirable. For this reason a series of further tests were conducted the results of which are presented in this paper.

II. DESCRIPTION OF TEST SPECIMENS

The specimens were of the push-out type. Such specimens have been used extensively in this country (2) and abroad (3) for determining the static strength of shear connectors. As shown in Fig. 1, a specimen consisted of an 8 WF40 section framed by two concrete slabs. To each flange of the WF beam four shear connectors were welded as specified in the figure. A substantial reinforcement was placed in the slab similar to the reinforcement of a deck slab in an actual composite beam bridge. Attention was given to the amount of contact area between the steel beam and the concrete slab for the reason that it may influence the test results. The ratio of the area to the total number of studs equaled 48.5 in^2/stud, a value which approaches the range of actual cases.

Three different types of specimens were used. Five specimens with 1/2 inch diameter L-Connectors (Fig. 1, Detail A) had flat concrete slabs. Transverse reinforcing bars were placed into the bend of the L-Connectors as indicated in Fig. 1. Straight studs (Fig. 1, Detail B) were used in three specimens of equal geometric proportions, the transverse reinforcement crossing in front of the studs. In the two haunched specimens with 3 inch deep haunches at 45° (Fig. 1), no connection existed between the 1/2 inch diameter L-Connectors of 2-1/4 inch height and the slab reinforcement. In Table I all ten specimens are listed indicating also the loading condition under which they were tested.

In the fabrication of the specimens, the following materials were used. The 8WF40 section was of A-7 steel. The stud material met the ASTM A15-54T Specification. The mechanical properties corresponded to the ones reported in reference (1), Appendix, Tables C and D. As reinforcement for the slab deformed bars of the type A-305 were used. The welding of all the studs was performed by K S M personnel with their own equipment using "K S M Solid Fluxed L-Connectors" for the 1/2 inch diameter and "K S M Solid Fluxed Stud Connectors" for the 3/4 inch diameter studs. No special preparation was given to the flange surface of the beam prior to welding. Forms, reinforcement, etc. were fabricated at Fritz Engineering Laboratory. All specimens were poured at the same time in the same upright position as they were subsequently tested. A ready mixed concrete with a maximum aggregate size of 1 inch was used. Compacting of the pour was accomplished with a mechanical vibrator. Fig. 2 is a picture of a haunched specimen prior to pouring. The concrete strength was checked on ten cylinders at appropriate intervals, the results obtained being shown in Table II.

* Refers to list of references, chapt. VIII

III. FATIGUE TESTS

Fig. 3 illustrates the test set-up for fatigue loading. The testing frame was erected with standard parts of the new Fatigue Testing Installation at Fritz Engineering Laboratory (4). The concrete slabs of a specimen rested on a 1/2 inch thick plywood panel in order to obtain a uniform distribution of the reactions from the floor. For the haunched specimens only the slab proper was supported on the panel, the haunch itself being free. A hydraulic jack with spherical seats at each end applied an axial load to the WF beam of the specimen. Cyclic loading at a frequency of 500 cycles/minute was produced by an "Amsler Pulsator". During testing the maximum and minimum load levels were automatically kept constant at the preselected values. The hydraulic measuring device allowed the determination of the load within an accuracy of 2%.

The instrumentation consisted of two dial gages only, measuring the relative slip between the slabs and the steel beam at two diagonally opposite flange tips. A special device, shown in Fig. 4, was developed such that the gage recorded the maximum slip only. Piece (A), holding the dial gage was mounted on a 1/2 inch diameter steel bar (B) imbedded in the concrete slab. The L-shaped piece (C) was held against piece (A) by two screws and spring washers. A flat bar (D) being welded to the steel beam, followed its movement during cyclic loading. As slip increased the L-piece (C) was progressivley pushed downward such that the dial gage recorded the maximum amplitude only. The simplicity and reliability of the device was evident during actual operations and lead to very satisfactory results.

No further instrumentation and especially no strain measuring devices were incorporated. Firstly, it was felt that strain measurements on the studs were practically impossible. Furthermore, even if such measurements could be accomplished successfully their use would be of rather limited value. For it is sufficiently recognized that the fatigue and static behavior of any connection can not be established on the knowledge of a local peak stress but depends on the general conditions in the immediate vicinity.

A total of seven specimens, namely No. 4 to 10, were tested in fatigue. The individual results are represented graphically in Fig. 5 to 11, showing the measured slip times 10^4 versus the number of cycles. For instance, specimen No. 4 with 1/2 inch L-Connectors was tested under load cycles varying between a maximum load P_{max} = 35,000 lb. and a minimum load P_{min} = 4,500 lb. These limits corresponded to average shearing stresses on the eight connectors of

$$v_{st}\ (\max) = \frac{35,000}{8 \cdot 0.196} = 22,290 \text{ psi} \qquad (1)$$

and

$$v_{st}\ (\max) = \frac{4,500}{8 \cdot 0.196} = 2,870 \qquad (2)$$

respectively. As seen from Fig. 5 the slip increased steadily with the number of load cycles and started to develop at an increased rate at about 150,000 cycles. Fracture of the studs occurred after a total of 223,200 cycles. The slip measurements of dials I and II indicated almost equal slip over the entire test. However, a check of the following Fig. 6 to 11 reveals that in general a considerable difference in the two slip measurements developed indicated that the connectors on one side of the specimen experienced greater damage. Similar, but slightly less pronounced differences developed in the static tests as will be reported in chapter IV.

Test No. 8 on a haunched specimen, Fig. 9, deserves special mention. The initial loading producing a maximum average shearing stress v_{st} (max) = 13,370 psi and a minimum stress of v_{st} (min) = 1,910 psi respectively, was kept constant over one million cycles. As no failure ocurred and the average slip amounted to 0.00115 inches only, the load was increased in three consecutive steps. Failure finally took place at a stress v_{st} (max) = 22,290 psi and a total of 2.815 million cycles.

The appearance of a typical fatigue fracture is illustrated in Fig. 12. As shown in the picture, the load applied to the steel beam was acting in a downward direction. Fatigue cracks started from the lower side of the studs. In general a crack initiated at the reinforcement of the weld and developed a concave depression toward the beam, removing even some of the beam material. The failure surface of the studs, Fig. 12b, showed a rounded shape. The typical crystalline texture of a fatigue fracture was apparent in the lower part. However, the upper region revealed marks of repeated impact, probably at the point of final fracture. The concrete surrounding the studs showed only local damage. Around the base of the studs it was completely pulverized and could be blown out leaving a clearance of about 0.01 inches.

Specimens No. 7 and 8 developed not only a fatigue fracture at the base of the studs but also approximately 3/4 inch above the base. Fig. 13 shows a picture of specimen No. 8 after failure. The short pieces broken out of the studs are visible on Fig. 13b. Inspection of the upper studs showed that the second fatigue fracture was progressed through almost the entire section. A possible failure mechanism is sketched in Fig. 14. The first fracture developed at the base of the stud. After considerable cracking the stud was still held in the concavity of the beam material. Under these conditions the bending moment at the base of the stud was greatly reduced and a new maximum developed, as indicated in Fig. 14b, causing ultimately the second fracture. It should be mentioned that both specimens, having L-Connectors, were tested at relatively low stresses, sustained 1.748 and 2.815 million cycles respectively and developed comparatively little slip prior to failure.

All fatigue results are summarized in Table III and Fig. 15. In addition to the test results, Table III lists the fatigue strength of the L-Connectors at 100,000 also 600,000 and 2,000,000 cycles respectively. The derivation of these latter values will be discussed in chapter V. Fig. 15 presents a standard logarithimic plot of the results. The stepped curve for specimen No. 8 indicates that the maximum average shearing stress on the studs was increased in steps as discussed previously.

A direct comparison between specimens with 1/2 inch diameter L-Connectors of 2-1/4 inch height (Fig. 1, Detail A) and 3/4 inch diameter straight connectors of 4 inch height (Fig. 1, Detail B) was performed at the two stress levels of v_{st} (max) = 22,290 psi and v_{st} (max) = 15,600 psi respectively. As seen from Fig. 15 and Table III, the straight studs exhibited definitely a lower fatigue strength than the L-Connectors, the ratio of the number of fatigue cycles being 169,400/223,200 = 0.76 and 474,000/1,748,000 = 0.27 at the two stress levels respectively. This behavior may be explained qualitatively by the following reasoning. The smaller 1/2 inch diameter L-Connectors introduce smaller local stress concentrations in the concrete. It is generally recognized that the ability of concrete to sustain such loading conditions increases the more localized they are. Secondly, the bearing pressure of the studs on the concrete is about inversely proportional to the stud diameter. However, the stud force varies with the square of the diameter such that heavier studs produce higher bearing pressures. Finally, the hook of the L-Connector produces a mechanical wedging action. The concrete, pressed against the connector, is prevented from escaping vertically by the hook. This results into an increased vertical pressure against the steel beam and consequently an increased friction. The upset head of a 4 inch high straight connector does not produce such action to a similar degree.

Comparing the behavior of the haunched specimen, No. 8, with 1/2 inch diameter L-Connectors and the flat specimens carrying the same connectors the first showed at least equal if not superior performance (Fig. 15). It should be remembered that the L-Connectors were only 2-1/4 inch high whereas the haunch was 3 inch deep. Furthermore, no connection between the slab reinforcement and the studs was provided. At the final failure load P = 35,000 lbs. the average concrete shearing stress v_c in the haunch at an elevation equal to the height of the studs reached

$$v_c = \frac{35{,}000}{2\cdot 24\cdot(8.077+2\cdot 2.25)} = 58 \text{ psi} \qquad (3)$$

There was no indication of any cracking in the haunch of the concrete slab during the entire test.

IV. STATIC TESTS

One of each of the three types of specimens was tested under static loading. The specimen was placed in an ordinary hydraulic testing machine as shown in Fig. 16. Relative slip between steel beam and the concrete slab was recorded by four dial gages located at the four flange tips of the WF beam. The loading was increased in steps. At appropriate levels the load was kept constant. After a waiting period in order to get the deformation stabilized the slip readings were taken. At high loads the waiting periods increased to about five minutes.

The results of the three tests are plotted in Fig. 17 to 19, showing the average slip recorded by the four dials versus the average nominal shearing stress of the studs, the latter being the load divided by the cross sectional areaof all studs. For specimens No. 1 and 2 the difference between the four individual slip readings did not amount to more than ± 20%. However, for specimen No. 3 it increased over 100% for high loads. It should be therefore recognized that not only under fatigue loading but also under static conditions a considerable spread between individual slip readings must be expected. Approaching ultimate load, a marked influence of the loading speed on the value of the load itself was experienced in all three tests. For that reason not too much significance should be attributed to the recorded ultimate load. Its value does not have any bearing on the design of shear connectors anyhow.

In Fig. 19 the end slip measured on the bridge specimen reported in (1) with 1/2 inch diameter L-Connectors is also shown. The curve presents the average of the slips on the South East and South West end plotted in Fig. 8a and 8b of the mentioned report. The correspondence with the curve obtained from the push-out specimen No. 3 is fair. It therefore may be concluded that the push-out test produced essentially the same conditions as normally existing in actual composite beams.

The relative behavior of 3/4 inch diameter, 4 inch long straight studs and 1/2 inch diameter, 2-1/4 inch high L-Connectors can be judged by comparing Fig. 17 and 19. Specimen No. 3 with the 1/2 inch L-Connectors showed definitely a higher stiffness and less residual slip and hence confirmed similar findings reported in (1). A qualitative explanation can be given using the reasoning previously applied to explain the superior fatigue behavior of these connectors. The smaller stress concentrations, the lower bearing pressures and the wedging action produced by the L-Connectors improve their performance.

No appreciable difference developed between the haunched speciment No. 2 and the flat specimen No. 3 with 1/2 inch L-Connectors up to 0.05 inch slip. The difference in the ultimate load is of no significance as this load does not enter into design considerations. At the failure load P = 116,000 lb. of the haunched specimen No. 2 the average concrete shearing stress v_c in the haunch at an elevation equal to the height of the connectors was

$$v_c = \frac{116,000}{2 \cdot 24 \cdot (8.077 + 2 \cdot 2.25)} = 192 \text{ psi} \qquad (4)$$

It should be mentioned that in the test only the slabs were supported leaving the end of the haunches completely free.

V. EVALUATION OF RESULTS

(a) Fatigue Consideration

An attempt is made to delineate the fatigue strength of 1/2 inch diameter L-Connectors and to suggest allowable values under working conditions which may be used as a basis of discussion for specification writing agencies.

The fatigue test results obtained on the specimen with L-Connectors as plotted in Fig. 15, establish only specific values of fatigue strength at preselected stress levels. In order to obtain the fatigue strength corresponding to a given number of cycles, recourse to some extrapolation procedure is necessary. Assuming the relationship

$$f = S\,(N/n)^k \qquad (5)$$

applies (5), (6), where S is the stress at which a given specimen failed after N cycles, n is the number of cycles for which the fatigue strength f is desired, and k is an experimental constant, the value of f can be computed. The parameter k corresponds to the slope of a straight line which averages the individual test points in a logarithimic S-N diagram. In Fig. 15 this line is shown considering the test point of specimens No. 4 to No. 7 only. The value k = 0.10 is in the range of corresponding values obtained from fatigue tests of welded joints (6). Using equation (5) the fatigue strength f for 100,000, 600,000 and 2,000,000 cycles of specimens No. 4 to 7 with L-Connectors have been calculated and are listed in Table III. The average values of the four results are as follows:

100,000 cycles: $f_{100,000} = 20,700$ psi

600,000 cycles: $f_{600,000} = 17,000$ psi

2,000,000 cycles: $f_{2,000,000} = 15,400$ psi representing the fatigue strength of 1/2 inch diameter L-Connectors for the corresponding number of cycles. The connector force producing failure is obtained by multiplying the cross sectional area of the stud by the fatigue strength f. Remembering that in testing the minimum load amounted to about 12% of the maximum load, the above values are considered to be representative for zero to full load cycles also.

Allowable values under working conditions can be derived if the number of load cycles and the margin of safety are specified. Presently no such criteria against the possibility of fatigue failure exist for any type shear connectors. However, as the connectors are welded to the steel beams it seems to be reasonable to follow the approach of the "Standard Specifications for Welded Highway and Railway Bridges" of the American Welding Society [7] for the design of welded connections. Their article 208 and Table I distinguish three types of loading for highway bridges depending on their length. The most severe conditions are produced by a "Short Critical Loading (not more than two panels or 60 feet of loaded single lane)". The corresponding recommendations are based on an occurance of 600,000 repetitions of full loading. The margin of safety incorporated into the allowable stresses depends on the type and orientation of the welds and is not constant. Appendix A of reference (7) and also newer publications of fatigue test results, such as (6), may be consulted for further information.

As composite beam bridges are generally used for shorter spans it was felt unnecessary at the present time to make any distinction between different loading cases in arriving at allowable values for shear connectors in fatigue. It is therefore proposed to use the most severe condition of 600,000 repetitions as a basis. Furthermore, on a comparative basis with the safety margins provided for welded joints, a safety factor s = 1.25 against fatigue is selected. Using these two criteria the allowable connector force for a 1/2 inch diameter L-Connector under zero to full load cycles is

$$Q_{all} = \frac{A \cdot f_{600,000}}{s} = \frac{0.196 \cdot 17,000}{1.25} \cong 2,700 \text{ lbs.}$$

Where: $A = 0.196$ inch^2 = cross sectional area of stud

$f_{600,000}$ = 17,000 psi = fatigue strength of L-Connector for 600,000 load cycles

$s = 1.25$ = factor of safety against fatigue

The results obtained on the bridge specimen reported in [1] give further support to the above value of Q_{all}. Under 1,000,000 cycles of design live load each stud was subjected to 2,060 lbs. (see Table III, reference (1)), under a further 290,000 cycles of 125% live load the stud force was increased to 2,580 lbs. Even the final 250,000 cycles of 150% producing a connector force of 3,090 lbs. did not result in any fatigue. It may therefore be concluded that Q_{all} = 2,700 lbs. presents a reasonable design value including a sufficient margin of safety against fatigue failure.

In actual bridges the shear acting on the connectors may not only vary between zero and a maximum value but reverse its direction. No tests under such alternating loading have been performed. However, it must be expected that such conditions will lower the fatigue strength of a connector to a similar degree as it occurs in the welded joints. Following the approach used in the "Standard Specifications for Welded Highway and Railway Bridges" (7) for welded joints the allowable connector force Q_{all} could be made subject to the ratio minimum to maximum connector force.

No data is available concerning the influence of the concrete strength on the fatigue behavior of the L-Connectors. The average concrete strength of the push-out specimens at the age of testing (average age 32 days) reached about 5,300 psi (see Table II). For the bridge specimen the corresponding values were 37 days and 3,400 psi (reference (1), Table I and Appendix, Table B). The latter figures suggest that a minimum specified cylinder strength at 28 days, $f_c' = 3{,}000$ psi, is sufficient to insure the necessary fatigue strength.

(b) Static Considerations

Considering the useful static capacity of the L-Connectors the two tests No. 2 and 3 give additional support to the recommendations made in reference (1). At a stress $v_{st} = 38{,}000$ psi the slip curve for specimen No. 3, Fig. 19, shows a definite break. The corresponding residual slip, as estimated from the figure, reached about 0.003 inches. The slip curve of the haunched specimen No. 2, Fig. 18, exhibits a somewhat less pronounced increase in slip at the same stress level. The increased residual slip of 0.007 inches should have no influence on a satisfactory performance. Multiplying the above stress by the cross sectional area of a L-Connector produces the connector force $Q = 38{,}000 \cdot 0.196 = 7{,}450$ lbs. This value corresponds very closely to the value $Q = 7{,}310$ lbs. determined on the bridge specimen as governing the useful capacity of L-Connectors in accordance with the appropriate criteria of the AASHO Specifications (8).

(c) Remarks on Slab with Haunches

The use of the relatively short L-Connectors of 2-1/4 inch height in conjunction with haunched slabs requires some special considerations. Depending on the shearing stresses present at a section through the top of the studs (2-1/4 inches above the top flange of the steel beam) shear reinforcement of the haunch may or may not be required. The haunched specimen No. 2 withstood an average concrete shearing stress $v_c = 192$ psi at static ultimate load whereas specimen No. 8 carried a stress $v_c = 58$ psi under fatigue at the failure of the studs. In the design of the haunches the appropriate clauses covering the web reinforcement of reinforced concrete members should be followed. If reinforcement is necessary it should be anchored to the L-Connectors. In a design, stresses produced by shrinkage and plastic flow are usually neglected. However, it is known that these effects may cause relatively large shearing forces at the ends of composite beams. A nominal amount of reinforcement of the haunches at the beam ends should be provided even if not required by the shearing stresses due to dead and/or live loads. It will prevent the development of possible shrinkage cracks along the haunch.

(d) Design Recommendations

Supplementing the recommendations of reference (1) by the above presented considerations, the following summary is presented for the design of composite beams with 1/2 inch L-Connectors:

(1) Bridge Design

(a) The geometric shape of the connector is shown in Fig. 1, Detail A.

(b) The hook should preferably be oriented against the direction of the horizontal shear (toward middle for simple beams).

(c) The maximum pitch should not be more than 24 inches.

(d) The useful static capacity of the shear connector in pounds is given by:

$$Q_{uc} = 7{,}300 \sqrt{\frac{f_c'}{3600}} \cong 120 \sqrt{f_c'}$$

Where f_c' = 28 days cylinder strength of concrete in psi.

The resistance value at working load is obtained by dividing Q_{uc} by the appropriate safety factor.+

(e) To insure a sufficient margin of safety against fatigue the allowable maximum connector force, Q_{all}, produced by live load (or dead load plus live load in case of shoring of steel beams) should be limited to

$$Q_{all} \leq 2{,}700 \text{ lbs.}$$

The cylinder strength of the concrete f_c' at the age of 28 days should have a minimum strength of 3,000 psi.

(f) If the slab is provided with haunches the shearing stress in the concrete at the height of the upper ends of the connectors should be checked. Reinforcement of the haunches, if necessary, should be provided in accordance with the applicable clauses for the web reinforcement of reinforced concrete beams.

(2) Building Design (Primarily Static Loading)

(a) The useful capacity of the shear connector in pounds

$$Q_{uc} = 120 \sqrt{f_c'}$$

Where f_c' = 28 days cylinder strength of concrete in psi.

(b) The allowable maximum connector force under the specified working loads

$$Q_{all} = \tfrac{1}{2} \; Q_{uc} = 60 \sqrt{f_c'}$$

VI ACKNOWLEDGEMENT

The investigation reported in this paper was sponsored by K S M Products, Inc., Stud Welding Division, Merchantville 8, N. J. The tests were performed at Fritz Engineering Laboratory, Lehigh University, of which Professor W. J. Eney is director. Mr. Jose Santos, Research Assistant, conducted the tests and processed the recorded data. Preparation of the specimens and building of the test set-ups was under the supervision of Mr. Kenneth Harpel, laboratory foreman.

+ See AASHO, reference (8).

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T14 (56), Art. 3.9.5

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Washington, 1956

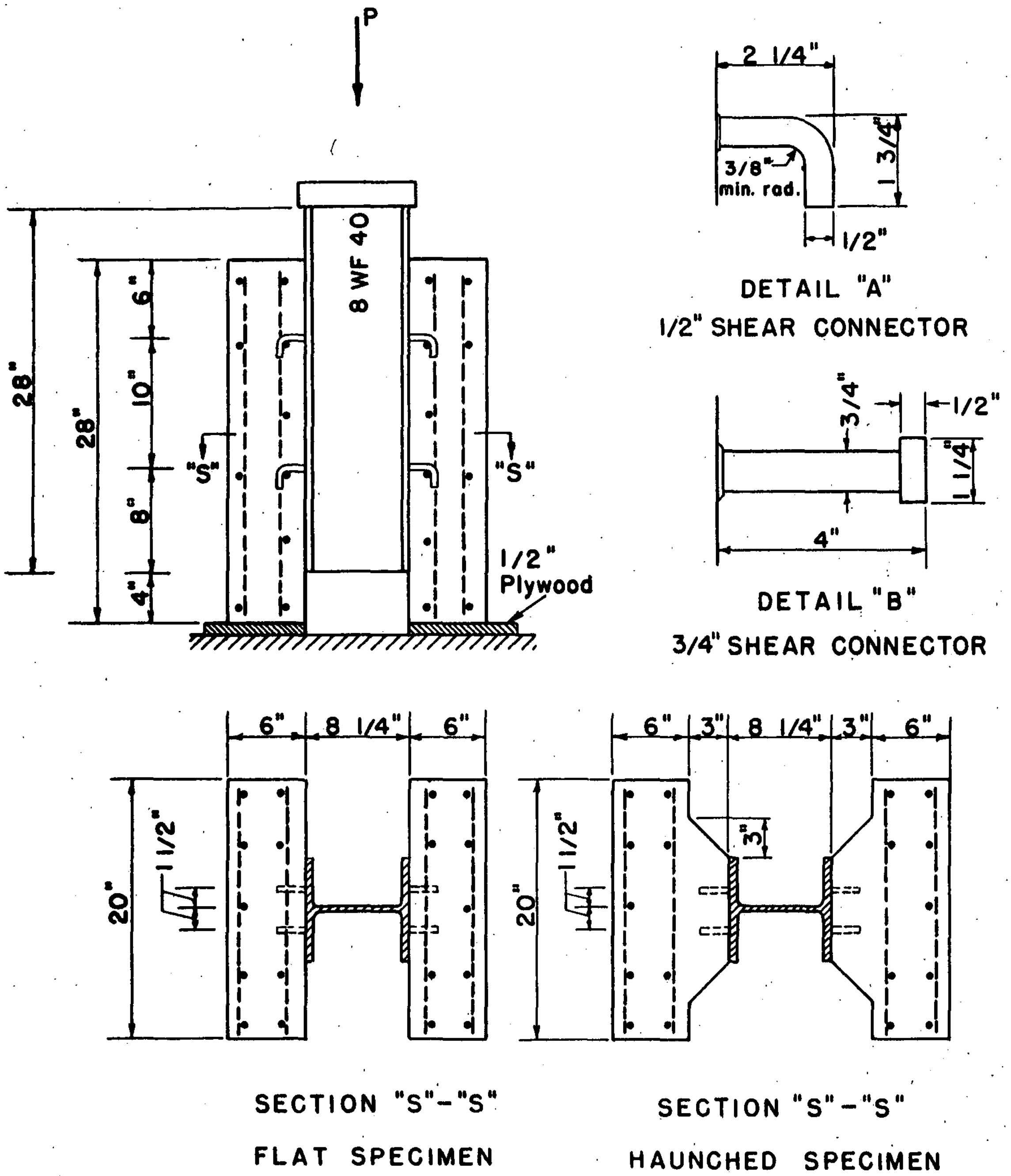


FIG. 1- DETAILS OF TEST SPECIMENS

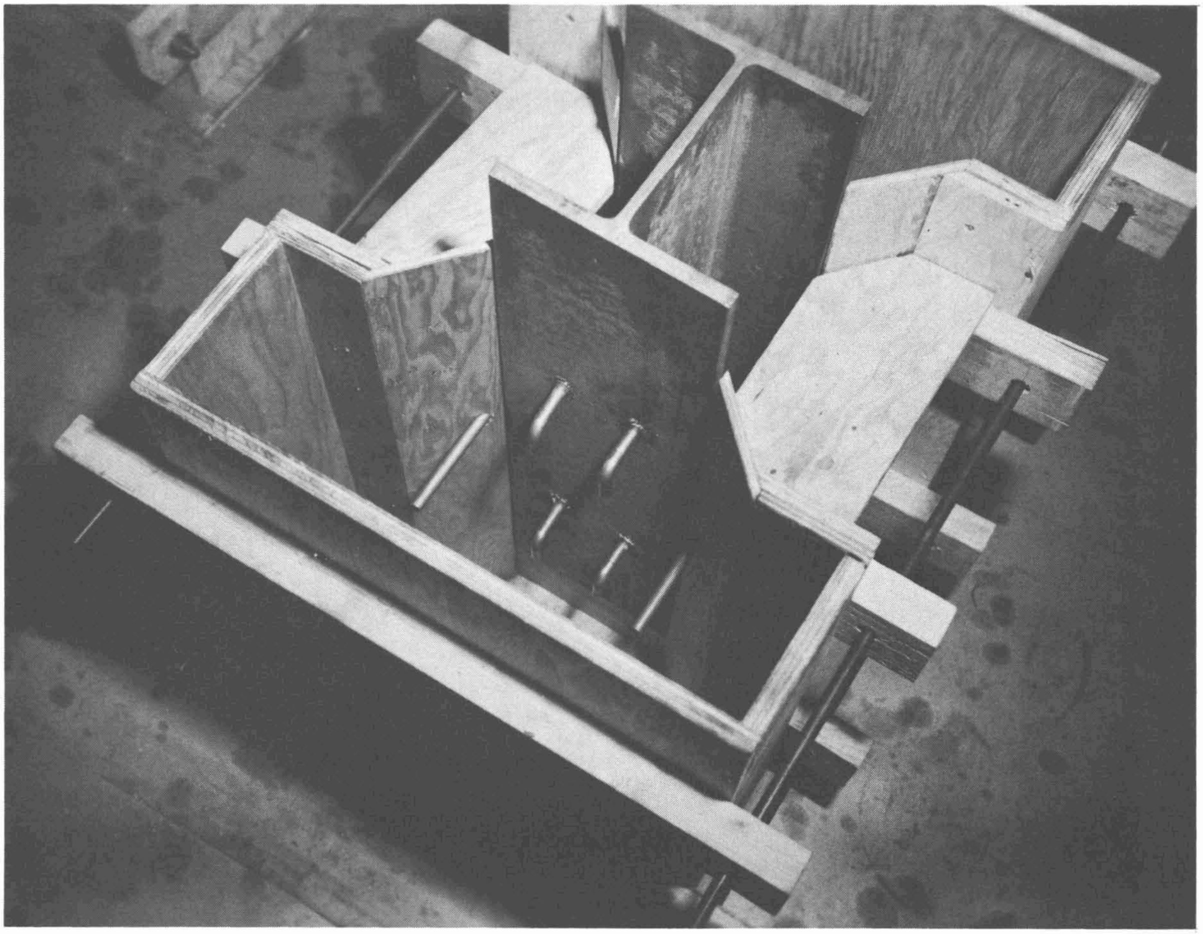


Fig. 2

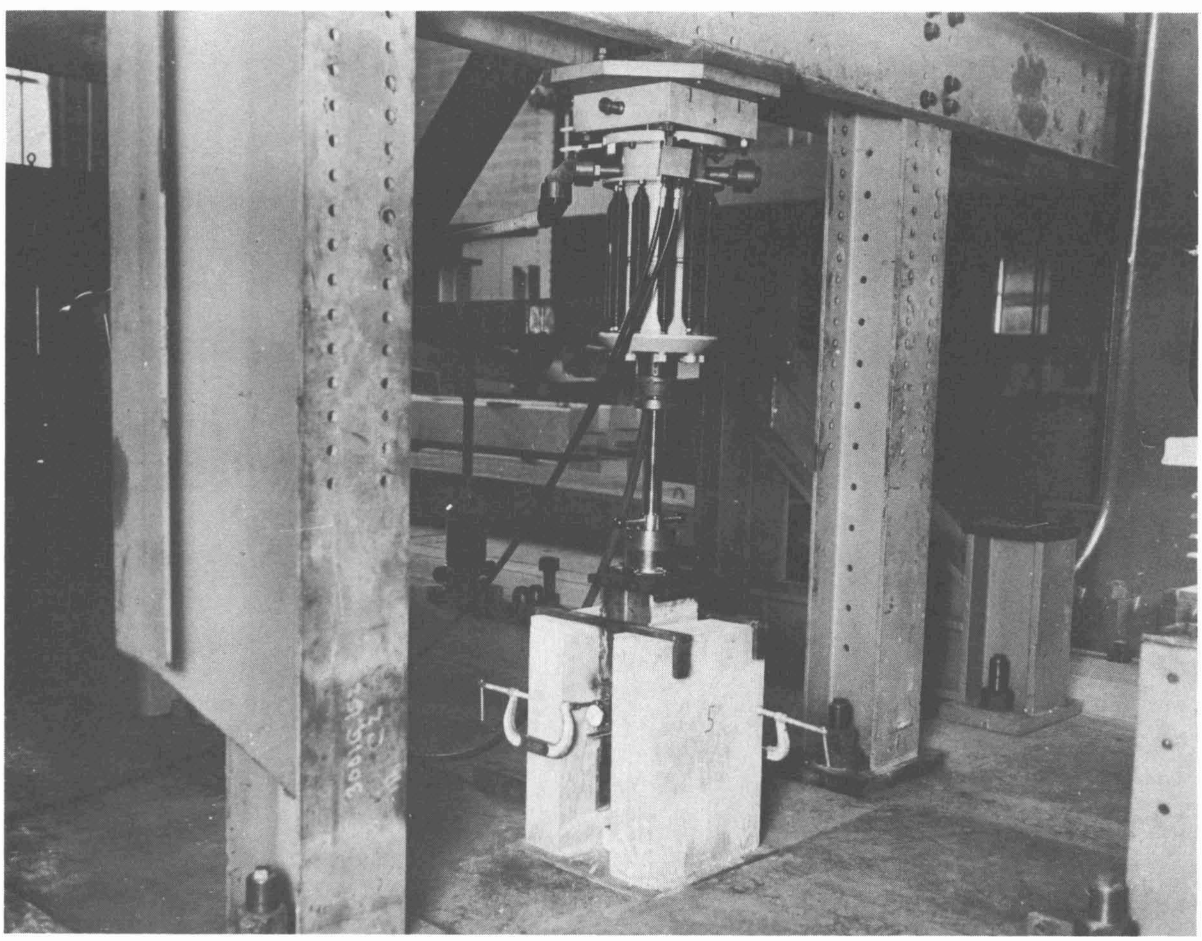


Fig. 3

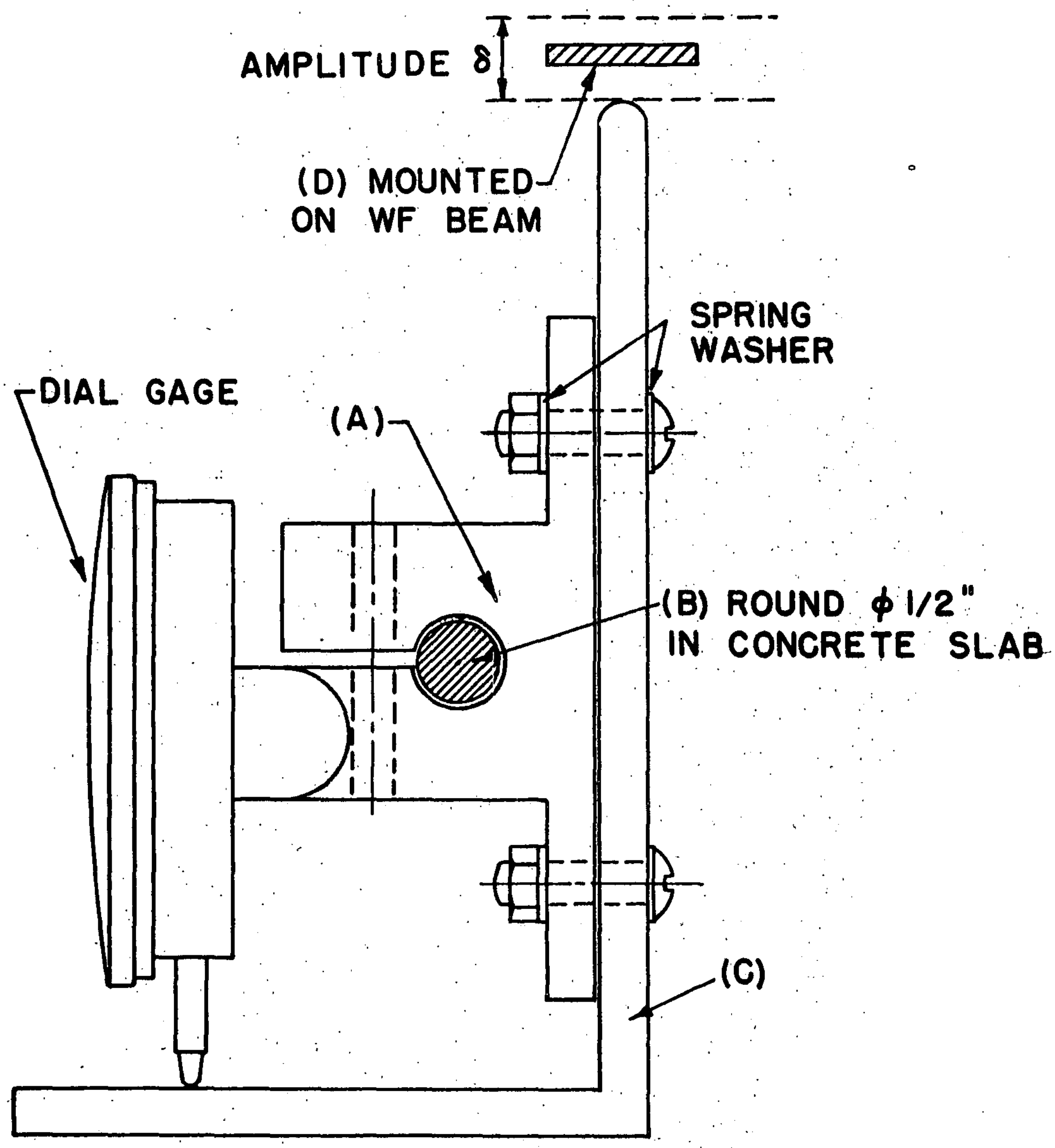
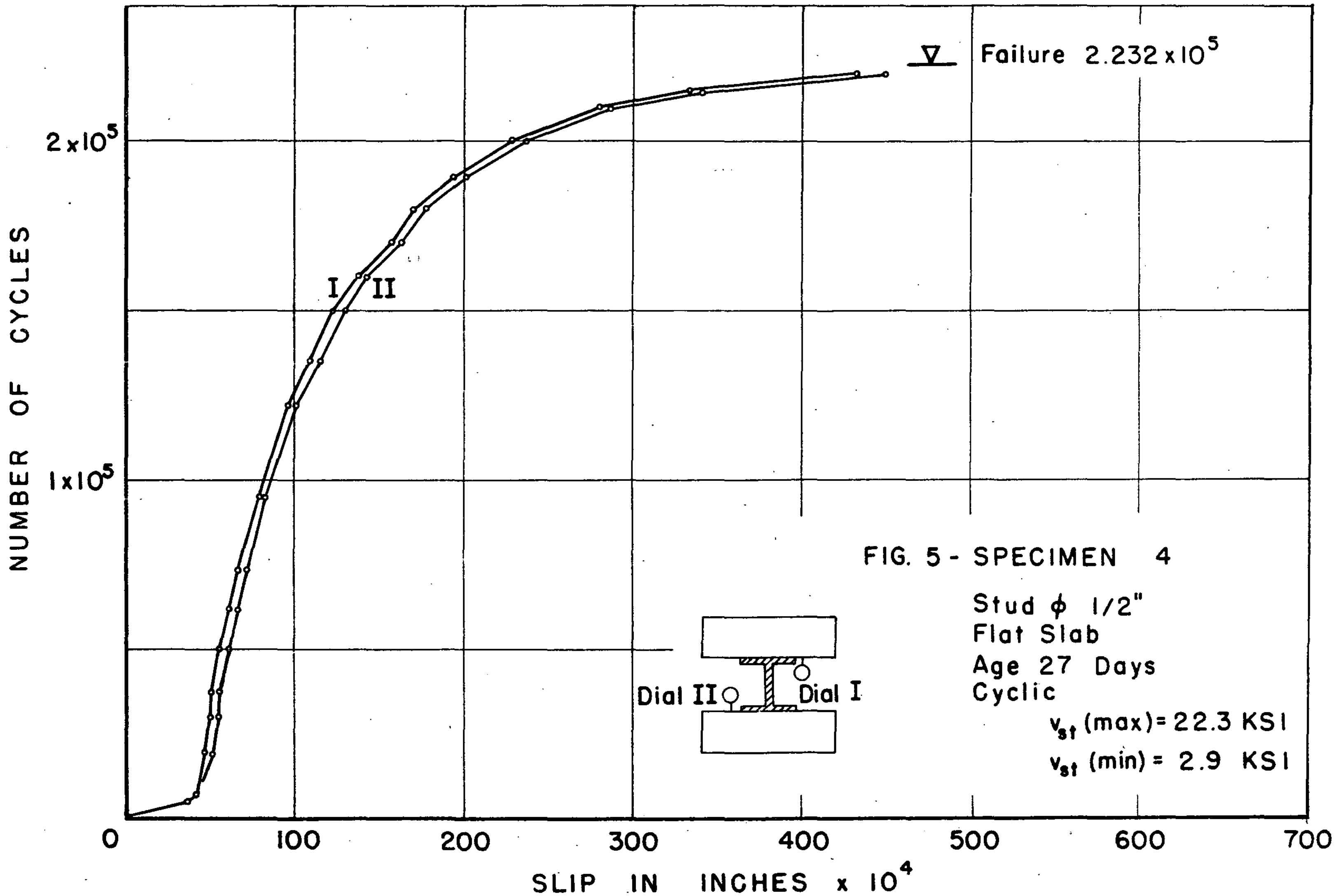
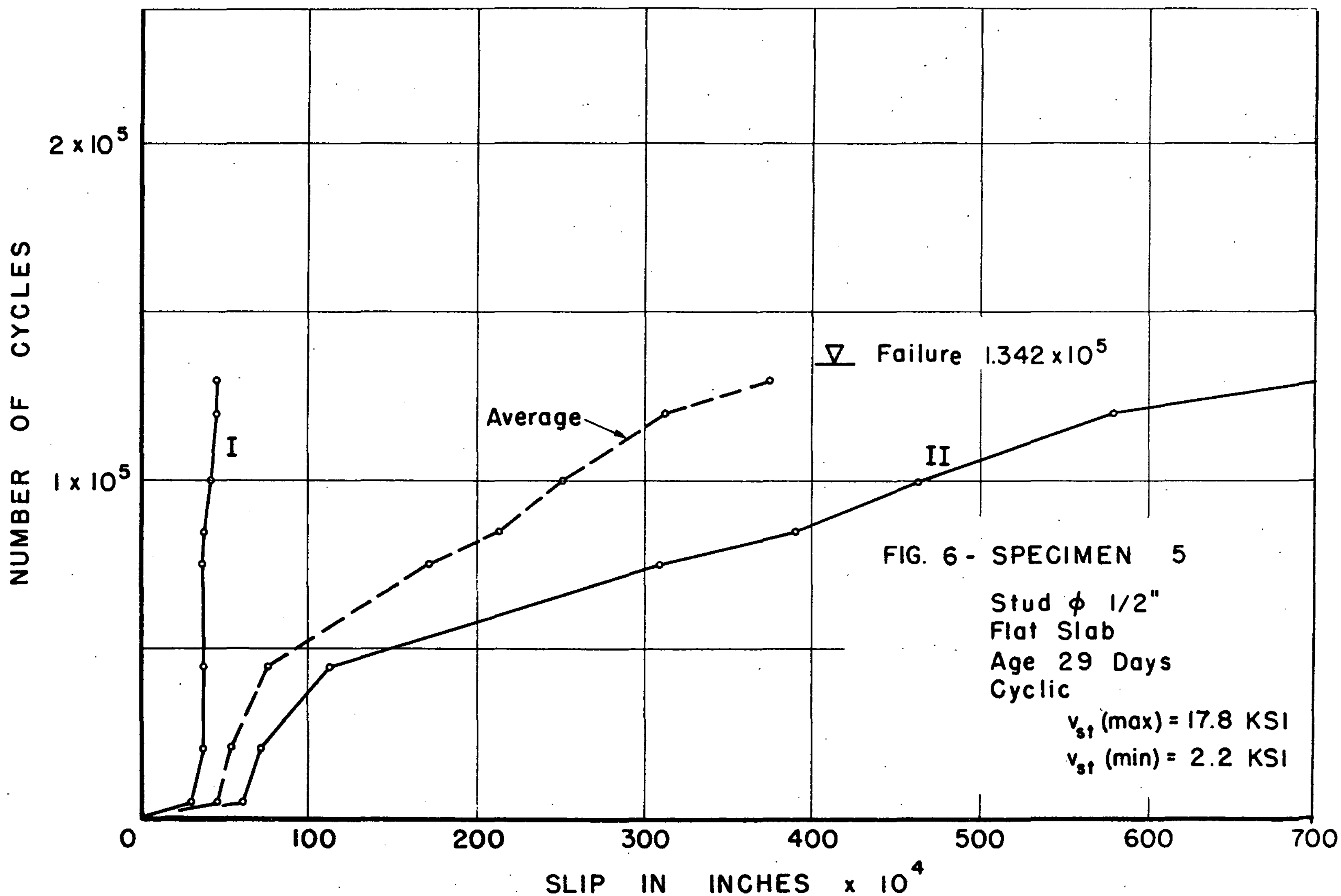


FIG. 4-SLIP MEASURING DEVICE


Failure 2.232 x 10^5
2 x 10^5
1 x 10^5
NUMBER OF CYCLES
I
II
FIG. 5 - SPECIMEN 4
Stud ϕ 1/2"
Flat Slab
Age 27 Days
Cyclic
v_{st} (max) = 22.3 KSI
v_{st} (min) = 2.9 KSI
Dial II
Dial I
0
100
200
300
400
500
600
700
SLIP IN INCHES x 10^4



2 x 10^5
1 x 10^5
NUMBER OF CYCLES
Failure 1.342 x 10^5
Average
I
II
FIG. 6 - SPECIMEN 5
Stud φ 1/2"
Flat Slab
Age 29 Days
Cyclic
v_st (max) = 17.8 KSI
v_st (min) = 2.2 KSI
0
100
200
300
400
500
600
700
SLIP IN INCHES x 10^4

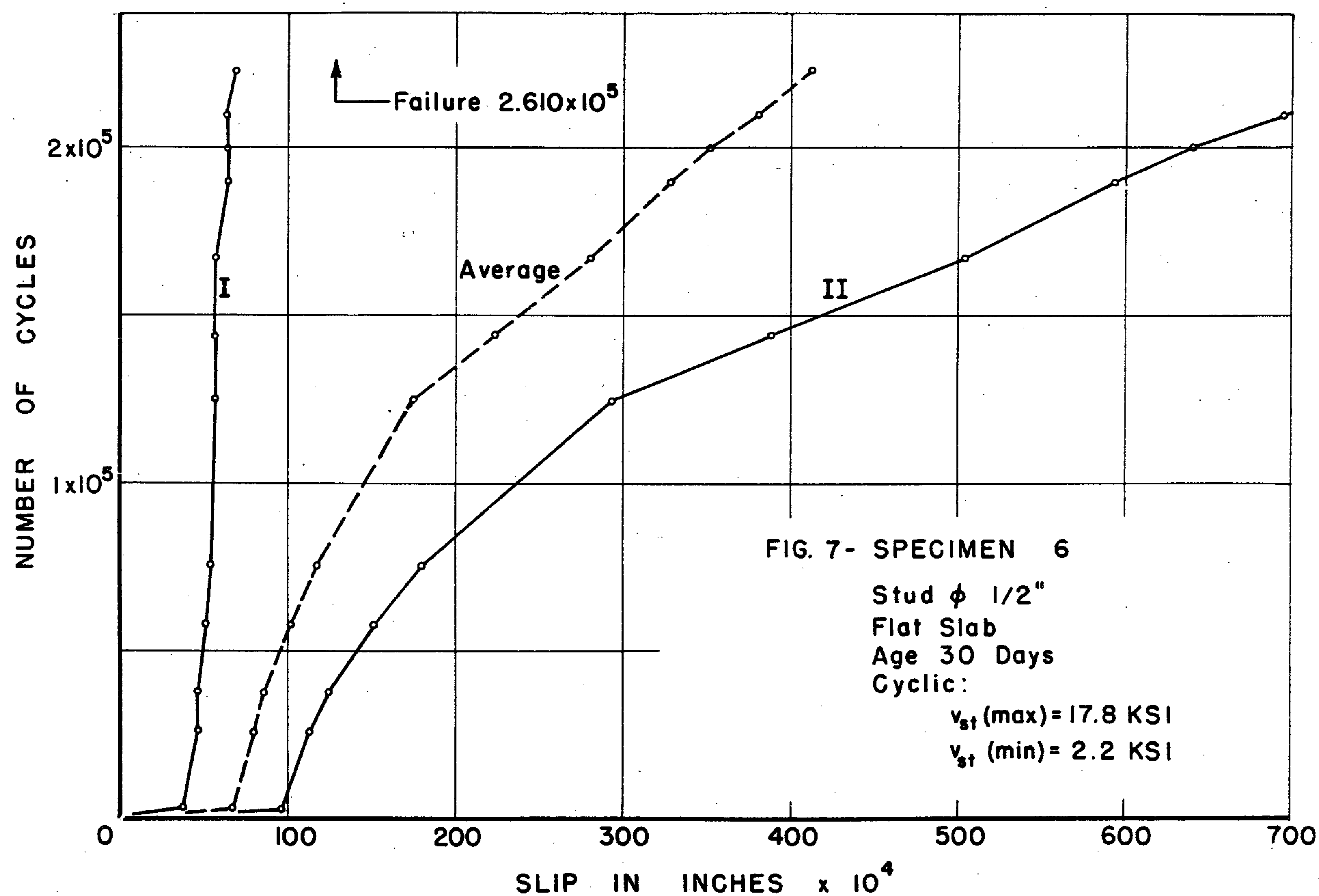


FIG. 7- SPECIMEN 6

Stud ϕ 1/2"
Flat Slab
Age 30 Days
Cyclic:
v_{st} (max) = 17.8 KSI
v_{st} (min) = 2.2 KSI

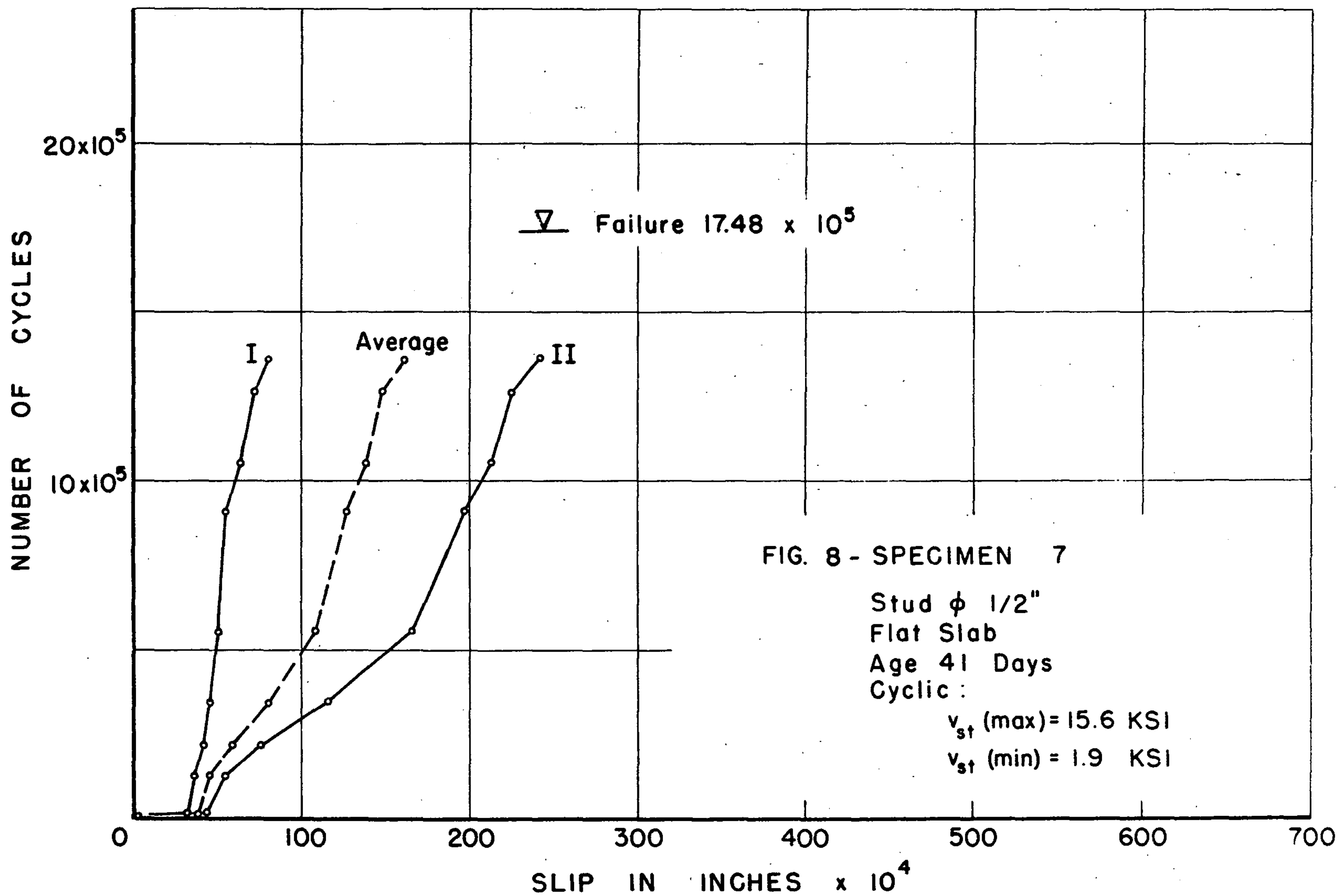


FIG. 8 - SPECIMEN 7

Stud ϕ 1/2"
Flat Slab
Age 41 Days
Cyclic :
v_{st} (max) = 15.6 KSI
v_{st} (min) = 1.9 KSI

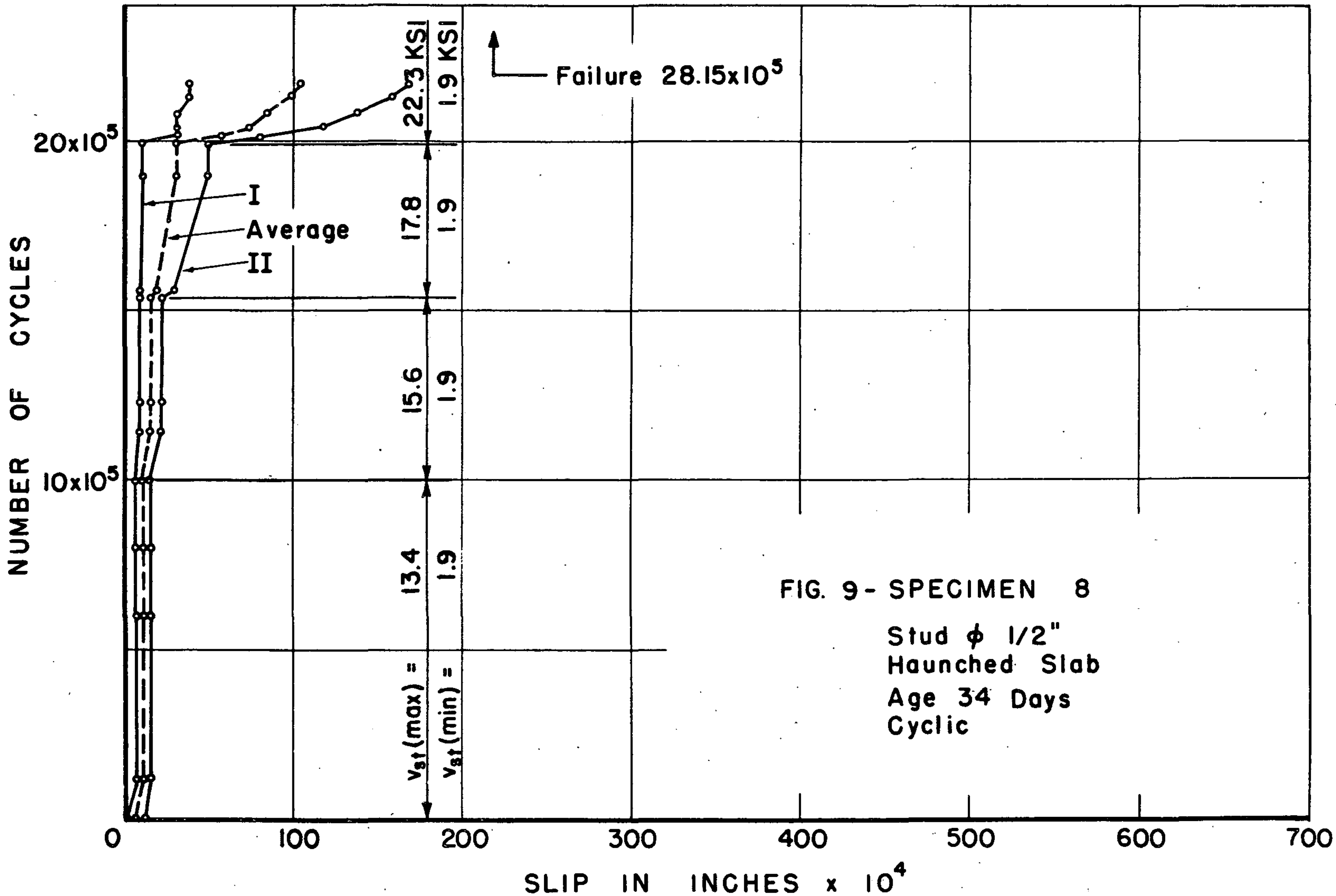


FIG. 9 - SPECIMEN 8

Stud ϕ 1/2"
Haunched Slab
Age 34 Days
Cyclic

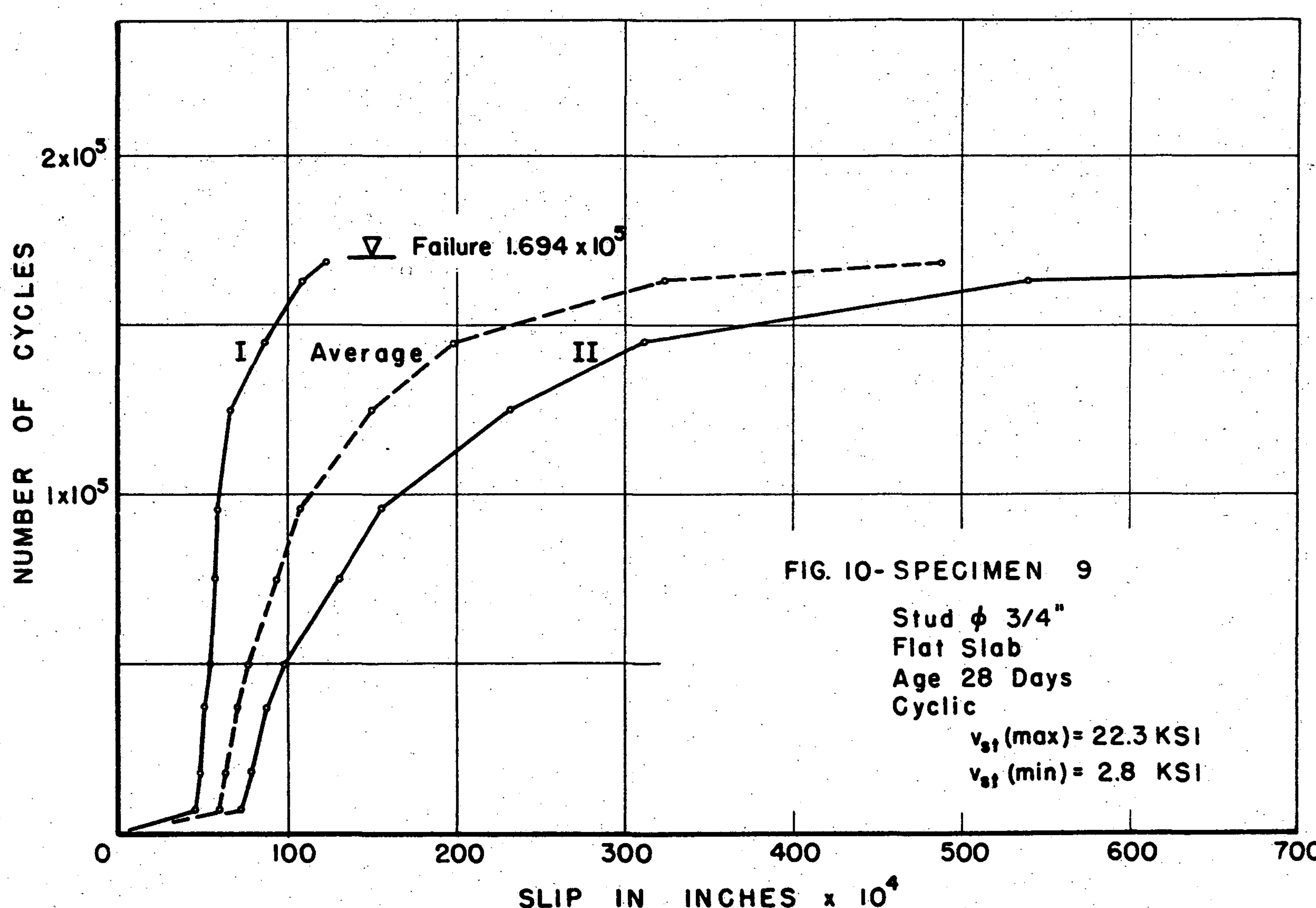


FIG. 10 - SPECIMEN 9

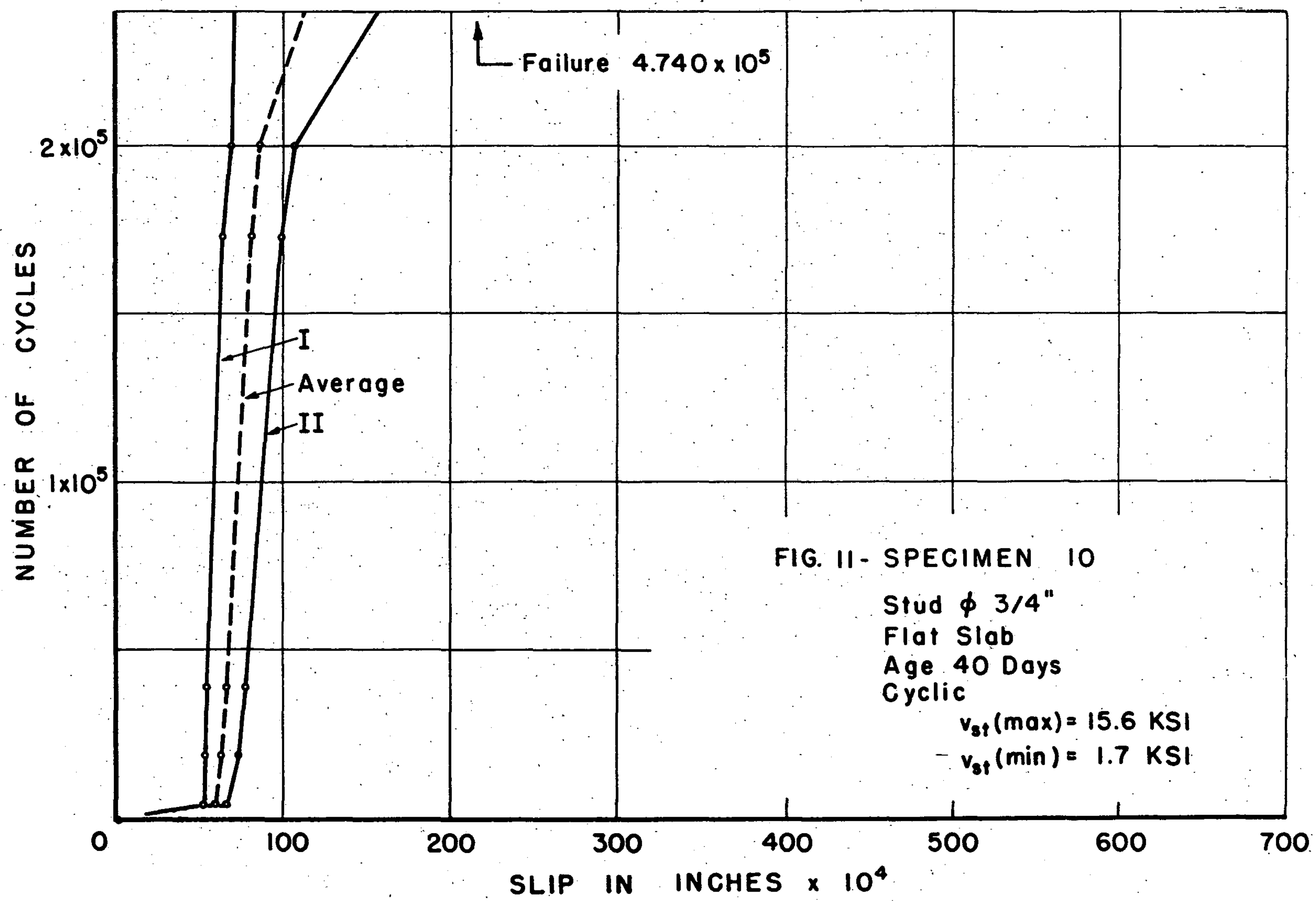


FIG. 11 - SPECIMEN 10

Stud ϕ 3/4"
Flat Slab
Age 40 Days
Cyclic
$v_{st}(max) = 15.6$ KSI
$v_{st}(min) = 1.7$ KSI

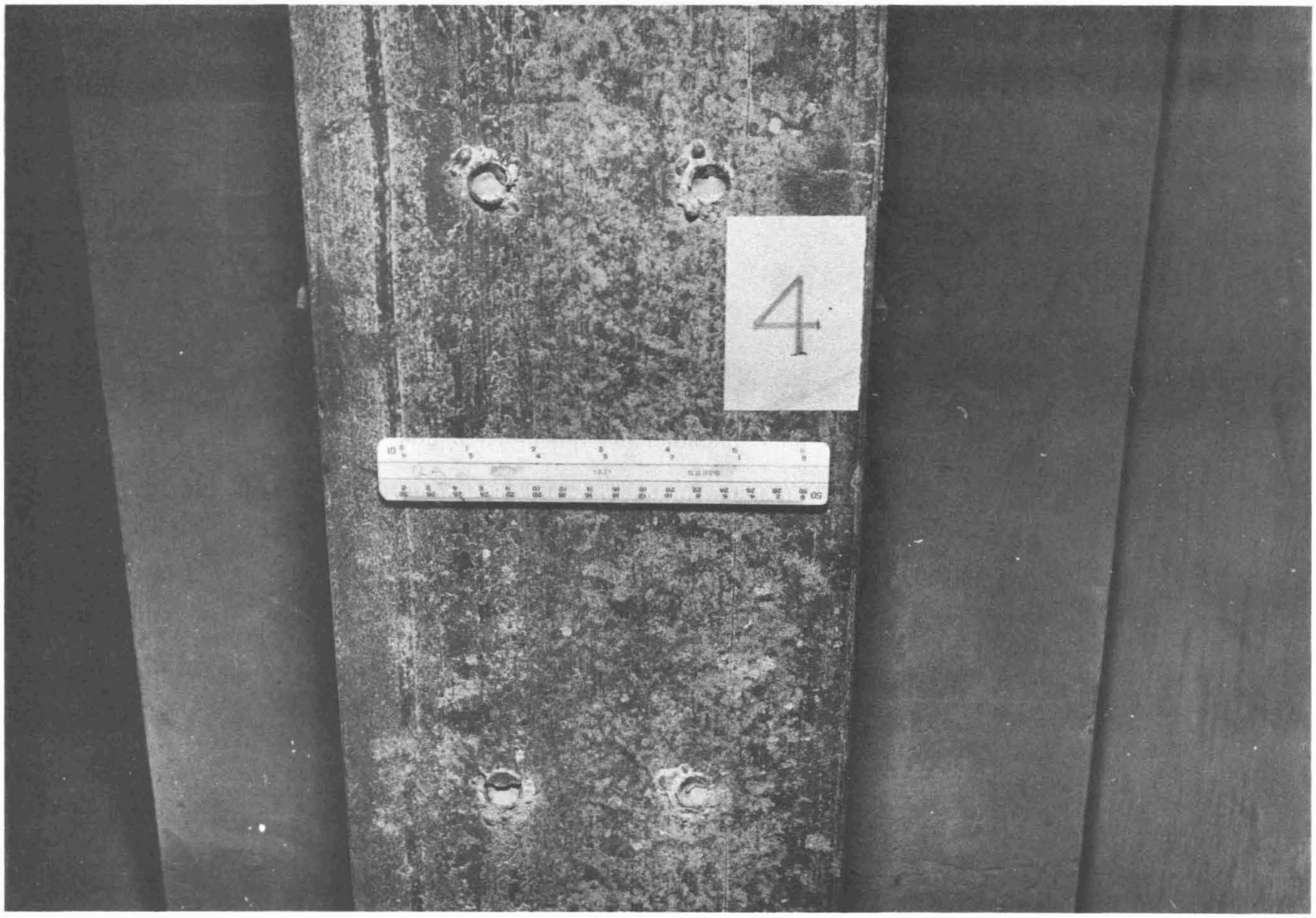


Fig. 12A

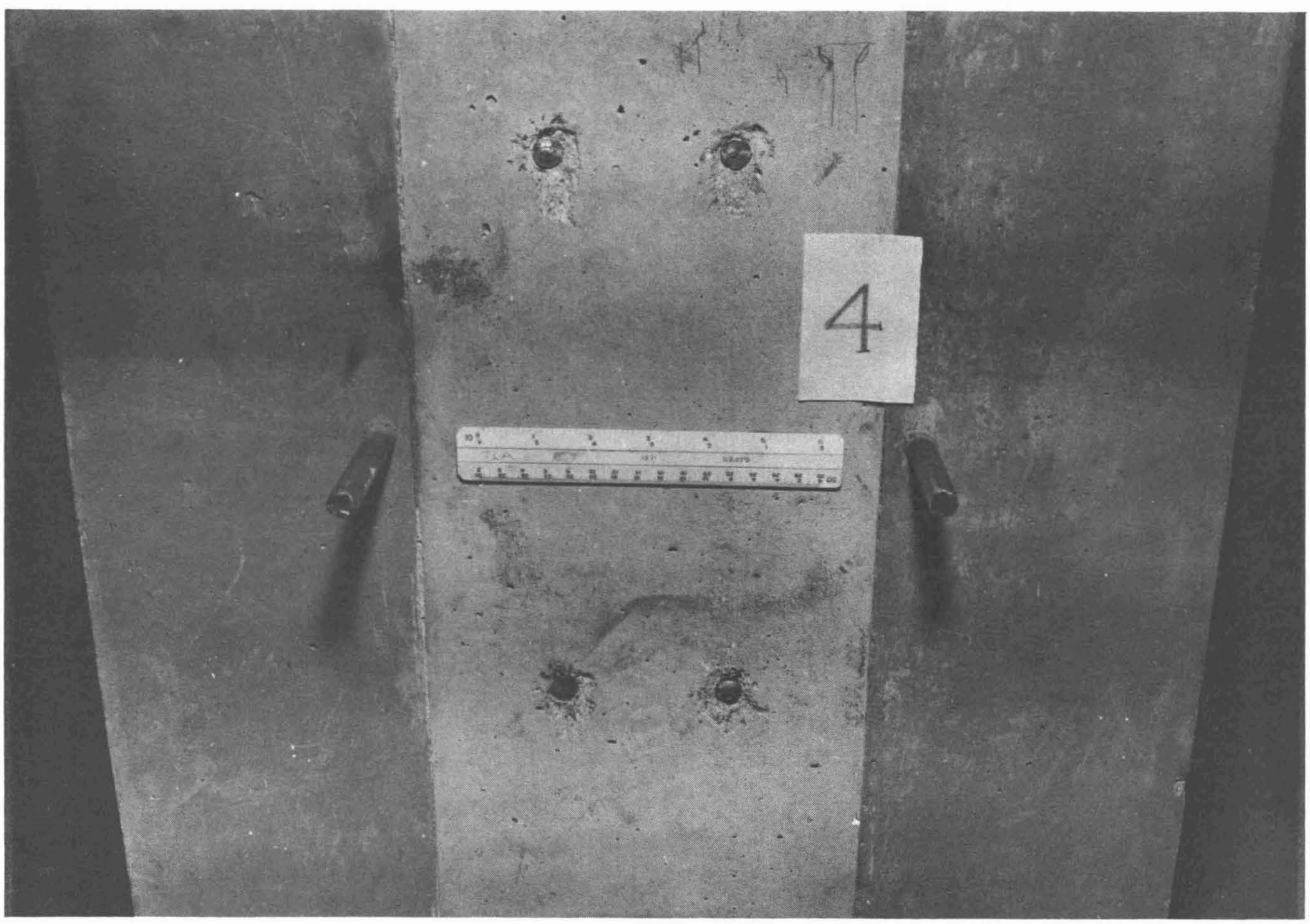


Fig. 12B

Fig. 13A

Fig. 13B

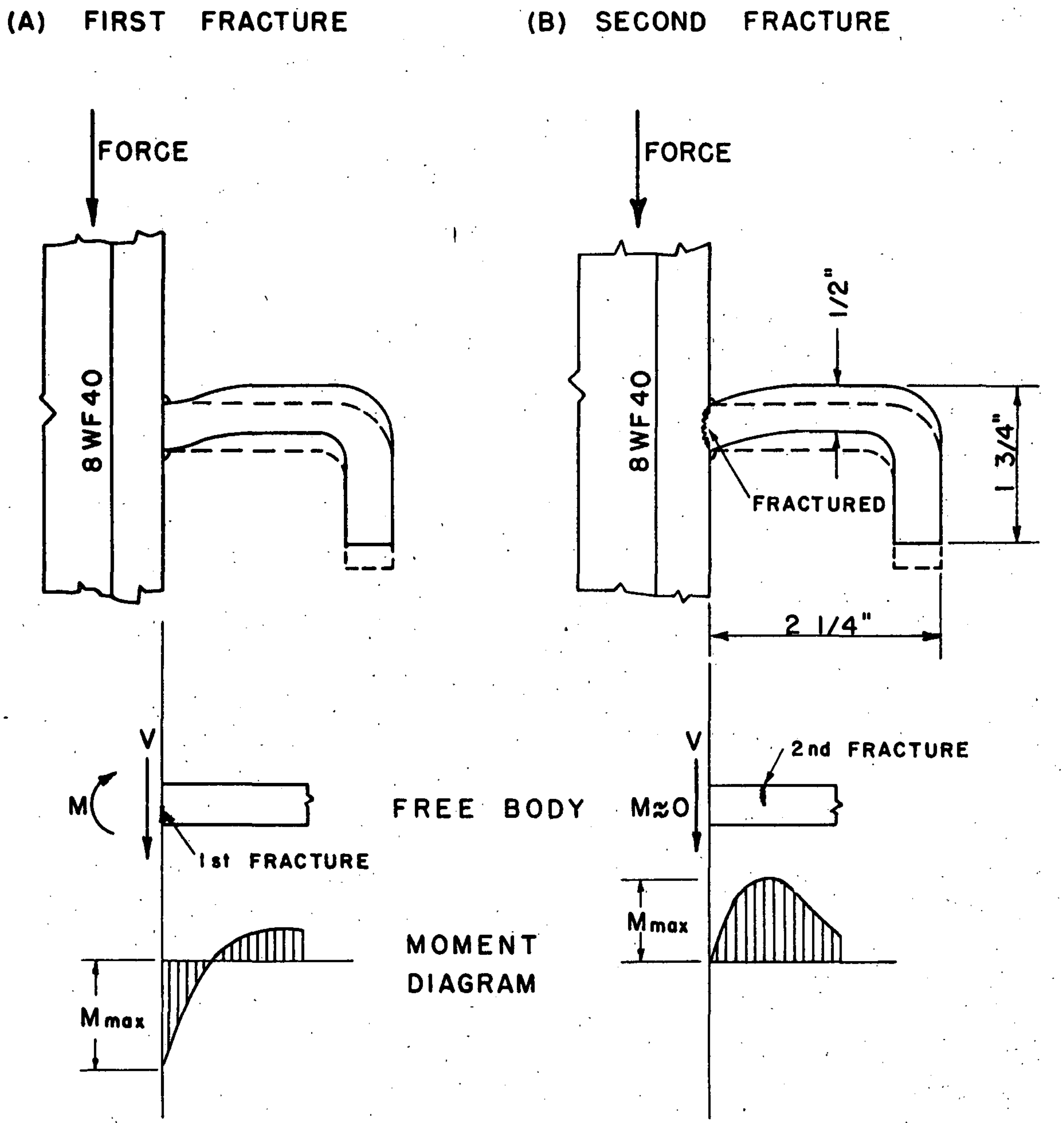


FIG. 14 - FAILURE SKETCH FOR STUDS OF SPECIMEN 8

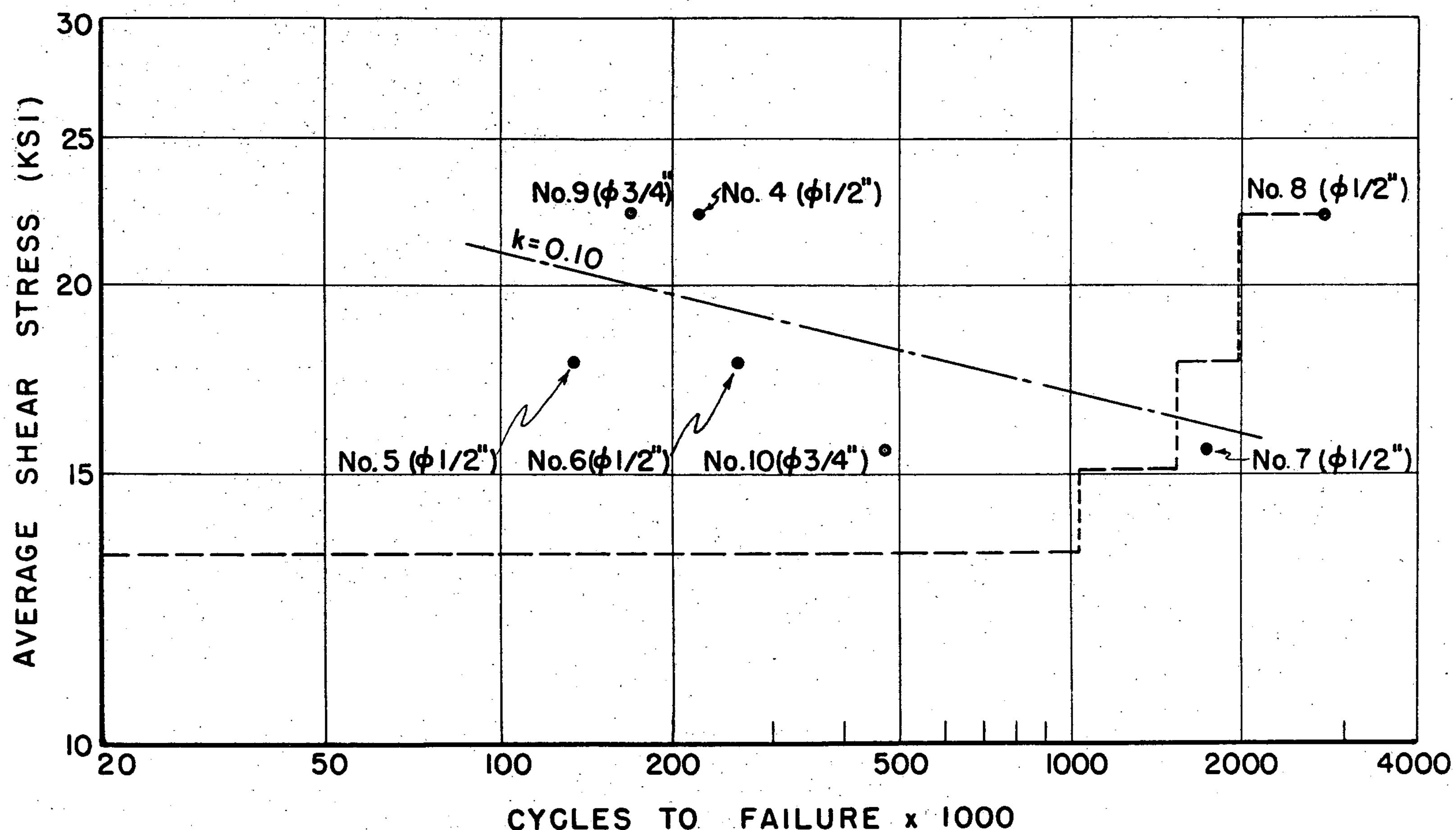


FIG. 15 - FATIGUE TEST RESULTS

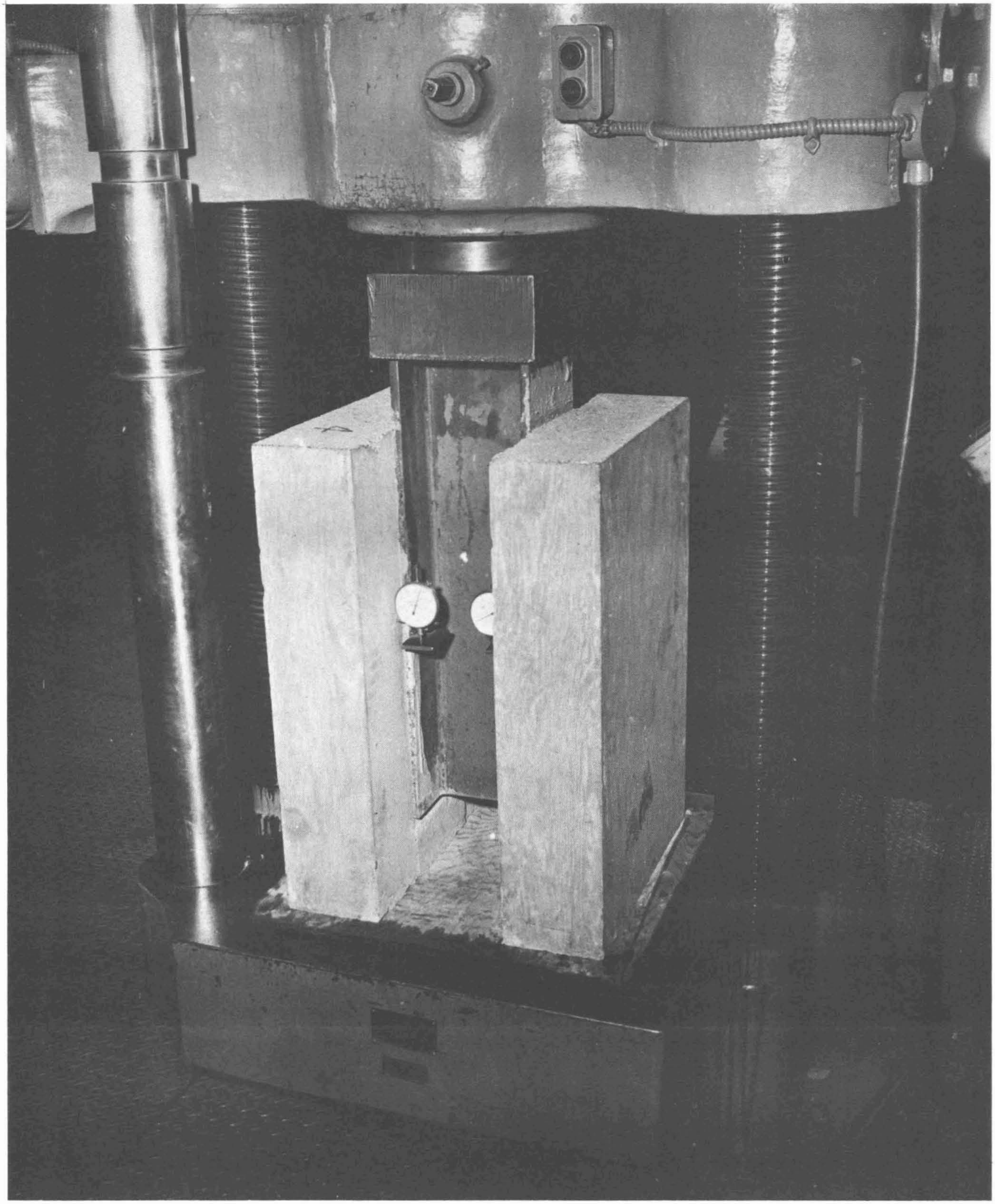


Fig. 16

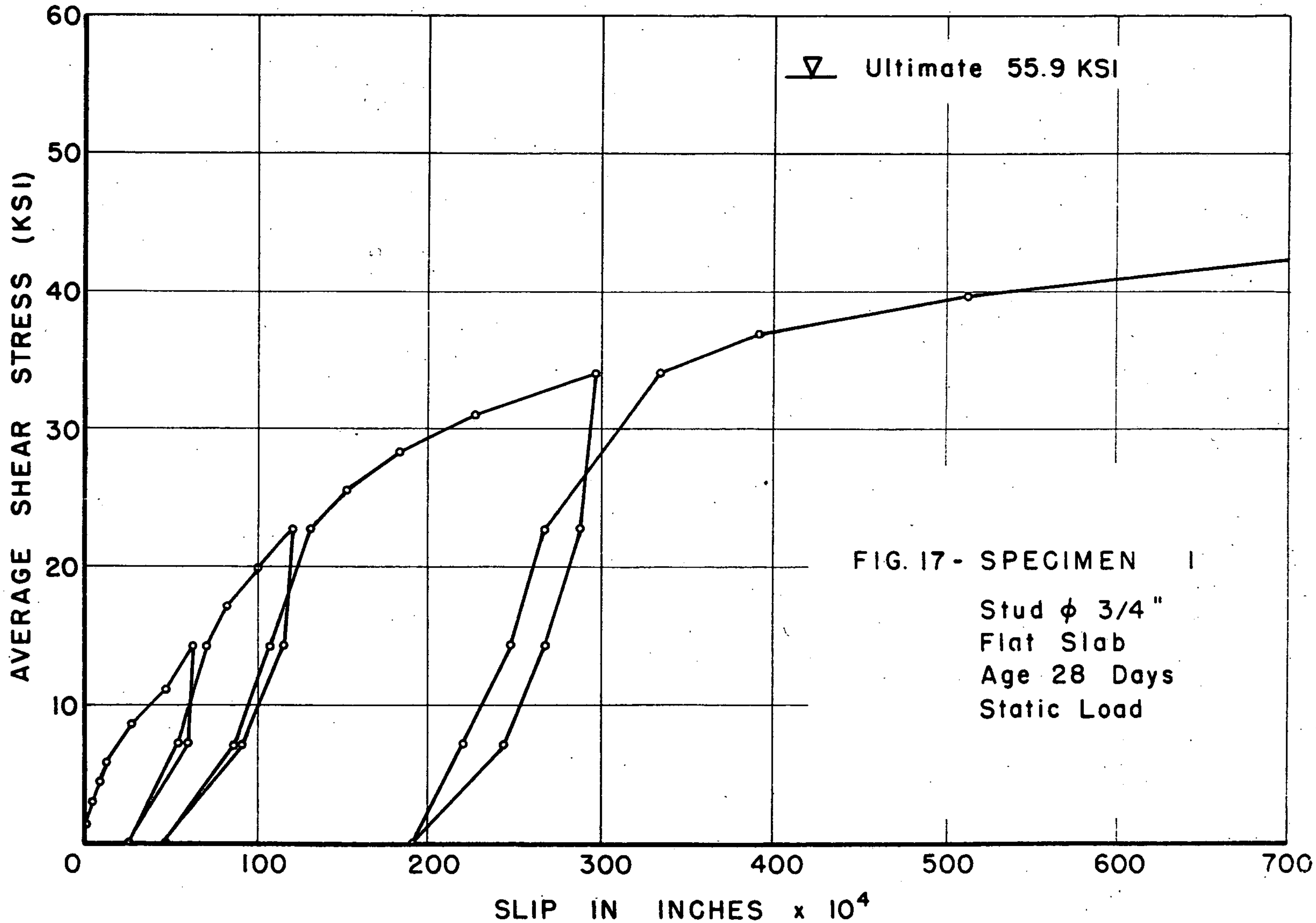


FIG. 17 - SPECIMEN I

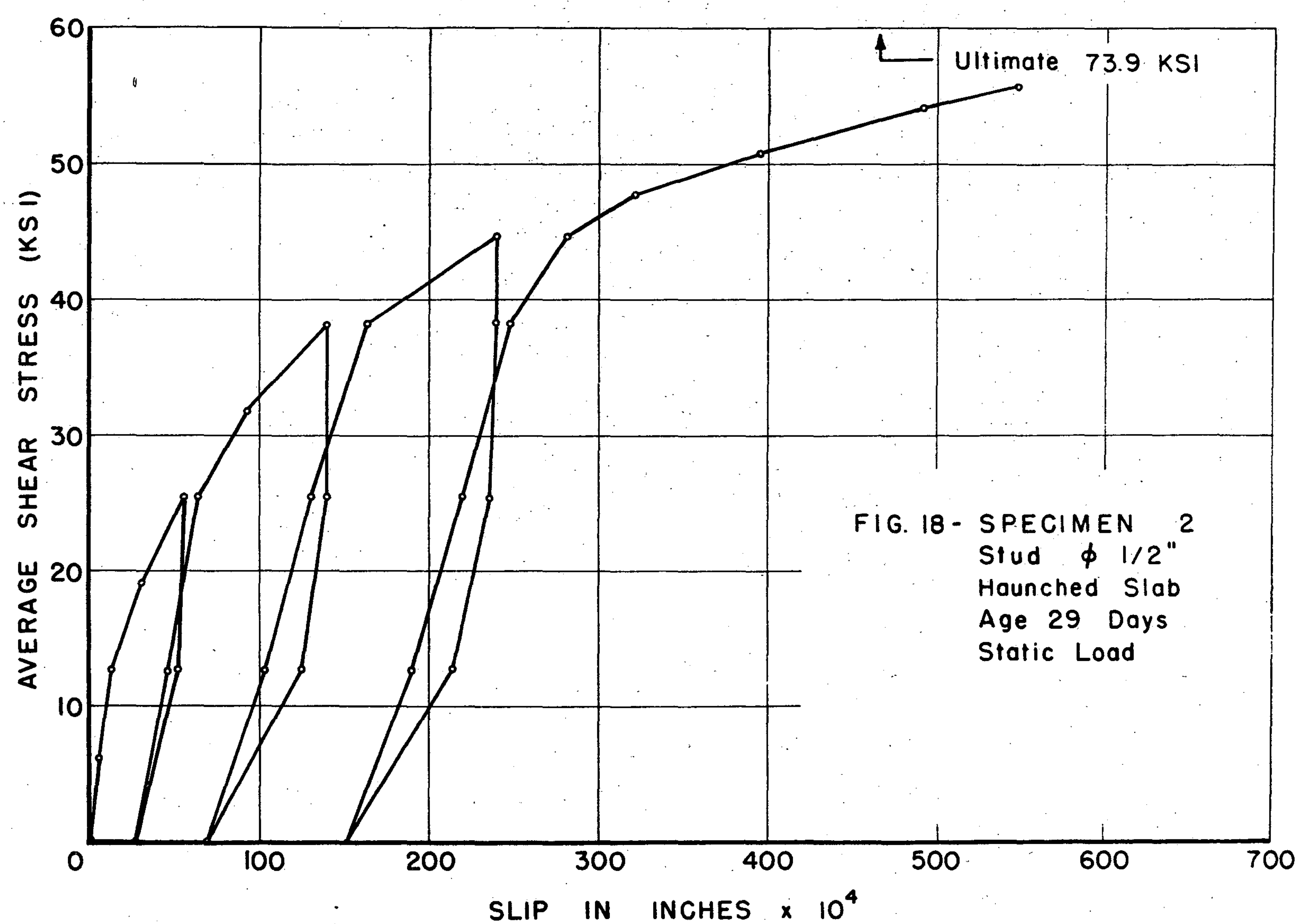


FIG. 18 - SPECIMEN 2
Stud ϕ 1/2"
Haunched Slab
Age 29 Days
Static Load

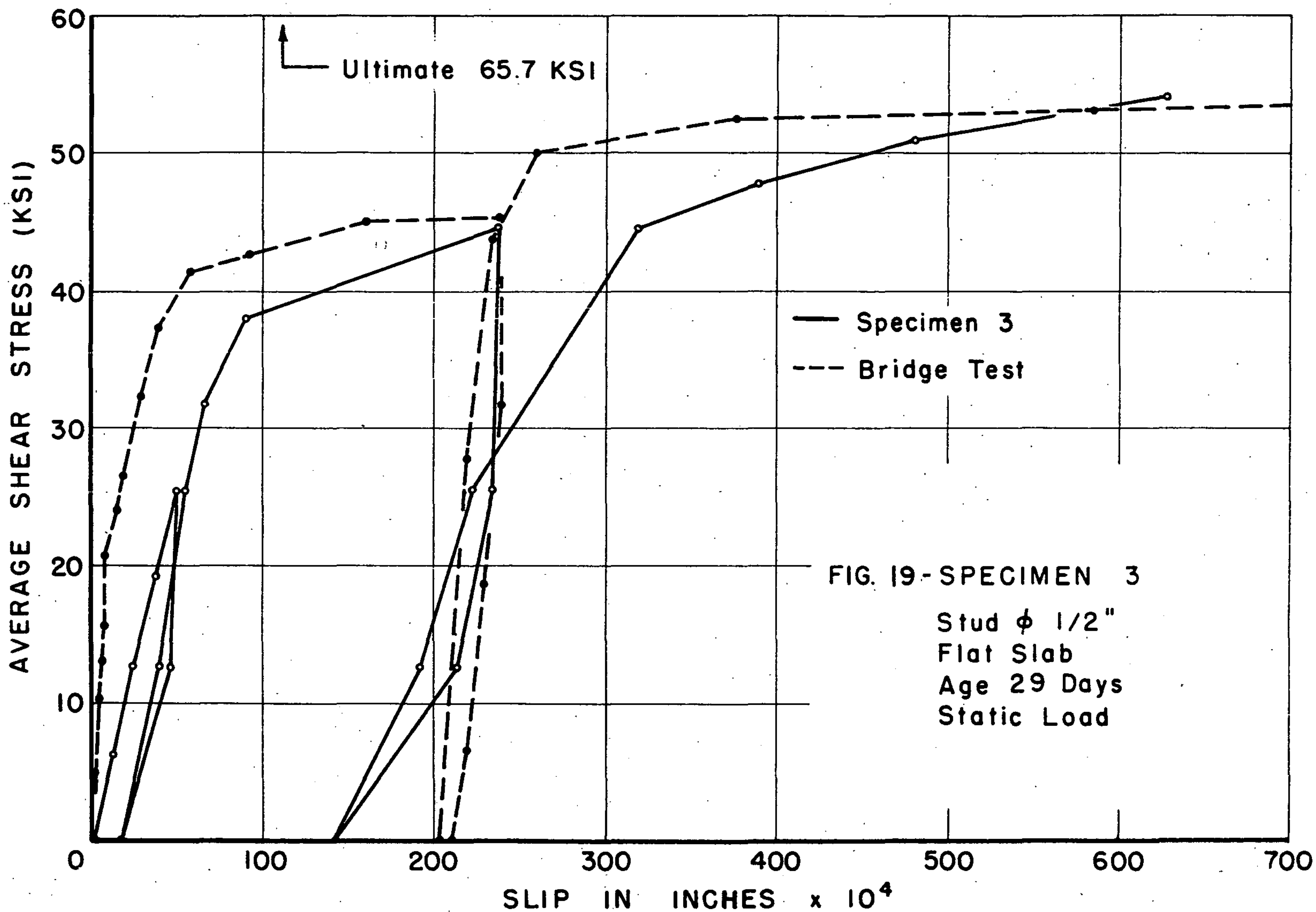


FIG. 19 - SPECIMEN 3
Stud ϕ 1/2"
Flat Slab
Age 29 Days
Static Load

TABLE I: DESIGNATION OF SPECIMENS

Specimen No.	Slab	Stud				Loading
		Number	Type	Diameter	Height	
#1	Flat	8	Straight	3/4"	4"	Static
#2	Haunched	8	L-Connector	1/2"	2-1/4"	Static
#3	Flat	8	L-Connector	1/2"	2-1/4"	Static
#4	Flat	8	L-Connector	1/2"	2-1/4"	Cyclic
#5	Flat	8	L-Connector	1/2"	2-1/4"	Cyclic
#6	Flat	8	L-Connector	1/2"	2-1/4"	Cyclic
#7	Flat	8	L-Connector	1/2"	2-1/4"	Cyclic
#8	Haunched	8	L-Connector	1/2"	2-1/4"	Cyclic
#9	Flat	8	Straight	3/4"	4"	Cyclic
#10	Flat	8	Straight	3/4"	4"	Cyclic

TABLE II: CYLINDER STRENGTH OF CONCRETE IN SPECIMENS

Cylinder No.	Age at Test (days)	Strength (psi)
1	6	2,650
2	6	2,970
	6	Average 2,810
3	29	5,270
4	29	5,100
5	29	4,600
6	29	5,360
	29	Average 5,080
7	41	6,200
8	41	5,760
9	41	5,930
10	41	5,780
	41	Average 5,920

TABLE III: RESULTS OF FATIGUE TESTS

Specimen No.	Stud Type	Max. Stress[a] (psi)	Min. Stress[a] (psi)	Cycles to Failure	Fatigue Strength (psi) $f_{100,000}$	$f_{600,000}$	$f_{2,000,000}$
#4	L-Connectors	22,300	2,900	223,200	24,200	18,800	17,900
#5	L-Connectors	17,800	2,200	134,200	18,300	15,400	13,600
#6	L-Connectors	17,800	2,200	261,000	19,700	16,400	14,500
#7	L-Connectors	15,600	1,900	1,748,000	20,500	17,300	15,400
				Average	20,700	17,000	15,400
#8	L-Connectors	22,300[b]	4,800[b]	2,815,000	---	---	---
#9	Straight	22,300	2,800	169,400	---	---	---
#10	Straight	15,600	1,700	474,000	---	---	---

(a) Load divided by cross sectional area of all studs

(b) For final 825,000 cycles (compare Fig. 9)