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INVESTIGATION OF STUD SHEAR CONNECTORS FOR
COMPOSITE CONCRETE AND STEEL T-BEAMS

By

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INTRODUCTION

In building and bridge floors of reinforced concrete supported on steel I-beams the slab and the beams are often connected so that they deform as a unit; the resulting structure may be considered as composed of a series of composite T-beams. Although the slab is usually connected to the I-beams through natural bond, such a connection is unreliable and may not provide composite action throughout the life of the structure (1).+ Thus, if the design calls for a composite structure, it is necessary to connect the slab to the I-beams by mechanical shear connectors fastened to the I-beams and embedded in the slab. Various types of shear connectors have been proposed and used (1); several of them have been found satisfactory through experimental investigations (2, 3, 4, 5, 6, 7, 8).

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+Numbers in parenthesis refer to items in the list of references.

The shear connectors are usually connected to the I-beams by welding. Although the welding of shear connectors presents no technical difficulties, it is ordinarily time consuming and it complicates the fabrication of beams; furthermore, if the welding has to be done in the shop, the shear connectors may create transportation and erection problems. It is desirable, therefore, to devise a shear connector which can be fastened rapidly to the I-beams both in the shop and at the construction site.

A rapid process known as electric arc stud welding is available for attaching steel studs from 3/8 to 20 in. long and having diameters from 6/32 to 1 1/4 in. (9). In this process the stud constitutes one electrode; an electric arc set up between the stud and the flange of the I-beam melts the end of the stud and the adjacent surface of the flange; then the molten end of the stud is forced into the pool of molten flange metal. The process may be accomplished in a few seconds with the aid of a stud welding gun. Portable equipment permits fastening of studs with equal ease in the shop or at the construction site.

This investigation was undertaken to study the behavior and load-carrying capacity of stud shear connectors. Tests were made on 12 push-out specimens. They paralleled in all important details earlier tests of channel shear connectors (8), since it was felt that in this manner some of the information obtained from the more extensive investigation of channels could be utilized in analyzing the results of the tests of stud connectors.

Four push-out specimens were made with granual flux filled studs of 3/4-in. diameter and two with similar studs of each of the following diameters: 1/2, 5/8, 1 and 1 1/4 in. Half of the specimens with diameters of the same size were tested with continuously increasing load applied in several increments until failure; in the tests of the remaining specimens the load was released a few times before failure occurred. In addition, the strength of concrete and the spacing of studs were varied to some extent.

TESTS OF PUSH-OUT SPECIMENS

Specimens

In response to bending, the concrete slab of a composite T-beam tends to slide along the flange of the I-beam; it is the function of the shear connectors to prevent such movement. The action of a connector may be simulated in the push-out specimen shown in Fig. 1. This type of specimen permits a convenient study of the behavior and characteristics of full size connectors. In the earlier study of channel connectors, such specimens have yielded results in good agreement with those obtained from tests of T-beams (8).

Each push-out specimen consisted of an 8-in. 48-lb wide flange steel beam 36 in. long, two rectangular concrete slabs 30x24x7 in. and four or eight studs. All studs for one specimen were of the same size, were stud welded to the I-beam on the same transverse cross-section, and were located symmetrically with respect to both vertical axes of beam symmetry. Equal number of studs was embedded in each slab.

A cross-section through a stud connector is included in Fig. 1 and a photograph of four studs welded to the I-beam is shown in Fig. 2. The stud connector is a short length of round steel bar welded to the I-beam at one end and having an enlarged head on the other end. The purpose of the head is to prevent the slab from lifting off the beam. The weld covers the full cross-section of the stud as is illustrated in Fig. 1.

Average dimensions of studs are listed for each push-out specimen in Table 1. The stud diameter varied from 0.50 to 1.25 in. and the height of the stud was approximately 4 in. for all studs. The diameter of the head was 0.2 to 0.5 in. larger than the stud diameter. The distance center to center of the studs, i.e. the spacing, was less than 2 in. in specimens 6A4 and 6B4, and approximately 4 in. in other specimens.

In all specimens an attempt was made to prevent bonding of concrete to the steel I-beam. For this purpose, the flanges of the I-beams were covered with a layer of cup grease prior to casting the concrete of the slab.

The slabs were cast horizontally to simulate the conditions in a composite beam. The horizontal casting required one-day delay between casting of the two slabs of one specimen. After the second slab was one day old, the specimen was placed in a moist room for six additional days of wet curing. During the period between the end of moist curing and the time of testing the specimens were stored in the laboratory. All specimens were tested about 28 days after the first slab was cast.

The slabs were reinforced with a single layer of wire mesh consisting of No. 10 wires spaced at 4 in. in each direction and placed approximately 2.5 in. below the outside surface of the slab.

Three or five 4 x 8-in. control cylinders were cast and cured with each slab. They were tested at about the same age as the corresponding specimen.

Materials

The concrete for the slabs was made of a standard brand of Type I Portland cement, Wabash River sand and Wabash River gravel. The fineness modulus of the sand was approximately 3.0 and the maximum size of the gravel was 1 inch. The proportions of the aggregates were 1:3.38:4.86 and the cement-water ratio was 1.3, all ratios by weight. The average strengths of the concrete in the slabs determined from the compression tests of control cylinders are listed in Table 2.

Since the studs were delivered to the laboratory already welded to the I-beam, the properties of the steel in the studs were determined from the tests of tensile coupons cut from the studs after the test of the push-out specimens. The coupons were cut from the free ends of the studs, i.e. from the ends with the head. They were 2 or 3 1/2 in. long shouldered specimens of circular cross-section with the straight test section approximately 1 1/4 or 2 1/4 in. long. Since the studs were permanently deformed during the test of the push-out specimens, only the tensile strengths and the reductions in area were determined. Except for two studs of specimen 5B2, the properties of all studs of one specimen were in close agreement: the average properties are listed in Table 2.

In order to determine the stress-strain characteristics of the steel in the studs, one tensile coupon was prepared from the original material used in making the 1-in. studs. It was round shouldered specimen 4 1/2 in. long with the straight test portion 3 1/4 in. long and 1/2 in. in diameter. Strains were measured with the aid of two electric resistance strain gages mounted on opposite sides of the specimen. The stress-strain curve obtained from this test is shown in Fig. 3. The yield point stress determined by the halt of the machine was 53,400 psi, and the ultimate strength was 70,300 psi. The reduction in area was 63%.

Test Procedures

All specimens were tested with static loading in an Olsen screw-type machine of 300,000-lb capacity. The specimen was placed in the testing machine with the lower ends of the slabs bedded in plaster of Paris. The load was applied to the upper end of the steel beam by the head of the machine through a spherical block and a steel distributing plate. To improve the distribution of load, strips of lead were inserted between the steel plate and the flanges of the beam. In placing the specimens in the testing machine, care was taken to insure concentric loading: the maximum eccentricity in the direction of the flanges of the I-beam was 1/16 in. and in the direction of the web 1/4 in.

In the tests of specimens designated with letter A the load was applied in increments varying from 2,000 to 10,000 lb until failure occurred (type A loading). In the tests of specimens designated with letter B the load was applied also in increments varying from 2,000 to 10,000 lb, but was released at several load levels before failure (type B loading). The load was usually released and recovered in three steps of varying magnitude.

In all tests slip between the slabs and the beam was measured by four 0.001-in. dial indicators at the level of the shear connectors (Fig. 1). The dials were attached rigidly to the beam with the stem bearing against brackets glued to the slabs. Readings were taken after each increment of load. The dials were read to the nearest 0.0001 in. at low loads and to the nearest 0.001 in. at high loads.

Specimens 5A2 and 6A2 were accidentally loaded while being placed in the testing machine. In specimen 5A2 this load was approximately 15,000 lb; in specimen 6A2 the magnitude of this load was not noted but the load slip-curves have indicated that it was approximately 40,000 lb. In both cases, the load was removed immediately. The data for both specimens were corrected for the effects of the overload.

Test Data

The results of the tests of push-out specimens are presented in Figs. 4, 5, 6, 8 and 9 and in Table 3. The slips reported in the figures and the table are the averages of readings on four dials of one specimen; the individual dial readings were in reasonable agreement in all tests and the differences between the readings were judged insignificant. The reported loads are the total loads applied to the push-out specimen divided by the corresponding number of studs; it is assumed in the following discussion that this load was carried by one stud. Such procedure involves two assumptions:

(1) that the load was transmitted from the I-beams to the slabs only through the studs, and (2) that the load was distributed evenly between the individual studs. Although neither of these assumptions is strictly correct, the error involved is believed to be within the range of the accuracy of the test data.

Load-slip curves for specimens with four studs subjected to type A loading are shown in Fig. 4. The specimens differed only in the stud diameter and in concrete strength. Since the variations of the latter were insignificant (except for 10A2 with concrete strength lower than that for other specimens) it can be seen from

Fig. 4 that the capacity of the stud connector increases rapidly with the diameter of the stud.

The results of the tests with type B loading are illustrated in Fig. 5, which includes a typical load-slip curve for type B loading and the corresponding curve for type A loading. The load-slip curve for specimen 5B2 shows that at low loads practically all slip was recovered on the removal of the load. At high loads large residual slips were observed, and the slips measured during the removal and reapplication of load were considerably larger than those measured at first application of load. The envelope curve for type B loading was, however, in good agreement with the load-slip curve for the corresponding specimen subjected to type A loading.

The increase of the residual slip with increase of the load applied prior to the load removal is illustrated in Fig. 6. In this figure, the maximum load before release is plotted against the residual slip measured after the load release; all four-stud specimens subjected to type B loading are included. Since the variations in the strength of concrete were insignificant (except for specimen 10B2 having low strength) the only important variable included in Fig. 6 is the diameter of the stud. It can be seen that at low loads only very small residual slips have taken place thus indicating that permanent deformations were very small. It is believed that at these loads inelastic action was confined to the concrete. At higher loads, however, residual slip increased greatly with small increments of load indicating the presence of large inelastic deformations; both plastic deformations of concrete and yielding of the steel of the studs probably occurred at these loads. It can be seen further that the residual slip curves for specimens with studs of small diameter (4B2, 5B2 and 6B2) exhibit a sharp break as the rate of increase of residual slip changes from slow to fast. For specimens with large stud diameter (8B2 and 10B2), however, the load-slip curves show a more gradual change in the rate of slip increase. This difference in the shape of the residual slip curves is believed to be caused by the yielding of the studs. The difference between the load at first yielding and the load at which yielding penetrated the full depth of the cross-section is smaller for studs of small diameter than for studs of large diameter. Since very large slip can occur only after the yielding penetrated deep into the cross-section, the residual slip curve will show a sharp transition when the difference between the two loads is small and a gradual transition when the difference is large.

The loads corresponding to the transition from small to large residual slips were determined for specimens with studs of small diameter (4B2, 5B2, and 6B2) by extrapolation shown in Fig. 6; they are listed in Table 3 as critical loads. It may be seen in this table that the corresponding slips were of the order of 0.01 in. and the corresponding residual slips were smaller than 0.003 in. For specimens with studs of large diameter (8B2, 10B2) it was not possible to follow a similar procedure because of the gradual transition from small to large residual slips; therefore the residual slip of 0.003 in. was selected as the criterion for determining the critical load for these specimens. It may be seen in Table 3 that the corresponding maximum slips were of the same order of magnitude as for specimens with studs of small diameter.

Before the critical load was reached, the specimens held the applied load while the slip readings were taken. As the loading continued beyond this load, however, it became increasingly difficult to stabilize the load. Before the maximum load was reached, some separation was observed of the slabs from the beam. The maximum load could not be stabilized: a few seconds after reaching the maximum level, the load started dropping off and final failure followed.

Six specimens failed by breaking off one to three studs at their welds at loads between 2.35 and 2.78 times the corresponding critical loads (Table 3). The slips at failure were 16 to 37 times the slips at critical loads.

An inspection after the test has shown that the studs have undergone large inelastic deformations (Fig. 7), but the slabs remained undamaged except for permanent deformations and shallow surface spalling under the connectors. The ultimate loads were proportional to the cross-sectional areas of the studs.

The six remaining specimens failed by tensile cracking of both slabs at loads between 2.01 and 2.22 times the critical. The slips at failure were 7 to 12 times the slips at critical loads. Inspection after the tests has revealed only small permanent deformations of the studs (Fig. 2). All six remaining specimens failed at approximately the same total load; obviously, their strength was governed by the slabs. Had the slabs been thicker or reinforced more heavily, the ultimate loads of these specimens would have been higher.

Effect of Concrete Strength

The strength of concrete for corresponding specimens (e.g. 4A2 and 4B2) differed up to 550 psi. These variations permit a study of the effect of concrete strength on the load capacities of studs at constant slip; the results are shown in Table 4. It may be seen that the load capacity of a stud connector at constant slip increases with increasing strength of concrete and that the increase is approximately proportional to $\sqrt{f'_c}$.

Effect of Stud Spacing

Specimens 6A4 and 6B4 had twice as many studs as specimens 6A2 and 6B2; the spacing of studs in the former specimens was approximately one half of that in the latter pair, namely 2 inches as compared to 4 inches. The load-slip curves for specimens 6A4 and 6A2, and the residual slip curves for specimens 6B4 and 6B2 are plotted in Fig. 8. It can be seen that the spacing of studs has no significant effect on either the load-slip or the residual slip curves. The same conclusion may be drawn from the data included in Table 3.

Effect of Stud Diameter

All test data show a definite increase of the load-carrying capacity of stud connectors with increasing diameter of the studs. The quantitative relationship between the critical loads and the diameter is illustrated in Table 5 and in Fig. 9.

It has been shown in a preceeding paragraph that the effect of the strength of concrete on the load capacity at constant slip is approximately proportional to $\sqrt{f'_c}$. Although the effect of concrete strength on critical loads may differ from that on the loads at constant slip, it undoubtedly is of a similar order of magniutde. It was decided, therefore, to correct the measured critical loads to one concrete strength by multiplying each load with the factor $\sqrt{4000/f'_c}$. The resulting values are listed in Table 5 as "corrected Q_{cr}"; also listed are the ratios of the corrected loads to the stud diameter and to the square of the diameter. The corrected loads are plotted against the stud diameter in Fig. 9.

It may be seen from Table 5 and Fig. 9 that the critical loads for studs of 1/2, 5/8 and 3/4-in. diameter are proportional to the square of the diameter; and for studs of 1.0 and 1.25 in. diameter, the critical loads are proportional to the diameter. Constants of proportionality equal to 21 kips/in^2 and to 20 kips/in. yield values in good agreement with the test data (Fig. 9).

Comparison with Channel Connectors

A qualitative comparison between the behavior of channel and stud connectors can be made with the aid of the curves in Fig. 5, 6, and 10. Fig. 10 shows slip and maximum strain data for a 6-in. long channel connector made of 4-in. 13.8-lb rolled steel channel; the curves were obtained from the test of a push-out specimen*. It may be seen that the load-slip and residual slip curves for the two types of connectors exhibit identical characteristics.

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*Fig. 10 was reprinted from reference 8, Fig. 23, p. 47.

In addition to the load-slip characteristics, the manner of the tension crack formation in the slabs of specimens which failed in the slab, the appearance of the connectors after failure and the effects of concrete strength also point out the similarity in the behavior of studs and channels. The behavior of a channel connector is characteristic of a flexible dowel (8): in view of the similarities mentioned above the behavior of a stud shear connector may be classified in the same category.

USEFUL LOAD CAPCAITY OF STUD CONNECTORS

Analytical expressions.

The function of a shear connector is to prevent slip between the concrete slab and the steel I-beams. However, tests of composite T-beams (7,8) have shown that small amounts of slip, such as those corresponding to elastic deformations of mechanical connectors, cause practically no decrease in the degree of composite action. Thus, in order to insure good composite action a steel shear connector must not yield and the adjacent concrete must not undergo any appreciable inelastic deformations.

It has been shown in the discussion of the test data that only negligible inelastic deformations were observed in tests of stud connectors until the critical load was reached. Thus the critical loads represent the useful load capacity of a stud connector. Analytical expressions for critical loads may be determined with the aid of data obtained from tests of push-out specimens.

Within the limits of these tests, the critical loads for stud connectors are proportional to the square root of the cylinder strength of concrete. Furthermore, the critical loads for stud connectors with diameters smaller than 1 in. are proportional to the square of the diameter and for stud connectors with diameters larger than 1 in. are proportional to the diameter of the stud. For $f'_c = 4000$ psi, constants of proportionality of 21 kips/in^2 and 20.0 kips/in. yield good agreement with the test data. Accordingly, the critical loads may be expressed as follows:

$$\text{For } d < 1 \text{ in:} \quad Q_{cr} = 5.25\, d^2 f'_c \sqrt{\frac{4000 \text{ psi}}{f'_c}} \tag{1}$$

$$\text{For } d \geqq 1 \text{ in:} \quad Q_{cr} = (5 \text{ in.})\, d\, f'_c \sqrt{\frac{4000 \text{ psi}}{f'_c}} \tag{2}$$

It may be noted from the earlier discussion of the test data that the critical loads for studs of small diameter (Eq. 1) are governed by yielding of steel, whereas the

the critical loads for studs of large diameter (Eq. 2) are governed by a limiting inelastical deformation of concrete chosen arbitrarly as a residual slip of 0.003 in.

Limitations.

The applicability of Eq. 1 and 2 is limited by the extent of the supporting test data. Yield point of the steel, strength of concrete, diameter and height of the stud, and spacing between the studs may reasonably be expected to affect the critical loads. The limits for each of these variables are discussed separately.

The yield point of the stud steel was not determined for the individual specimens, but the ultimate strengths in conjunction with the yield point for one specimen indicate that the yield point of the steel in the studs ranged between 48,000 and 56,000 psi. Eq. 1 is believed to represent the load corresponding to first yielding and it seems reasonable to expect that the load at first yielding would increase with increasing yield point of the steel. Thus the use of Eq. 1 should be limited to studs of steel having a yield point of at least 50,000 psi. The same limit should be applied to Eq. 2 since studs having a lower yield point may begin to yield before the limiting residual slip of 0.003 in. has been reached.

The strength of concrete varied from 3190 to 4390 psi; its effect is included in Eq. 1 and 2. The effect of concrete strength on the load capacity of stud connectors is approximately the same as the effect of concrete strength on the load capacity of channel shear connectors (10). Since the tests of channel connectors included concrete strengths varying from 1970 to 6320 psi, an extrapolation of the stud data beyond the range of concrete strengths encountered in the tests of stud connectors is believed to be warranted. A lower limit of $f'_c = 2500$ psi and an upper limit of $f'_c = 5000$ psi are suggested for use of Eq. 1 and 2. For concretes with $f'_c > 5000$ psi Eq. 1 and 2 should yield safe values if $f'_c = 5000$ psi is substituted.

Stud diameters ranged from 1/2 to 1 1/4 in. Eq. 1 was derived from tests of studs with diameter smaller than 1 in. Accordingly, it is applicable only to studs with $d < 1$ in.: no lower limit on diameter appears to be necessary. Equation 2 was derived from tests of studs 1 and 1 1/4-in. diameter; it should not be applied outside these limits.

All studs were 4 in. high so that no experimental evidence is available on the effects of the stud height. However, it has been pointed out that a stud is a flexible dowel and it is well known that the load capacity of a dowel is either independent of or increases with its length. Thus Eq. 1 and 2 are applicable to studs 4 in. high and higher.

Push-out specimens with stud connectors spaced at 2 and 4 in. center to center were tested. Since no effect of spacing on the stud capacity was observed and the 2-in. spacing represents a practical lower limit, Eq. 1 and 2 may be applied without regard to spacing requirements.

DESIGN RECOMMENDATIONS

Extensive tests of flexible channel shear connectors reported earlier (7,8) included two types of specimens: push-out specimens and T-beams. Measurements of slip between the slab and the I-beam, and of strains on the surface of the shear connectors have shown that the behavior of a channel shear connector in a composite T-beam is essentially the same as the behavior of similar connectors in a push-out specimen. This finding permitted the application of the results of tests of push-out specimens to the design of connectors for composite T-beams (10).

The push-out specimens for the tests of stud shear connectors were in all important details identical with the push-out specimens utilized in the earlier tests of channel connectors. Furthermore, the test results have shown that the behavior of the two types of connector is similar. In view of these facts, it seems reasonable to assume that the behavior of stud connectors in a composite T-beam is essentially the same as in a push-out specimen. This assumption permits the design of stud shear connectors for composite T-beams to be based on the results of the tests of push-out specimens.

The tests of push-out specimens have shown that the useful capacity of a stud connector is equal to its critical load. Thus the full composite action will be retained only while the load transferred by one stud is smaller than the critical load determined from Eq. 1 and 2.

The design capacity of a stud connector may be computed from the critical load:

$$Q_{des} = \frac{Q_{cr}}{F.S.} \tag{3}$$

where Q_{cr} is obtained from Eq. 1 and 2 and F.S. designates the factor of safety. The magnitude of the factor of safety depends on the design requirements for the composite T-beams. Expressions for factors of safety which assure composite action at all levels of loading up to ultimate load may be found in a paper on the design of channel shear connectors.*

*See Eq. 11 and 15 in reference 10.

CONCLUDING REMARKS

Tests pf round steel studs were made to determine the behavior and load carrying capacity of stud shear connectors. Studs of five diameters ranging from 1/2 to 1 1/4 in. were tested in twelve push-out specimens.

The tests have shown that a steel stud is suitable for use as a shear connector in composite concrete and steel construction. The behavior of a stud shear connector is similar to that of a flexible channel shear connector. The useful load-carrying capacity of a stud connector is determined either by first yielding of the steel or by large inelastic deformations of concrete. The corresponding "critical load" may be computed from empirical equations presented in this report.

If the critical load is exceeded, the stud connector permits large slip between the steel beam and the concrete slab. It is recommended, therefore, that the design load for a stud shear connector be based on its critical load.

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TABLE 1

DIMENSIONS OF STUDS*

Specimen**	Stud d, in.	Diameter of Head D, in.	Diameter of Weld d', in.	Height of Stud H, in.	Thickness of Head t, in.	Edge Distance+ a, in.	Spacing^x u, in.
4A2	0.500	0.69	1.00	4.00	0.50	1.88	3.86
4B2	0.500	0.69	1.00	4.00	0.50	1.88	3.86
5A2	0.625	0.81	1.13	3.94	0.50	1.81	3.87
5B2	0.625	0.81	1.13	3.94	0.50	1.81	3.87
6A2	0.750	0.94	1.25	3.90	0.50	1.75	3.87
6B2	0.750	0.94	1.25	3.90	0.50	1.75	3.87
6A4	0.750	0.94	1.25	4.00	0.50	0.83	1.86
6B4	0.750	0.92	1.25	4.08	0.50	0.76	1.91
8A2	1.000	1.25	1.50	3.98	0.75	1.62	3.80
8B2	1.000	1.28	1.50	3.97	0.75	1.62	3.80
10A2	1.250	1.75	1.75	3.84	0.75	1.38	4.12
10B2	1.250	1.73	1.75	3.94	0.75	1.38	4.12

* All values are averages for all studs of each specimen; differences between individual studs were negligible.

** In the designation of specimens the first number gives the stud diameter in eighths of an inch; the letter designates the type of loading (A-continuously increasing, B-intermittently released); and the last number gives the number of studs in one slab.

\+ Distance from the edge of the I-beam to the nearest face of the stud (Fig. 1)

x Distance center to center of studs.

TABLE 2

PROPERTIES OF MATERIALS

Specimen	Compressive Strength of Concrete[x]			Steel of Studs[+]		
	Slab 1,[*] psi	Slab 2, psi	Average Slabs 1&2, psi	Coupon Diameter, in.	Ultimate Strength, in.	Reduction in Area, percent
4A2	3520	4160	3840	1/4	70,700	43
4B2	4110	4660	4390	1/4	70,800	47
Average			4120		70,700	45
5A2	3440[xx]	4140	3790	3/8	68,000[++]	36
5B2	3780	4720	4250	3/8	63,300	37
Average			4020		65,700	36
6A2	3880	3850	3870	1/2	69,900	37
6B2	4170	4310	4240	1/2	70,400	37
Average			4060		70,100	37
6A4	3510	3200	3360	1/2	67,700	62
6B4	3140	3380	3260	1/2	69,300	62
Average			3310		68,500	62
8A2	3980	3530	3760	3/8	73,600	64
8B2	4320	4130	4230	3/8	73,600	64
Average			4000		73,600	64
10A2	3060	3320	3190	1/2	63,800	63
10B2	3300	3700	3500	1/2	63,600	64
Average			3350		63,700	64

[x] Compressive strength of each slab is the average of three or five 4x8-in. cylinders cast and cured together with the corresponding slab. Cylinders were moist cured for 7 days and tested at the age of approximately 28 days.

[+] Determined from tension tests of coupons cut from studs after the test of the push-out specimen. All coupons were of circular cross-section 2 or 3 1/2-in. long. One coupon was cut from each stud. The values are averages for all coupons of the same push-out specimen.

[*] Slab cast first.

[xx] Average for two cylinders only.

[++] Studs located in Slab 1 had ultimate strength of 55,100 and 59,200 psi.

TABLE 5

SUMMARY OF TEST RESULTS[x]

Specimen	Load-Carrying Capacity in kips at Average Slip of			Critical Load			Ultimate Load		
	0.003 in.	0.006 in.	0.020 in.	Load, kips	Slip, in.x10^3	Residual Slip, in. x 10^3	Load, kips	Slip, in.x10^3	Type of Failure[+]
4A2	3.0	4.6	8.3	....	....	....	14.4	163	Weld
4B2	2.9	4.3	8.0	5.8	9.8	2.3	13.9	170	Weld
Average	3.0	4.5	8.2				14.2	167	
5A2	3.9[xx]	6.3	11.8	....	....	....	23.8	319	Weld
5B2	4.7	7.3	12.6	9.6	12.0	2.5	22.5	279	Weld
Average	4.3	6.8	12.2				23.2	299	
6A2	4.8[xx]	7.8[xx]	13.2	....	....	....	32.0	246[++]	Weld
6B2	5.7	8.7	15.4	11.7	10.2	1.9	32.5	382	Weld
Average	5.3	8.3	14.3				32.5		
6A4	5.4	8.1	13.6	....	....	....	21.2	79	Slab
6B4	5.4	7.8	13.5	11.3	13.2	2.7	22.5	99	Slab
Average	5.4	8.0	13.6				21.9	89	
8A2	8.9	12.6	22.6	....	....	....	42.0	90*	Slab
8B2	10.7	15.4	24.8	20.2	11.8	3.0	45.0	138	Slab
Average	9.8	14.0	23.7				43.5		

TABLE 3 (continued)

10A2	10.0	14.7	27.1	....	....	....	50.0	109	Slab
10B2	10.8	15.8	28.9	23.3	13.1	3.0	47.5	100	Slab
Average	10.4	15.3	28.0				48.8	105	

x All loads are for one stud; they are equal to the respective test loads divided by the total number of stud connectors in the push-out specimen. Slips are averages of four dial readings.

+ Failures occurred either by shearing off at the welds of 1 to 3 studs, or by tensile cracking of both slabs.

xx Test data corrected for the effect of overloading.

++ Slip at the load of 30 kips per stud.

* Slip at the load of 40 kips per stud.

TABLE 4

EFFECT OF CONCRETE STRENGTH

Specimens	Ratio of Concrete Strength f'_c	Ratio of $\sqrt{f'_c}$	Ratio of Load Capacities at Average Slip of 0.0C3 in.	0.006 in.	0.020 in
4B2/4A2	1.14	1.07	0.90	0.87	0.89
5B2/5A2	1.12	1.06	1.15	1.09	1.01
6B2/6A2	1.09	1.05	1.12	1.08	1.11
6A4/6B4	1.03	1.02	0.98	1.00	0.99
8B2/8A2	1.12	1.06	1.13	1.15	1.04
10B2/10A2	1.10	1.05	1.03	1.03	1.02
Average	1.10	1.05	1.05	1.04	1.01

TABLE 5

CRITICAL LOADS[x]

Specimen	Stud Diameter d, in.	Concrete Strength f'_c, psi	Critical Load Measured kips	Critical Load Corrected Q_{cr},[+] kips	$\frac{Q_{cr}}{d}$	$\frac{Q_{cr}}{d^2}$
4B2	0.500	4390	5.8	5.5	11.0	22.0
5B2	0.625	4250	9.6	9.3	14.9	23.8
6B2	0.750	4240	11.7	11.4	15.2	19.9
6B4	0.750	3260	11.3	12.5	16.9	22.6
8B2	1.000	4230	23.3	19.4	19.5	19.5
10B2	1.250	3500	20.0	24.9	19.9	16.0

[x]All loads are for one stud.

[+]Measured loads multiplied by $\sqrt{4000/f'_c}$.

Note:- Specimens 6A4 and 6B4 had four stud connectors in each slab

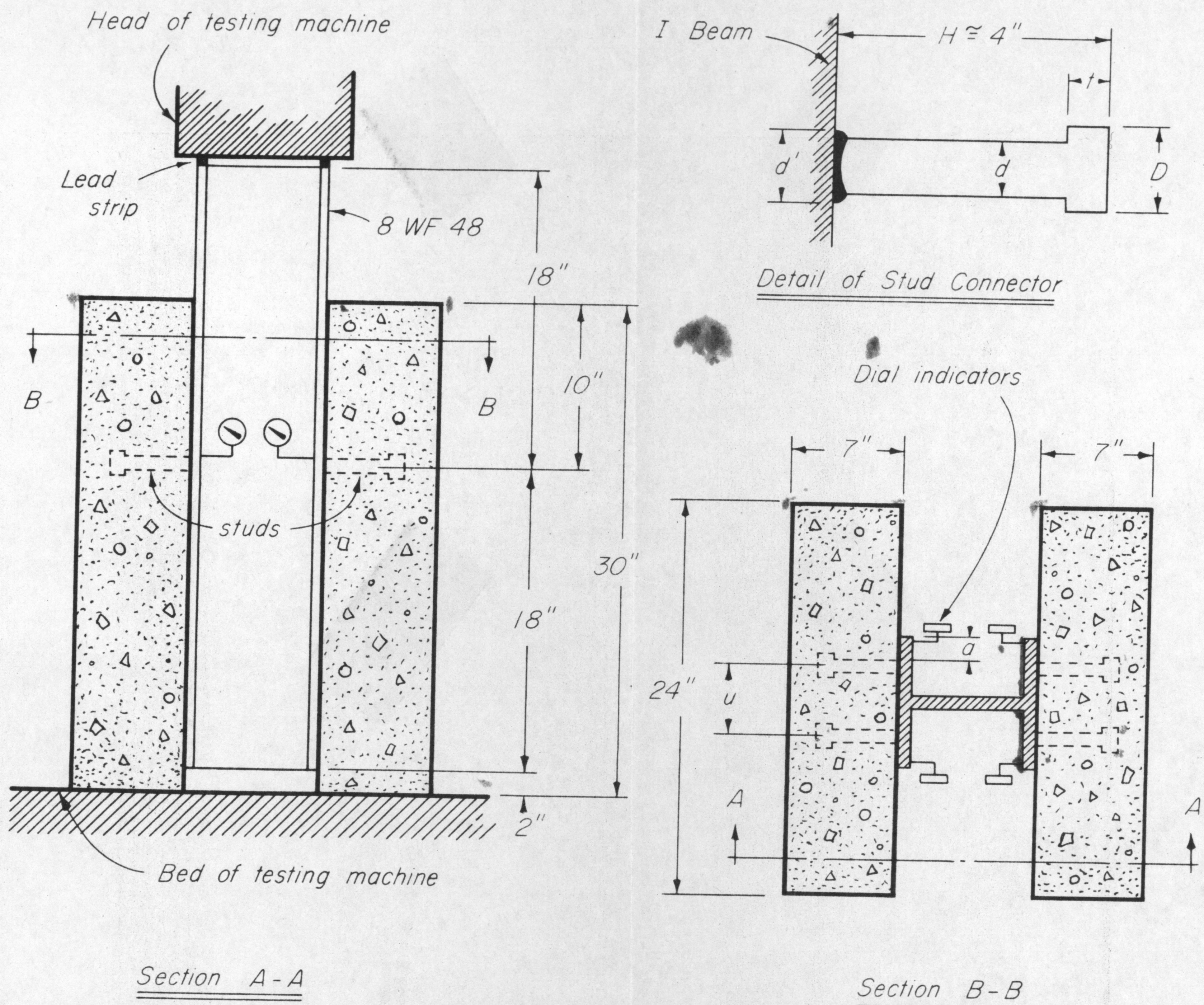

FIG. 1 DETAILS OF PUSH-OUT SPECIMENS

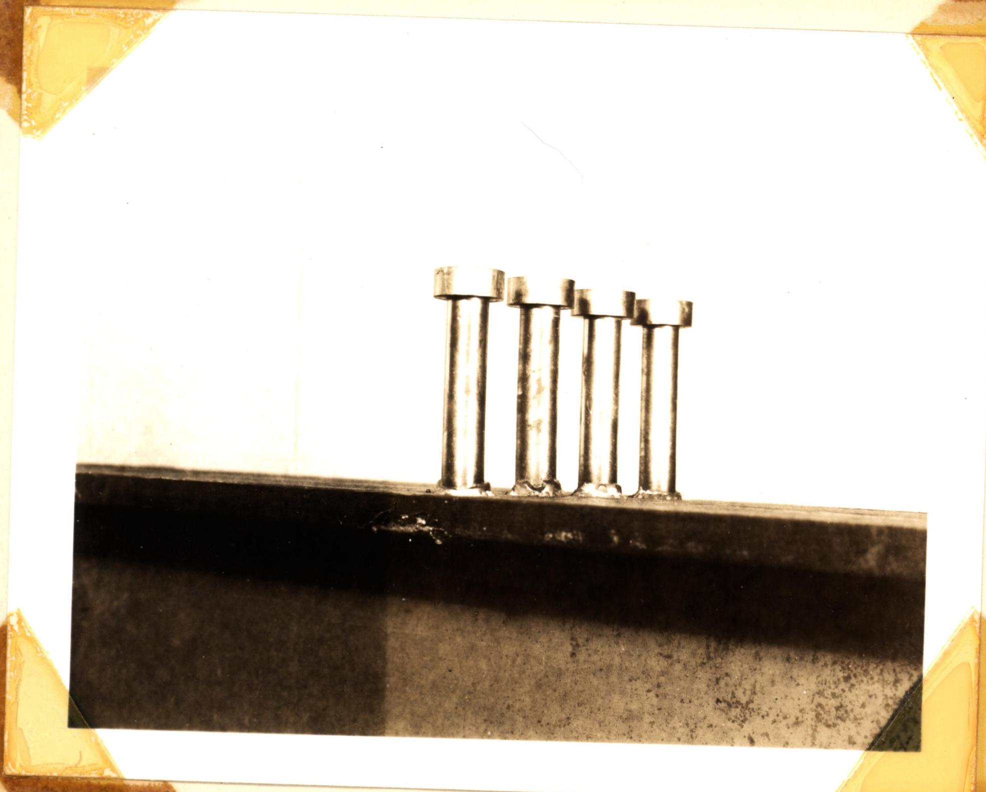

FIG. 2 STUDS OF SPECIMEN GA4

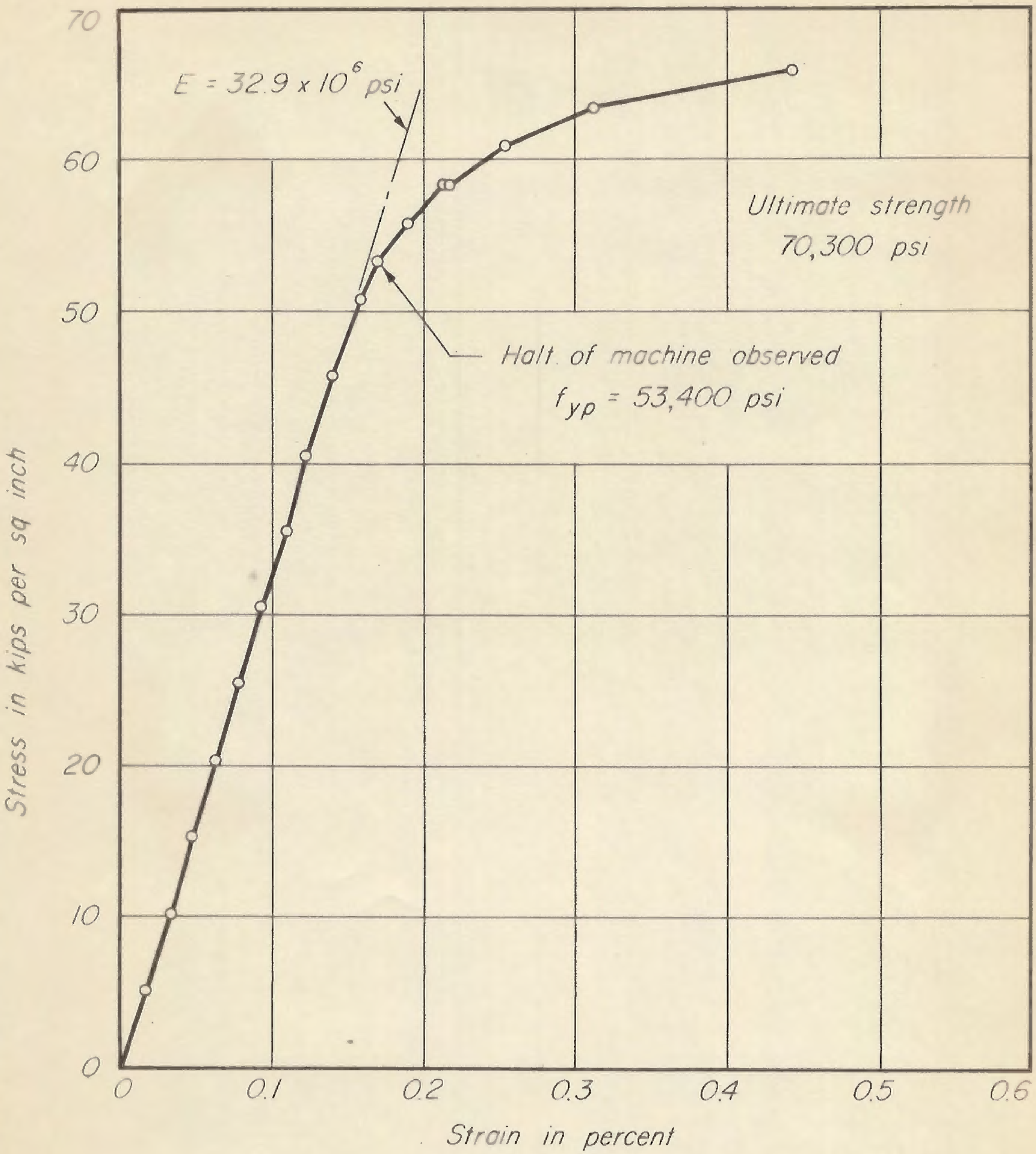

FIG. 3. STRESS-STRAIN DIAGRAM FOR STUD STEEL

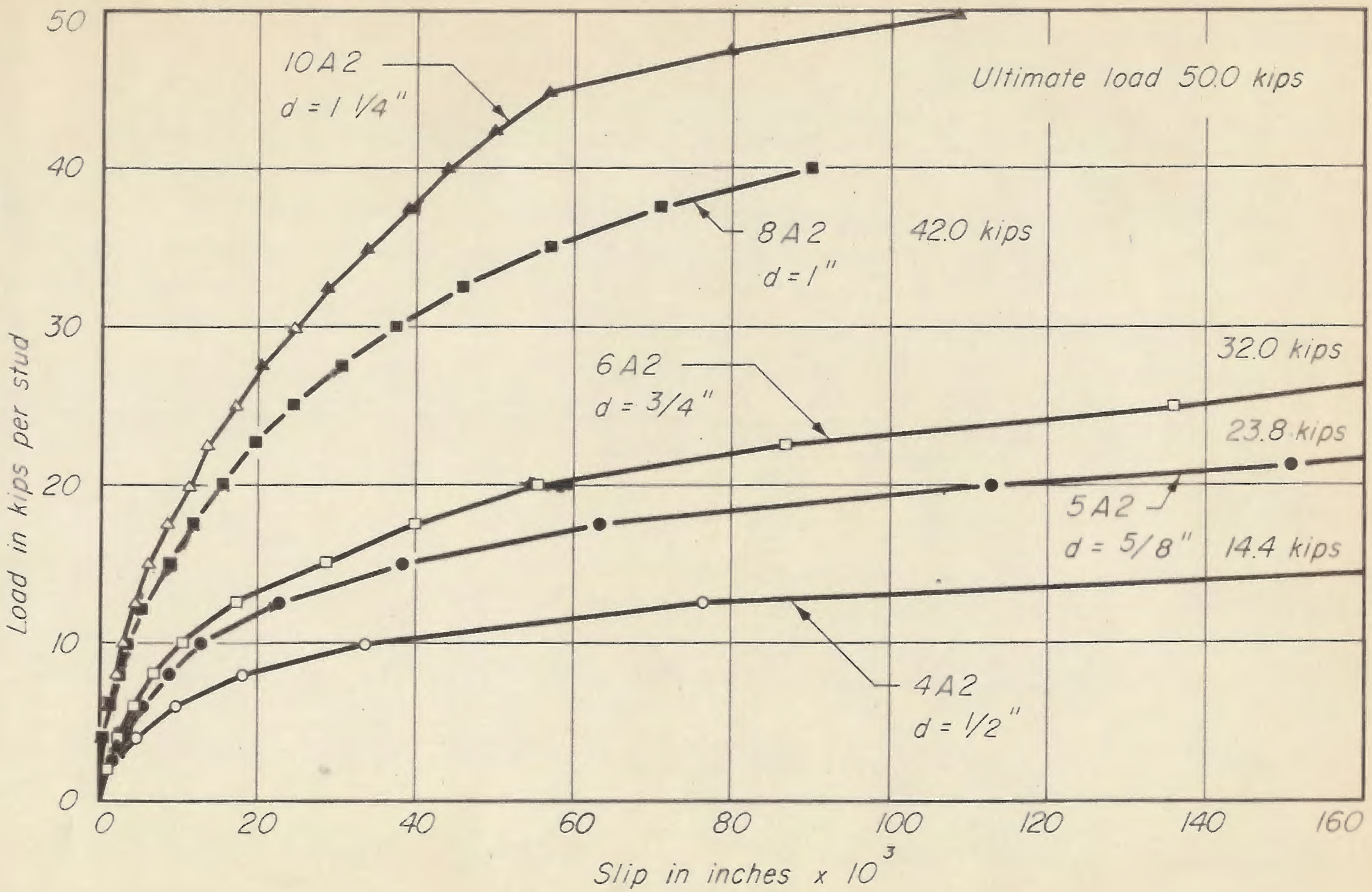

FIG. 4. LOAD-SLIP CURVES FOR TYPE A LOADING

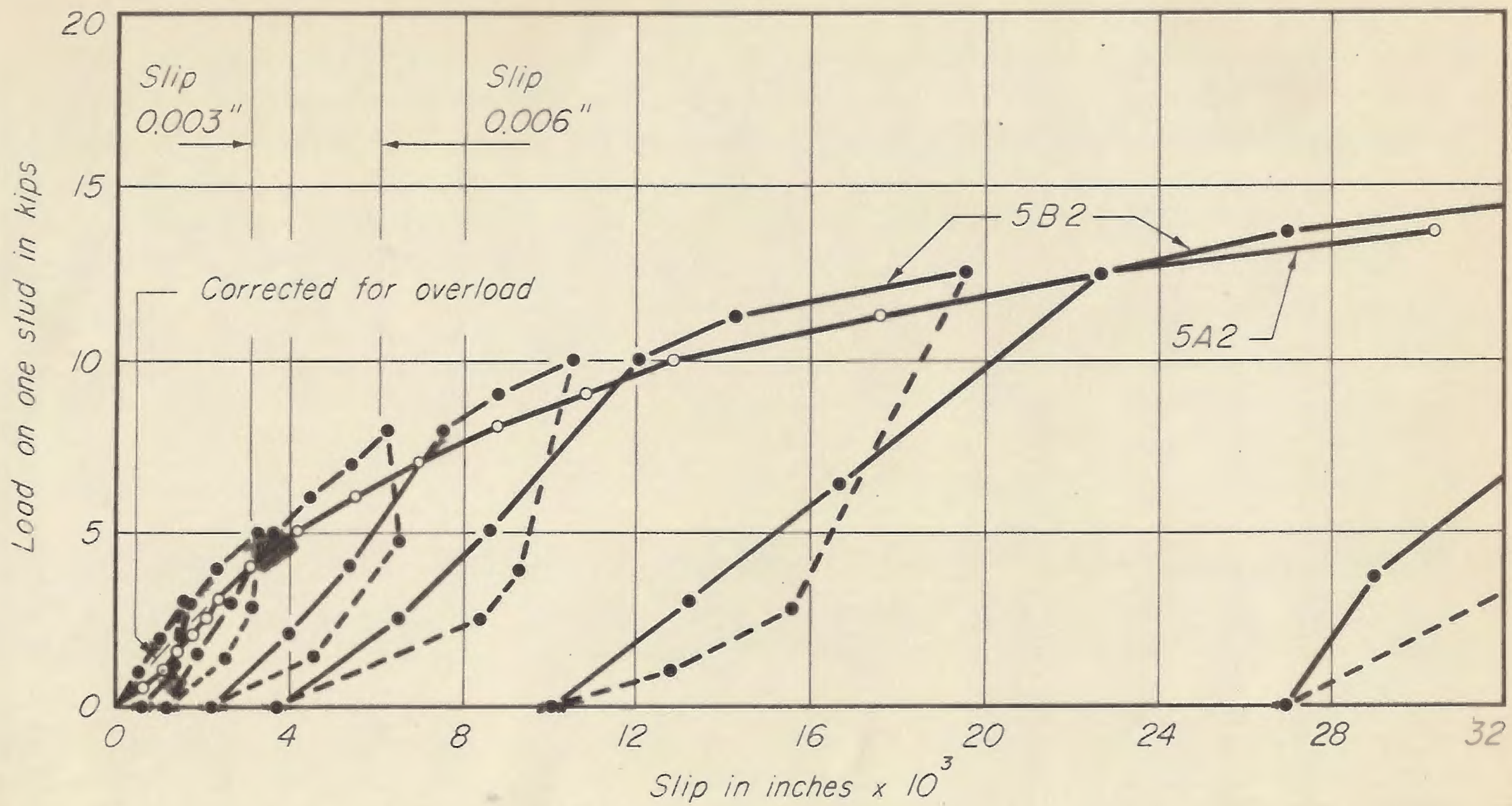

FIG. 5. TYPICAL LOAD-SLIP CURVE FOR TYPE B LOADING

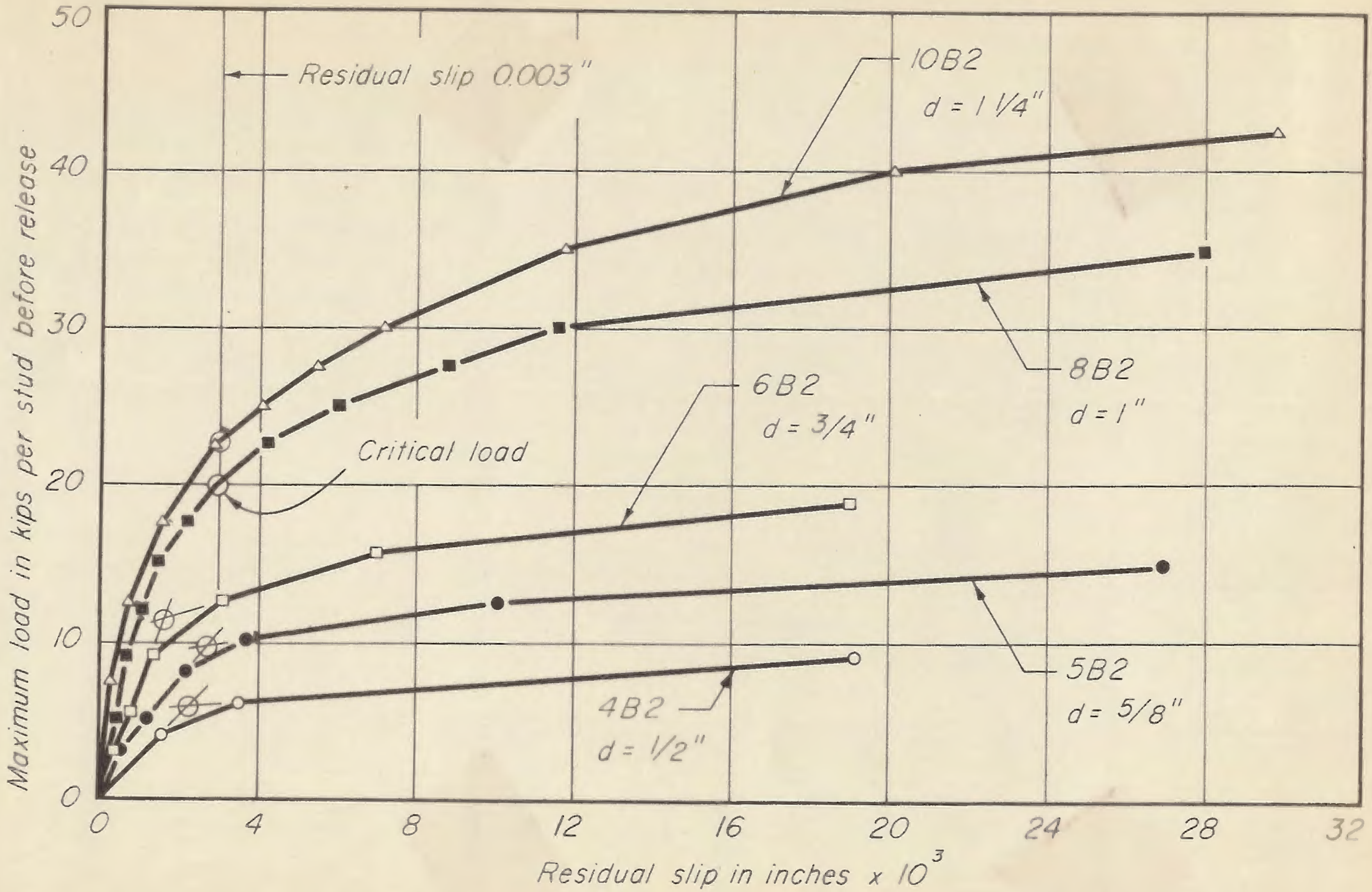

FIG. 6. RESIDUAL SLIPS

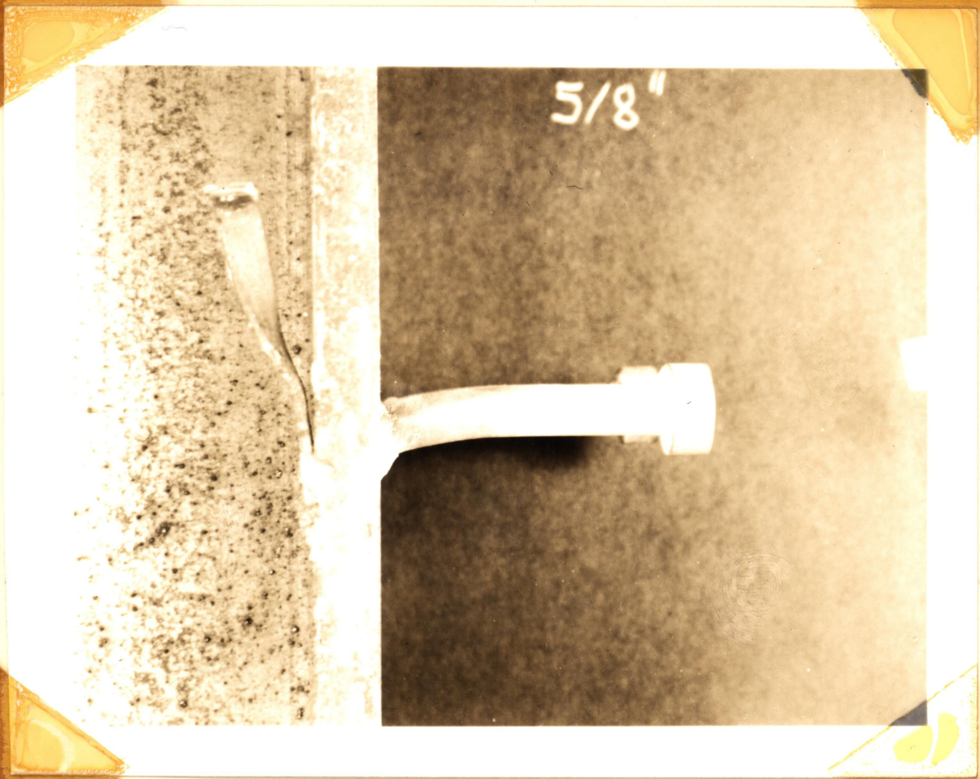

FIG. 7 5/8-IN. STUDS AFTER FAILURE

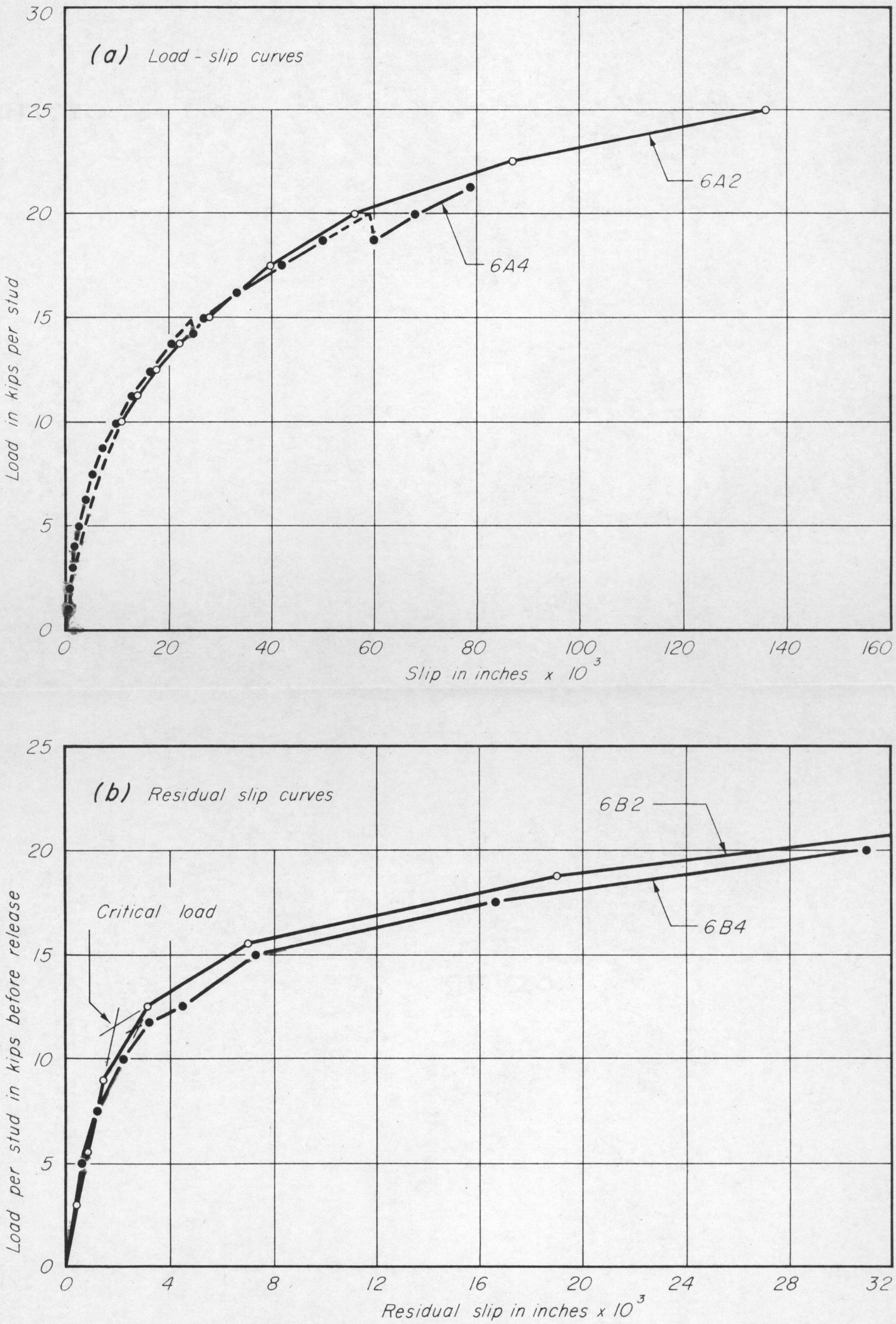

FIG. 8. COMPARISON OF SPECIMENS WITH DIFFERENT NUMBER OF 3/4-IN. STUDS

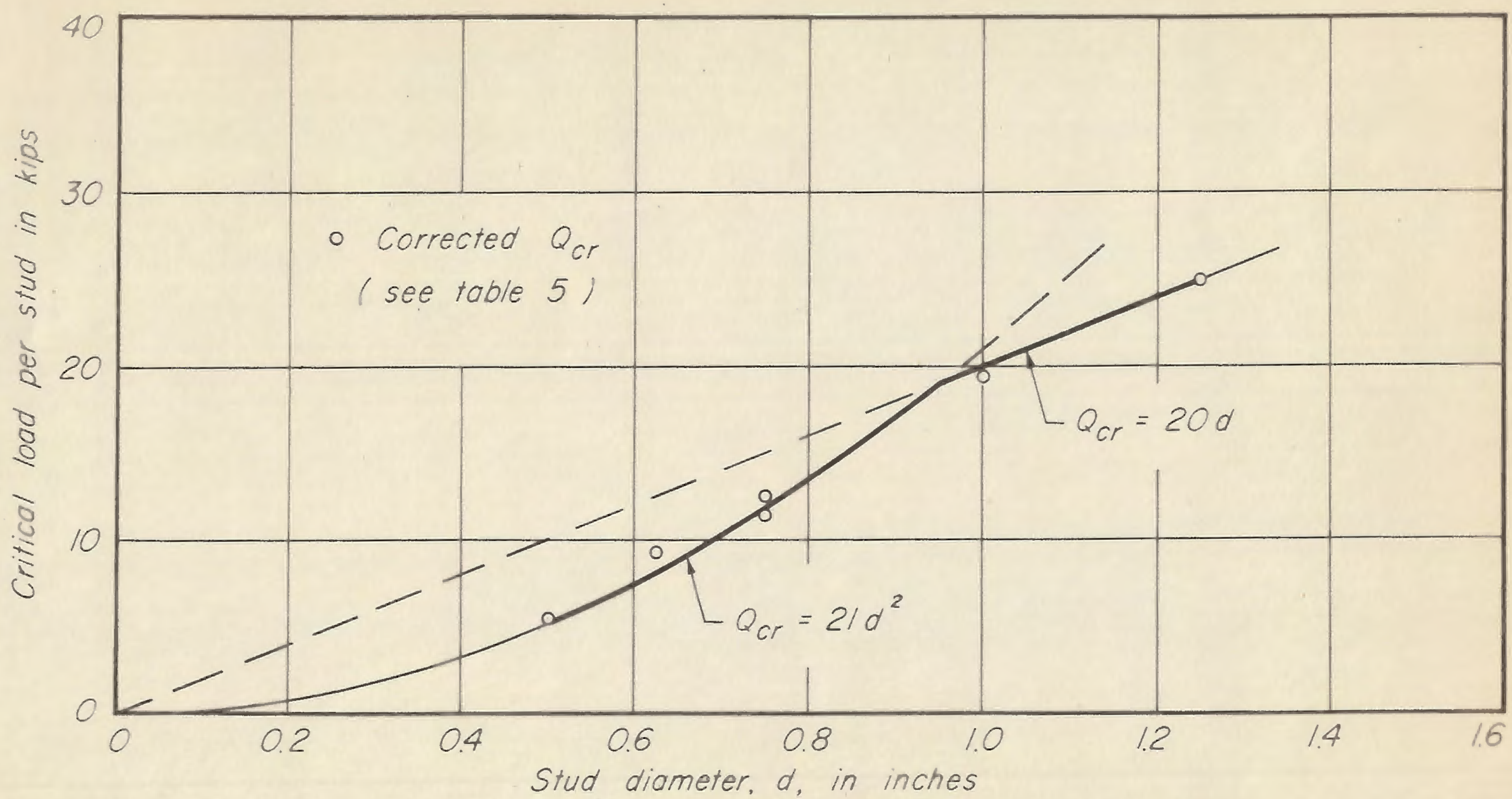

FIG. 9. EFFECT OF STUD DIAMETER ON CRITICAL LOAD

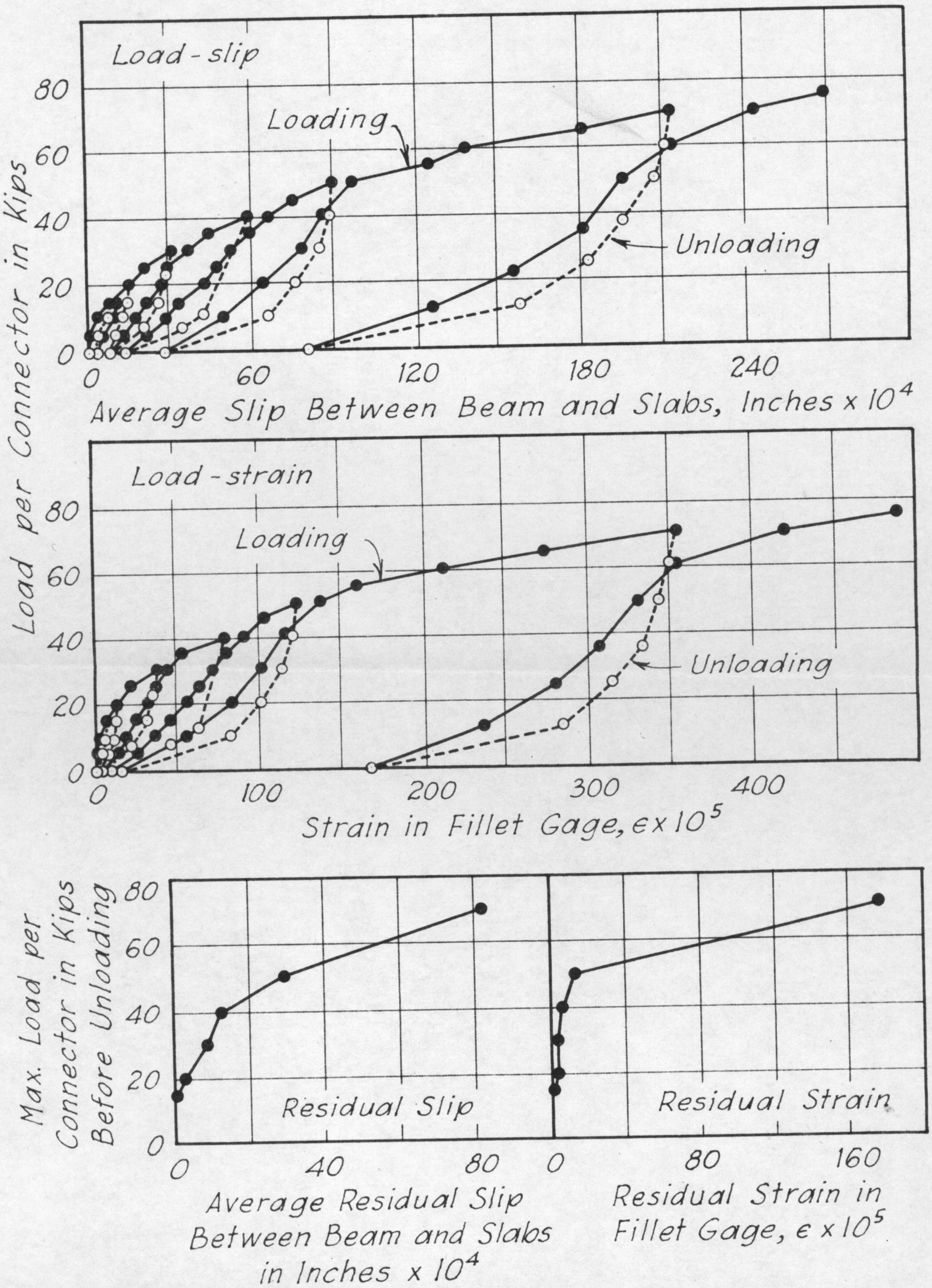

FIG. 10. TEST OF A CHANNEL SHEAR CONNECTOR