Fatigue Strength of ¾-Inch Stud Shear Connectors

A. A. TOPRAC, Professor, Department of Civil Engineering,
University of Texas

Results of fatigue tests performed on seven steel-concrete composite beams are presented. The shear between the steel 24 WF 68 beams and the 6-in. thick slab is developed by means of ¾-in. diameter headed steel studs. The beams were all identical except for the number of studs. The beams were 36.0 ft long and were divided into two groups: (a) four commercially good and acceptable specimens, and (b) three inferior specimens not acceptable to the Texas Highway Department.

Test results indicate that: (a) there is a difference (as much as 3 ksi) in fatigue strength between ½- and ¾-in. studs; (b) the American Association of State Highway Officials (AASHO) specifications allowable stress for stud shear connectors prudently could be increased by reducing the factors of safety presently in use; (c) two of the three defective specimens tested exhibited fatigue strengths equal to those of commercially acceptable specimens; and (d) for 10 million cycles the fatigue strength of the studs tested, expressed in terms of stress range, is at least 13 ksi.

•COMPOSITE construction consisting of a concrete slab attached to a steel beam with mechanical shear connectors has become quite common in buildings and bridges. The shear connectors, welded to the steel section and embedded in the concrete, force the slab and the steel beam to act as an integral unit in resisting loads on the structure. When attached to the compression flange, the slab is very effective as a cover plate for the steel beam. As a result, the deflection of the structure is significantly reduced and savings in steel are possible (1). Channels, spirals, and welded studs have been used successfully as shear connectors. Due to ease in fabrication and flexibility in design, welded studs are currently the most popular.

Research on composite construction with shear connectors dates from 1933. However, studies of welded studs as shear connectors in composite construction began in 1954. These tests included static and fatigue tests of pushout specimens (direct shear), fatigue and static tests of one double T-beam (flexure), fatigue tests of bare studs, and static tests of plate-reinforced concrete beams (2). More recently a program of fatigue tests on flexural members with welded studs as shear connectors was instituted at Lehigh University (3, 4). A total of 12 beams, four with 10-ft spans and eight with 15-ft spans, were studied. For all of these beams ½-in. diameter welded studs were used as shear connectors.

Results from the Lehigh tests correlate well with other fatigue investigations (5) and it appears that a design criteria for ½-in. diameter studs can now be established which will give a realistic factor of safety against fatigue failure. One question which still remained unanswered was the validity of applying the results of tests on small-scale specimens to full-size composite beams, and the "size effect," if any, for studs larger than ½-in. diameter.

Paper sponsored by Committee on Steel Superstructures.

This paper is a report of fatigue tests performed on seven full-size composite beams. The purpose of the investigation was (a) to observe the overall effects of fatigue loading on composite beams, (b) to determine whether results of tests on small-scale specimens could be extrapolated and applied to full-size beams, (c) to obtain additional information concerning the minimum number of stud shear connectors required for beams under dynamic loading conditions, and (d) to investigate the effect of defective stud welds on the fatigue strength of such beams.

TEST SPECIMENS

In this study, seven composite beams were tested. The specimens were divided into Group 1, consisting of four beams, and Group 2, consisting of three beams. Each specimen consisted of a 24 WF 68 steel beam connected with $^3/_4$-in. diameter welded stud shear connectors to a 6-ft wide by 6-in. thick concrete slab. The test specimens were tested as simple beams with a span of 36 ft. They were loaded with two equal concentrated loads 14 ft from each support. Figure 1 shows the overall dimensions of the test specimens.

Specimen Details

Steel Sections.—The dimensions of the test specimens were identical except for the number and spacing of the welded stud shear connectors. The $^3/_4$-in. diameter shear connectors had a height of 4 in. and were welded to the steel beam by a stud welding process. As shown in Figure 2, the studs were placed in pairs throughout the 14-ft shear span on each end of the beam. In addition one pair of studs was placed at the center of the beam. Specimens A, B, and C in each group had 90, 66 and 54 studs, respectively, and specimen 1-D of Group 1 had 78 studs. Figure 2 shows the details and spacing of the studs.

Concrete Reinforcement.—Intermediate grade, deformed steel bars were used for the concrete reinforcement. The transverse bars were $^1/_2$ in. in diameter, placed at 6-in. intervals throughout the length of the beam. The longitudinal bars were $^3/_8$ in. in

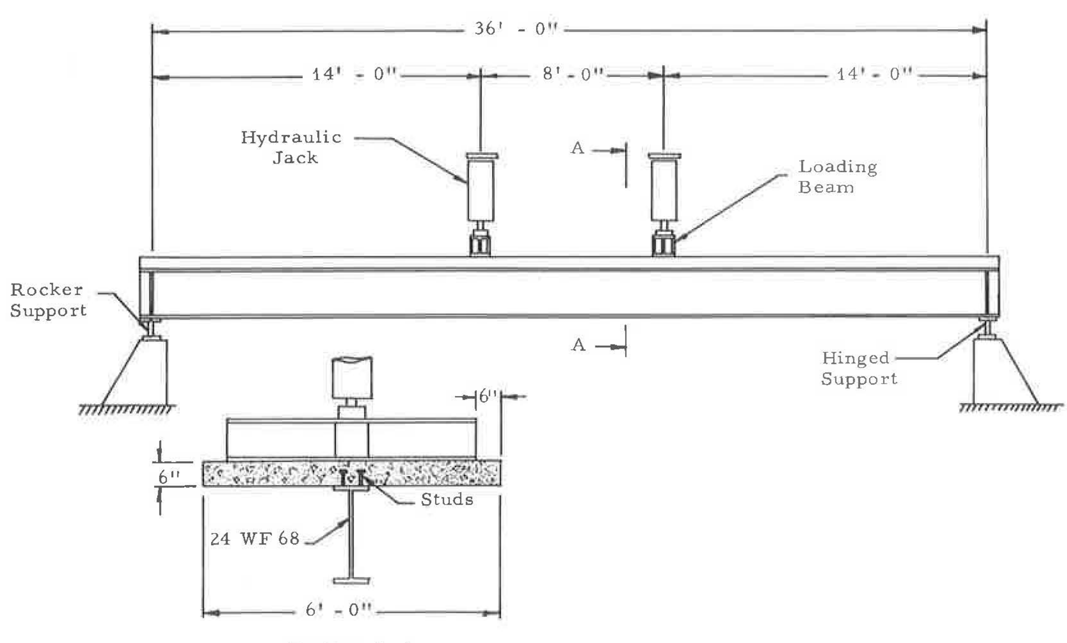


Figure 1. Loading arrangement.

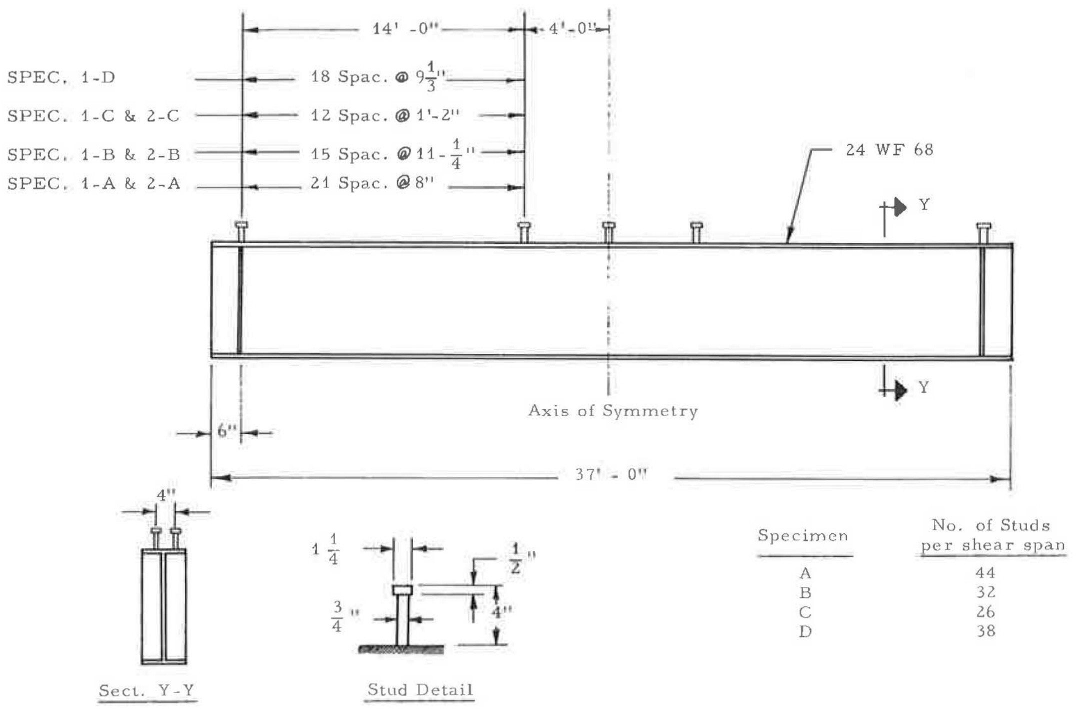


Figure 2. Details of steel beams.

diameter, placed at 12-in. intervals. There were two layers of identical steel in each of the slabs with 1-in. concrete cover on both top and bottom. Figure 3 shows the reinforcement in a typical section and also the details of the reinforcement at the end of the beam.

Concrete. —The concrete for these specimens was ready-mix concrete and was supplied by a local contractor. High early-strength cement was used, and the maximum aggregate size was 1 ½ in.

Since the composite beams were designed to simulate unshored construction, the entire weight of the forms and the concrete were supported by the steel beam. The steel beam itself was supported by concrete blocks only at the ends. The forms for the concrete slab were made of exterior-grade plywood and built in sections so that they were readily reusable. The weight of the concrete was transferred from the wooden forms to the bottom flange of the steel beam by a flange hanger arrangement.

To insure that no bond would exist between the slab and the steel beam, the top flange of the steel beam was given a light coat of oil which, before the concreting operation, was wiped off so that only a very thin film remained.

The casting operation always began at one end of the beam and proceeded toward the other end in a continuous manner. After the concrete was troweled to a smooth finish, the exposed surface was covered with a polyethylene sheet for curing. This covering and the forms were left in place from 4 to 6 days to allow the slab to cure under moist conditions. At the end of this period the forms were removed and the specimen was allowed to cure for a minimum of about 10 days under dry conditions before testing was begun.

End Supports. —To reduce vibrations caused by small, practically unavoidable eccentricities in the loading setup and the test specimens, braces were used at both ends of the beams. These braces consisted of four pairs of angles bolted to the supports and to the steel beam and the slab. Figure 4 shows the details of these braces. The braces were not only effective in reducing vibrations but also important as lateral supports for the specimens which, because of their top-heavy nature, were unstable.

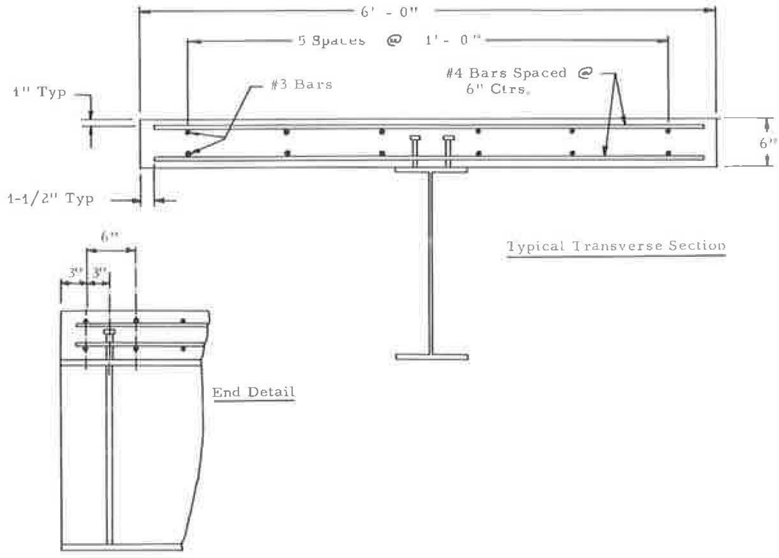


Figure 3. Concrete reinforcing steel.

TABLE 1
TENSILE TESTS OF BEAM STEEL

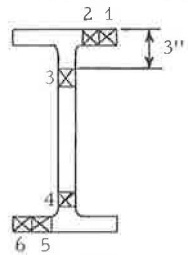
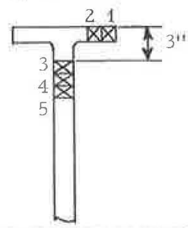
Beam Group	Coupon No.	Coupon Location	Static Yield Point (psi)	Diagram
1	3	Web	56,000	
	4	Web	55,000	
	Avg.	—	55,500	
	1	Flange	35,000	
	2	Flange	35,800	
	5	Flange	39,300	
	6	Flange	35,300	
	Avg.	—	36,350	
2	4	Web	35,800	
	3	Web	35,000	
	5	Web	36,200	
	Avg.	—	35,700	
	2	Flange	34,200	
	1	Flange	36,400	
	Avg.	—	35,300	

TABLE 2

MILL TEST REPORT FOR BEAM STEEL

Beam Group	Yield Point (psi)	Tensile Strength (psi)	Elongation In.	Elongation %	Chemical Analysis C	Mn	P	S
1	41,800	72,500	8	27	0.24	0.78	0.016	0.025
2	37,600	66,600	8	27	0.25	0.73	0.013	0.019

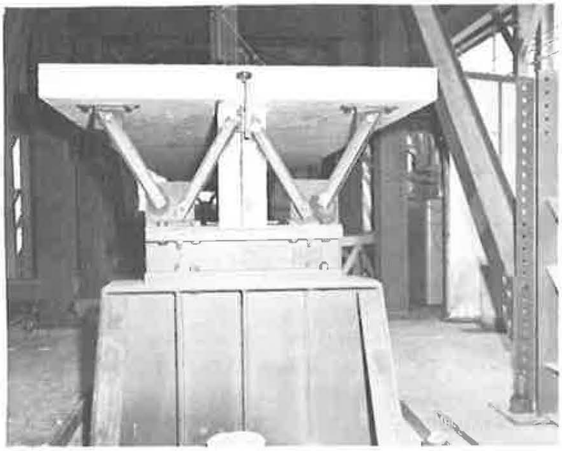
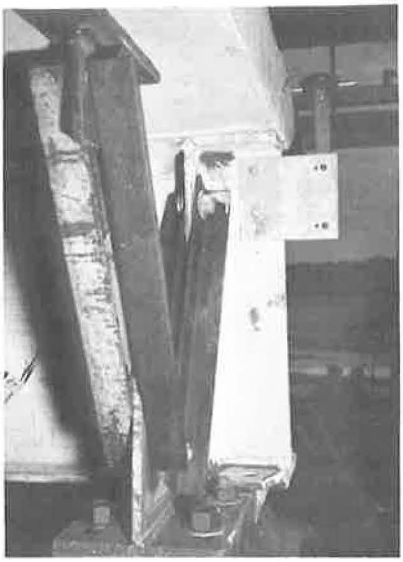
 

Figure 4. End braces and end slip measuring device.

Lifting Apparatus. —Moving test specimens to and from the testing area was accomplished by using a 25-ton overhead crane. To avoid tensile stresses in the concrete while moving the specimens, a 40-ft lifting beam was used. Near each end of the test specimens, pipe sleeves were cast into the concrete slab. This allowed $\frac{5}{8}$-in. diameter cables to be put completely around the bottom flange of the test specimens and then attached to the lifting beam.

Material Properties

To obtain the mechanical properties of the materials used, concrete cylinders and tensile coupons from the steel in the beams and the studs were tested.

Steel Sections. —The seven steel sections were of ASTM A36 steel, each group of the same heat. Tensile coupons were made from a 12-in. stub of the same 24 WF 68 used in the specimens of each group. The coupons were 12 in. long, 1 in. wide, and machined to a constant width of $\frac{1}{2}$ in. in the center portion. An extensometer with a 2-in. gage length was used to measure the strain. After the tensile coupon had reached its yield point, further straining of the specimen was stopped for a period of about 6 to 8 min. The stress at the end of this waiting period was recorded as the static yield strength. This procedure was repeated several times in the plastic range. Table 1 gives the results of all tensile tests and the averages of the web and flange coupons.

Table 2 gives the chemical analysis of the steel as taken from the mill report. The yield point shown in the mill report was 41,800 psi for Group 1 beams and 37,600 psi for beams of Group 2. A faster rate of loading than described previously is the reason for the higher yield point values given in the mill reports.

Concrete. —Usually nine standard test cylinders were made with each beam. Three of the cylinders were made from concrete taken near one end of the beam, three from the opposite end, and three from the center. Approximately one-third of these cylinders were tested at the beginning, one-third at the end, and one-third during the dynamic test. The average concrete strength for the various beams tested varied from 4,150 psi for specimen 2-C to 5,730 psi for specimen 1-A. Table 3 gives a complete summary of the results from the cylinder tests.

Shear Connectors. —The stud shear connectors were made from a low carbon steel. Tensile coupons were machined from two extra studs furnished by the manufacturer

TABLE 3

RESULTS OF CONCRETE CYLINDER TESTS

Beam	No. Cylinders Tested	Cylinder Stress (psi)			Age (days)
		Avg.	Min.[a]	Max.[a]	
1-A	9	5,730	4,630	6,650	41
1-B	5	5,425	4,780	5,940	34-39
1-C	9	5,710	5,500	6,050	32-39
1-D	6	4,570	4,210	4,880	16-53
2-A	9	4,440	4,030	4,670	26-50
2-B	10	4,724	4,620	4,920	16-34
2-C	9	4,151	3,650	4,580	6-20

[a]Of particular group tested.

and tensile tests were made. Yield points of 54,000 and 57,100 psi were recorded with ultimate tensile strengths of 64,000 and 67,600 psi, respectively. The elongation in the 2-in. gage length was 22 percent.

Stud Welding Inspection

All beam specimens for this project were inspected by Texas Highway Department inspectors. The welds in beams of Group 1 were found satisfactory and acceptable for highway construction. These specimens were regarded as beams of commercial quality.

The beams of Group 2 were also checked by the Texas Highway Department inspectors to ascertain the degree of deficiency of the faulty stud welds. Visual inspection indicated that all beams had corrective welds, deficient fillets, undercuts, etc. Of the three beams in this group, only specimen A was inspected thoroughly. The results of this inspection, reported in a letter from the Texas Highway Department (6), were as follows:

1. The criteria for inspection were as described in Texas Highway Department Construction Bulletin C-5 (7).
2. Visual inspection of stud welding was made. It was estimated that approximately 30 percent of the studs did not have a full fillet around the base of the stud, indicating that these studs did not have 100 percent weld. Some of the studs with deficient fillets had corrective manual fillet welds.
3. Of the studs with manual repair and insufficient fillet welds, ten were selected for bending to approximately 30 deg off vertical. Two studs failed (broke off). The failure was in the stud side heat-affected zone, which appeared crystallized. The stud appeared brittle at the point of fracture.
4. The inspector expressed the opinion that additional testing would merely produce additional failures and further testing was discontinued.
5. The studs welded on the girder inspected did not comply with Texas Highway Department Construction Bulletin C-5.

The deficiencies in beams 2-A, 2-B, and 2-C were enough to make them unacceptable for bridge construction. Since it was thought that useful data might be obtained, the specimens were tested after the studs that were broken off or bent as a result of the inspection were replaced.

Specimen Design Philosophy and Objectives

Fatigue tests by other investigators (3, 4) have indicated that fewer shear connectors than presently required by the American Association of State Highway Officials (AASHO) specifications (8) may be satisfactory for bridges. The primary objective of this study was to determine whether or not the results obtained from small-scale fatigue tests can be extrapolated to full-size beams, and to obtain data with substandard beam specimens. All seven specimens were identical except for the number of shear connectors.

Specimen A.—The number of shear connectors for beams 1-A and 2-A was determined from the AASHO specifications (8), assuming a maximum permissible steel stress of 20,000 psi in the tension flange of the beam. The "factor of safety" as defined in the AASHO specifications was 3.70. Forty-four shear connections were required in each shear span. Thus, a total of 90 studs were required. (It should be noted, however, that the maximum test stress in the steel beams was in excess of yielding in beam 1-A and 31,000 psi in all other beams, so that the maximum load in connectors was at least 50 percent over that allowed by AASHO specifications.)

Specimen B. —These beams (1-B and 2-B) were designed according to Section 1.11.4 of the American Institute of Steel Construction (AISC) Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings (9). This design called for 32 studs in each shear span or a total of 66 studs for the beam. According to conventional design procedures, this beam had the minimum number of shear connectors for static loading conditions. AISC specifications for shear connectors are based on a factor of safety of 2.50 against their demonstrated ultimate strength (10).

Specimen C. --These beams (1-C and 2-C) were designed to have the theoretical minimum number of shear connectors required for development of the full flexural static strength of the beam (11). This required 26 studs per shear span or a total of 54 studs for the beam.

Specimen D. —This specimen had 38 shear connectors in each of the shear spans. Thus, the number of studs for this beam was between specimens B and A.

INSTRUMENTATION AND TEST PROCEDURE

The instrumentation for these tests included measurements of vertical deflection of the beam, slip between the concrete slab and the steel beam, flexural stresses in the steel beam, and localized stresses in the upper flange of the steel beam caused by the presence of the studs. The steel beam was also whitewashed with a lime-water solution so that yield lines could be easily observed. Instrumentation was essentially the same for all specimens. Exceptions to this are noted in the following discussion.

Vertical Deflection Gages

Vertical deflection was measured on the bottom flange of the beam at the center and 4 ft on either side of the center with dial indicators. On beams 1-D, 2-B and 2-C, only the differential deflection (the deflection observed as the load was increased from zero to the maximum) was recorded. No attempt was made to measure residual deflection as it accumulated during the dynamic test. On all other specimens, however, the residual deflection as well as the differential deflection was measured. As a result it was possible to record total deflection for specimens 1-A, 1-B, 1-C and 2-A.

End Slip Gages

The slip of the concrete relative to the steel beam was measured at each end of the beam using dial indicators. The dial indicators were rigidly attached to the steel beam and held in the same position throughout the test. To avoid damage to the gage, the point was restrained from touching the concrete during dynamic tests. Figure 4 shows the apparatus used to measure end slip.

Electric Strain Gages

Baldwin paper-backed wire strain gages were used throughout these tests to measure flexural strains in the steel beam and to determine an approximate index of the load on the studs.

Flexural Stresses. —Three strain gages were used on each specimen to determine flexural strains at the center of the beam. One gage was placed on the bottom of the

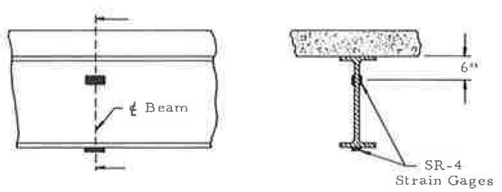


Figure 5. Strain gages for flexural strain measurement.

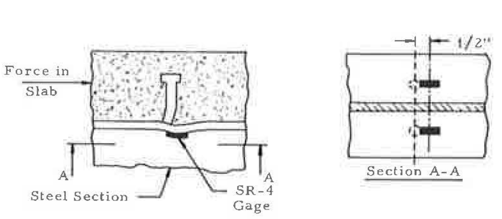


Figure 6. Strain gages for loads on studs.

tension flange. The other two gages were placed 6 in. below the top flange on either side of the web (Fig. 5).

Shear Connector Forces. —An attempt was made to measure, at least qualitatively, the load on individual studs. This was done by placing strain gages on the underside of the top flange of the steel beam in the immediate vicinity of the stud under consideration. The force on the stud created localized deformations in the top flange which were recorded by the strain gages. As shown in Figure 6, the strain gages were not directly under the studs but were located ½ in. from the center of the stud on the side of the connector nearest the end of the beam. In every case tensile strains were recorded from these gages. This procedure, developed in earlier investigations (4), made it possible to determine when an individual stud started to fail. At least three pairs of studs near each end of the beam were instrumented in this way. In beams A, B and C of Group 1, the second pair of studs beyond the load points was also instrumented in the same manner.

Test Procedure

Initially each beam was loaded statically 3 times to the maximum load which was to be applied dynamically. During these cycles of static testing, strain, deflection, and end slip data were taken at load increments from zero to the maximum. This procedure was followed so that any small inelastic deformations caused by the initial loading could be determined before the start of the dynamic tests.

Following this initial static test, the beam was tested dynamically from the minimum to the maximum load at the rate of about 180 cycles/min. Periodically, the dynamic test was interrupted to make static tests in which the strain, deflection, and end slip measurements were again taken at incremental loads. This general procedure was followed throughout the tests with only slight variations in the number of cycles between static tests. No deflection, slip or strain measurements were made while the specimen was under dynamic loading.

The applied dynamic load for all beams, with the exception of 1-A, was identical and ranged from 4 to 33 kips. For beam 1-A, the imposed dynamic load range was 5.2 to 51 kips per hydraulic jack.

TEST RESULTS

The results of the fatigue tests are presented in the following sections. Each beam was tested from a condition of complete composite action to one approaching no composite action. All of the test specimens failed by shear failure in the studs. Usually the studs in one shear span failed completely and the studs on the opposite end showed definite deterioration but were not completely sheared off. As should be expected, the

TABLE 4

SUMMARY OF FATIGUE TEST RESULTS

Specimen	Load[a] (kips)		Avg. Stud Stress (ksi)		Stress Range (ksi)	No. Cycles to Failure		
	P_{max}	P_{min}	Max.	Min.		First Stud	First Pair of Studs	Beam
1-A	51	5.2	18.4	1.9	16.5	70,000	85,000	105,200
1-B	33	4	16.5	2.0	14.5	1,620,000	4,330,000	4,490,000
1-C	33	4	20.3	2.5	17.8	205,000	230,000	260,500
1-D	33	4	13.9	1.7	12.2	1,400,000	2,380,000	2,870,000
2-A	33	4	11.9	1.4	10.5	1,500,000	1,800,000	2,282,000
2-B	33	4	16.5	2.0	14.5	600,000	900,000	1,333,000
2-C	33	4	20.3	2.5	17.8	90,000	103,000	120,000

[a]Force (two on each specimen) applied by each hydraulic jack.

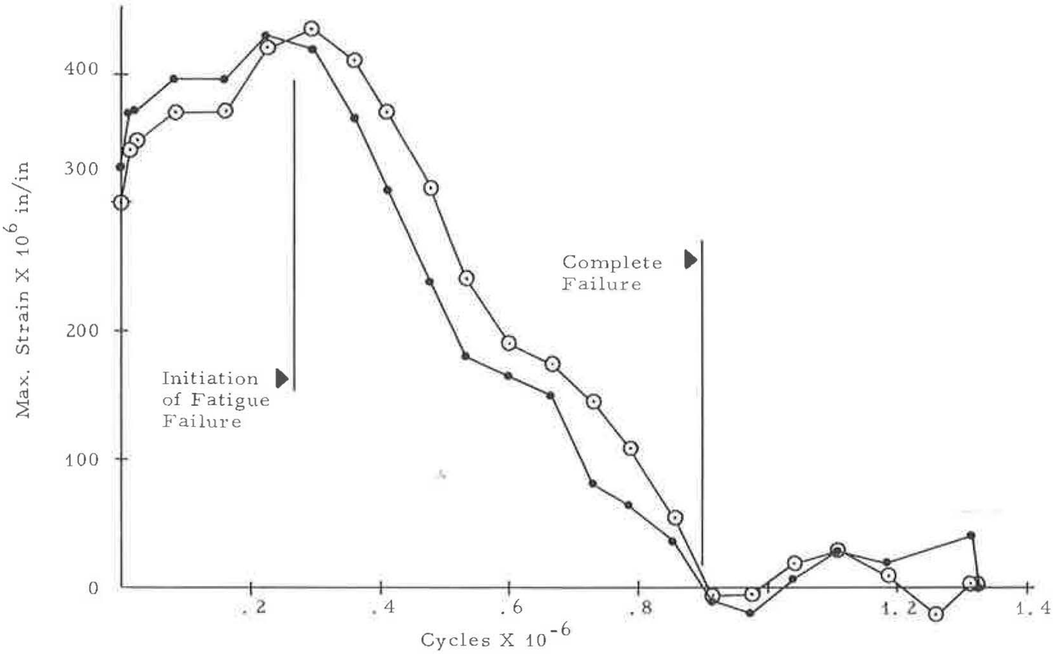


Figure 7. Local strain readings vs cycles for one pair of studs.

end slip in the shear span that failed was much greater than that on the opposite end. This is illustrated in the discussion of beam 1-B. The fatigue test results are summarized in Table 4.

Loads on Individual Studs

As described earlier, an attempt was made to measure the effectiveness of individual shear connectors. In the initial static test, at maximum load, tensile strains of 150 to 400 μ in./in. were recorded from these gages. When the beam was unloaded, small residual strains were present. As the dynamic test progressed, the strains from these gages increased. Then, as the studs cracked and became less effective as shear connectors, the strains gradually went to zero and eventually went into compression when the stud was completely sheared off. Figure 7 shows strain readings from a pair of studs in specimen 2-B. The readings plotted in Figure 7 are qualitatively typical of all the studs instrumented.

It should be noted that the strains recorded from these gages serve only as an index to the magnitude of the load transferred from the slab to the steel section by a particular stud. The information derived from these measurements is relative and shows only the effectiveness of a stud as the number of cycles increases. These local strains cannot be compared from stud to stud.

In all beam tests, stud failure was observed by the foregoing technique before measurements of deflection and end slip indicated any significant deterioration of composite action.

Steel Beam Stresses

The maximum stress under dynamic loading conditions in the tension flange of the steel beam varied from about 9,900 to 31,000 psi for all specimens except 1-A, whose bottom flange yielded extensively during the initial loading. These values were effectively constant throughout the fatigue tests. The only measurable increase in tension flange stress occurred near the end of the dynamic test when each beam rapidly lost composite action.

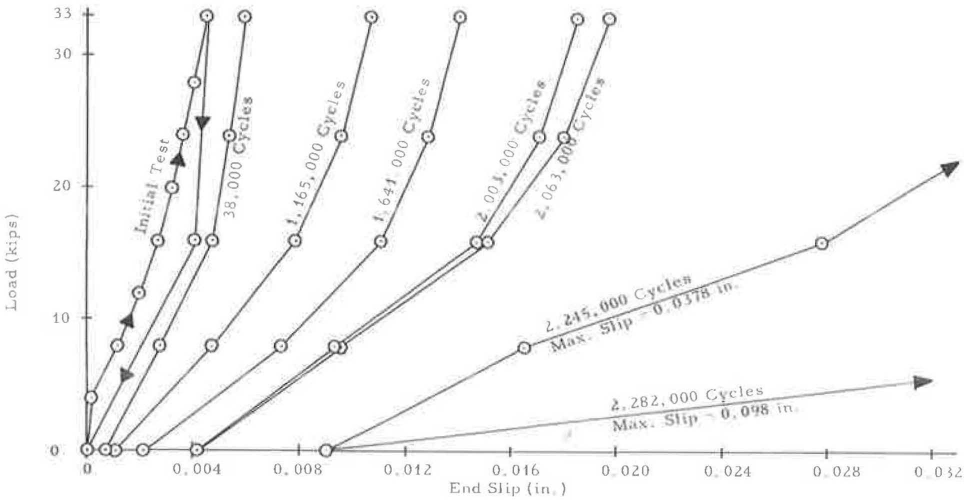


Figure 8. End slip vs load at hinged support on specimen 2-A.

Effects of Bond

As discussed previously, an attempt was made to eliminate bond between the concrete and the top flange of the steel beam by applying a light film of oil to the beam before the concrete was cast. An evaluation of the success of this effort is necessary to determine accurately the loads applied on the shear connectors.

The initial static load test produced end slip which gradually increased as load increased. This condition was true for both ends in all beams. It seems unlikely that a significant amount of slippage would occur if bond was present. Furthermore, measured midspan deflections in the first static test were always slightly greater than those calculated.

Other data pertinent to the evaluation of the effects of bond come from the strain gages used to determine loads on specific studs. On the initial static test all of the studs instrumented produced tensile strains which progressively increased with the applied load, although the gages were on the compression side of the neutral axis. This indicated a significant load on the shear connectors which, in turn, showed that there was little, if any, bond between the steel beam and the concrete.

In addition, a quantitative appraisal of the strains recorded from the gages placed underneath the studs not only reinforced the hypothesis that bond was not present, but also indicated that the strains were simply proportional to the force on the stud.

This evidence seems to indicate that bond was not a significant factor in these tests. Therefore, in calculations of connector shear stresses, no bond was assumed between the slab and the steel beam.

Group 2—Beams with Defective Stud Welds

Specimen A (44 Studs per Shear Span).—Under dynamic loading the average stud shear stress for beam 2-A, as computed from elastic theory and assuming complete composite action, fluctuated from 1,400 to 11,900 psi (range, 10,500 psi). The first stud failed at 1,500,000 cycles and the first pair failed at 1,800,000 cycles. (Failure was measured by local strains produced by the force on each stud. When this strain becomes zero the stud is considered to have cracked throughout.) Two additional pairs had failed before 1,900,000 cycles were recorded. As the fatigue test was continued, other studs failed. At 2,282,000 cycles, dynamic testing was discontinued because the beam exhibited very little composite action and tension cracks in the concrete were noted in the constant moment region. The stud shear failure occurred primarily on

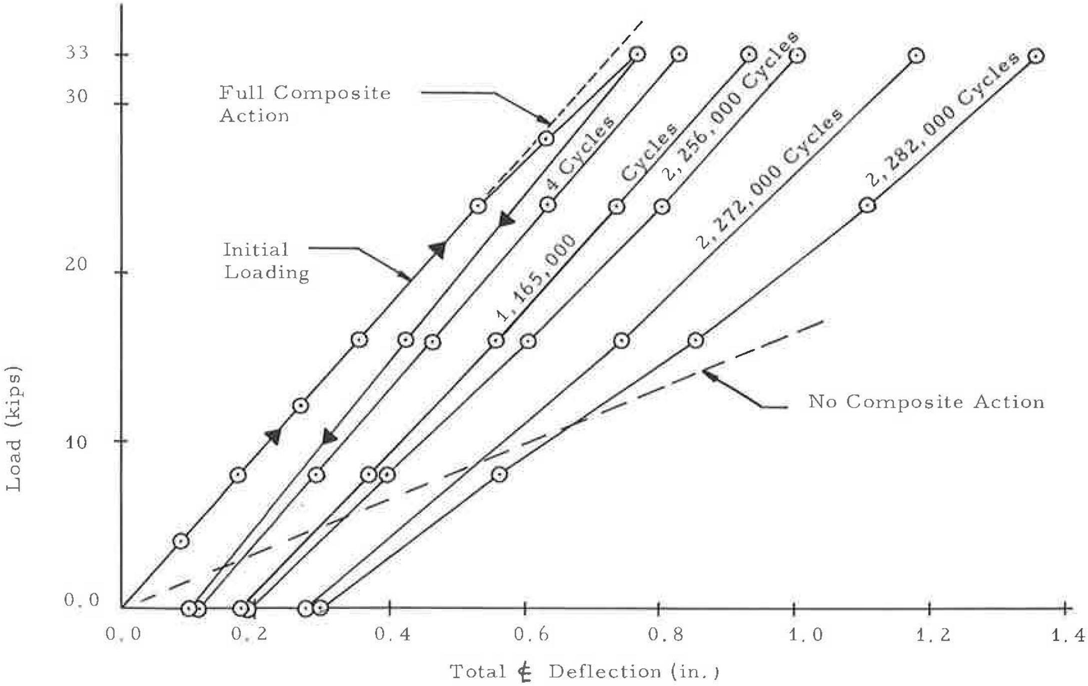


Figure 9. Total centerline deflection vs load for specimen 2-A.

the end with the hinged support. Figures 8 and 9 show the end slip on the hinged end and the midspan deflection of specimen 2-A. (The load in these and subsequent figures denotes the force applied by one of the hydraulic jacks; the total load on each beam is twice as much as shown.)

On specimen 2-A the concrete was carefully broken up and removed from both ends of the beam, allowing visual inspection of the studs. In the shear span which showed the principal failure, all of the studs were completely sheared off. Most of the fractures were in the heat-affected beam metal. Twelve of the studs were fractured in two places, in the beam metal forming a crater and about $^3/_4$ in. above the beam flange. It is of interest to note that eleven of the studs which fractured in two places were located in a longitudinal row on one side of the flange.

None of the studs in the shear span with the smaller end slip were sheared off completely. Most of these studs, however, were visibly cracked on the side nearest the center of the beam and were easily removed from the beam flange by striking them with a light hammer. Several of the studs were detached from the beam in this manner. The studs which were the most difficult to remove had a fatigue crack extending over about 40 percent of the stud area. Other studs which were more easily removed had fatigue fractures over about 90 percent of their area. It was quite evident from these observations that complete stud fracture in this shear span was imminent.

Specimen 2-B (32 Studs per Shear Span).—Stud shear stress on specimen 2-B ranged from 2,000 to 16,500 psi (range, 14,500 psi). The first stud failed at 600,000 cycles and the first pair of studs failed at 900,000 cycles. Additional stud failures were noted as cycling continued and the test was concluded at 1,333,000 cycles. In contrast to specimen 2-A, stud shear failure occurred primarily in the shear span nearest the rocker support.

The concrete was also removed from specimen 2-B. In the shear span that failed, all of the studs were completely sheared off. Most of the fractures were in the heat-affected beam metal. Seven of the studs were fractured in two places as in specimen 2-A. Figure 10 shows the bottom portion of one of these studs in place on the steel

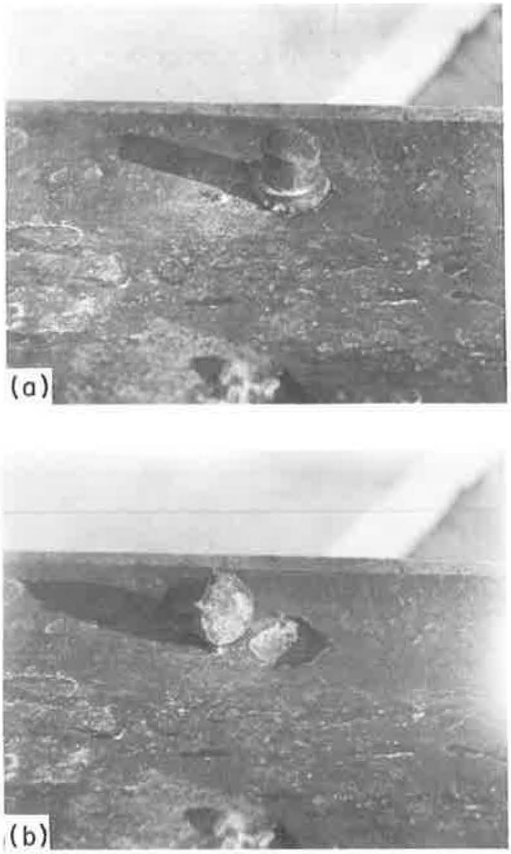


Figure 10. Stud failure on specimen 2-B.

beam flange. Figure 10b shows the same stud and the crater in the beam flange. The crater in the beam flange is typical of stud failures of specimens 2-A and 2-B.

Figure 11 presents load-slip curves after various cycles of dynamic loads were applied. Figure 12 shows the deflection at midspan measured after the indicated cycles of loading.

As in specimen A, tension cracks in the concrete were developed in the constant moment region under the loads and near the centerline. In the shear span showing the smaller slip, the studs were still attached to the beam flange. Serious fatigue cracking was present, however, and several of these studs could be pushed over and separated from the beam flange by hand. This indicates, as in specimen 2-A, that failure occurred in both shear spans at about the same number of cycles.

Specimen 2-C (26 Studs per Shear Span). —Stud shear stresses on specimen C fluctuated from 2,500 to 20,300 psi (range, 17,800 psi). Because this beam failed earlier than anticipated, sufficient data were not taken to determine precisely the first stud failure. Data from other studs on this test seem to indicate that the first stud failed at about 90,000 and the first pair of studs failed at 103,000 cycles. Fatigue testing was continued and at 120,000 cycles, all of the studs in the shear span nearest the hinged end had failed. As in specimens 2-A and 2-B, tension cracks in the concrete developed in the constant moment region.

Concrete was also removed from specimen C. Visual inspection of the stud fracture seemed to indicate defective welds. One of these failures is shown in Figure 13.

Figures 14 and 15 give the slips and deflections for this beam after the indicated cycles of loading were carried by the beam.

Group 1—Beams with Acceptable Stud Welds

Specimens B, C and D. —These otherwise identical beams had 32, 26 and 38 studs per shear span, respectively. They were subjected to the same dynamic loads. Accumulated deflections were measured for 1-B and 1-C and are plotted in Figures 16 and 17. The load-deflection curve for beam 1-D is shown in Figure 18.

Load slips after various cycles are shown for these beams in Figures 19 through 22. It was stated that invariably one shear span indicated more slip than the other. Some specimens showed the larger slips at the rocker end, whereas others slipped more at the hinged end. There was no definite pattern. In Figures 19 and 20 the measured slips for both ends are plotted. It can be noted that at the end of the test while the north end slipped over 2 in., the south slipped less than ½ in. The load-slip curves presented for all the other six specimens show the data obtained only from the end that slipped most.

The beam, which sagged about 2 in. due to this failure, was jacked up and repaired by welding a T-section replacing two-thirds of the lower portion of the beam. Beam

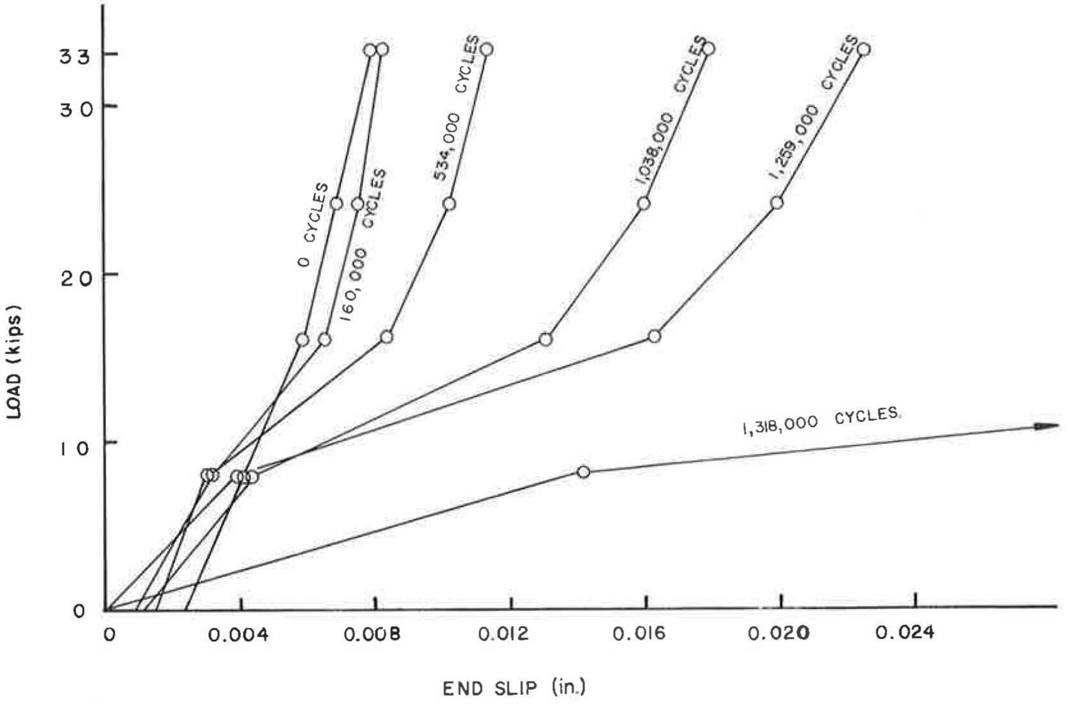


Figure 11. End slip vs load for specimen 2-B.

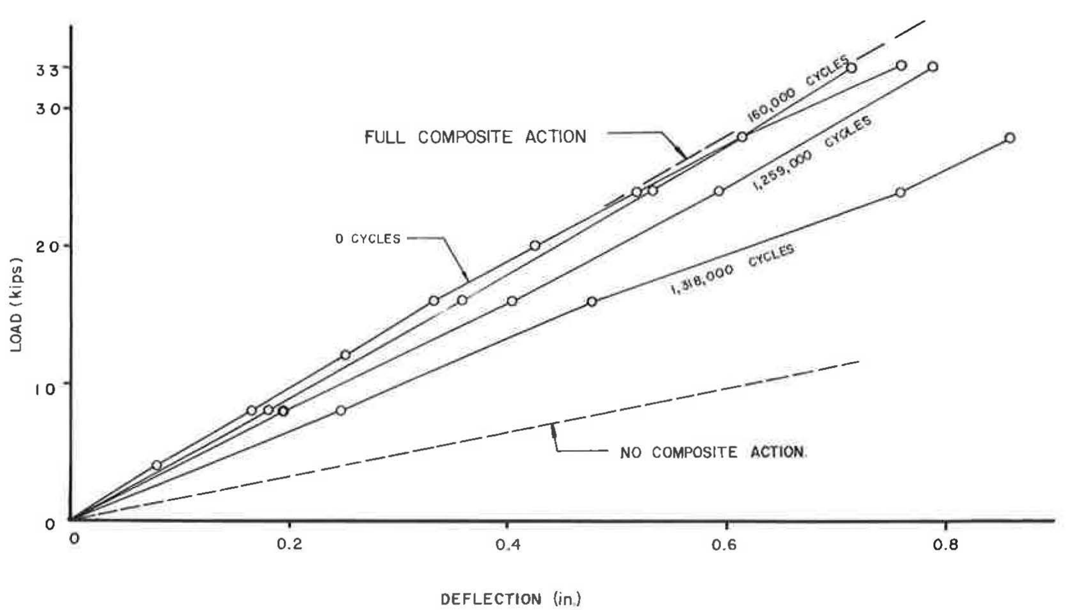


Figure 12. Deflection vs load for specimen 2-B.

Figure 13. Stud failure in specimen 2-C.

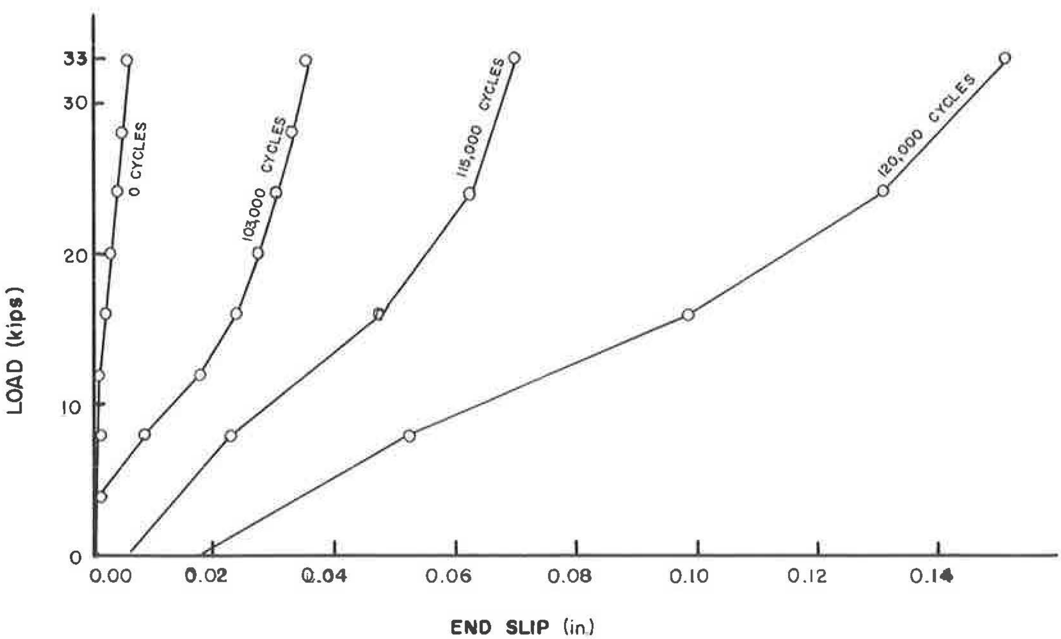


Figure 14. End slip vs load for specimen 2-C.

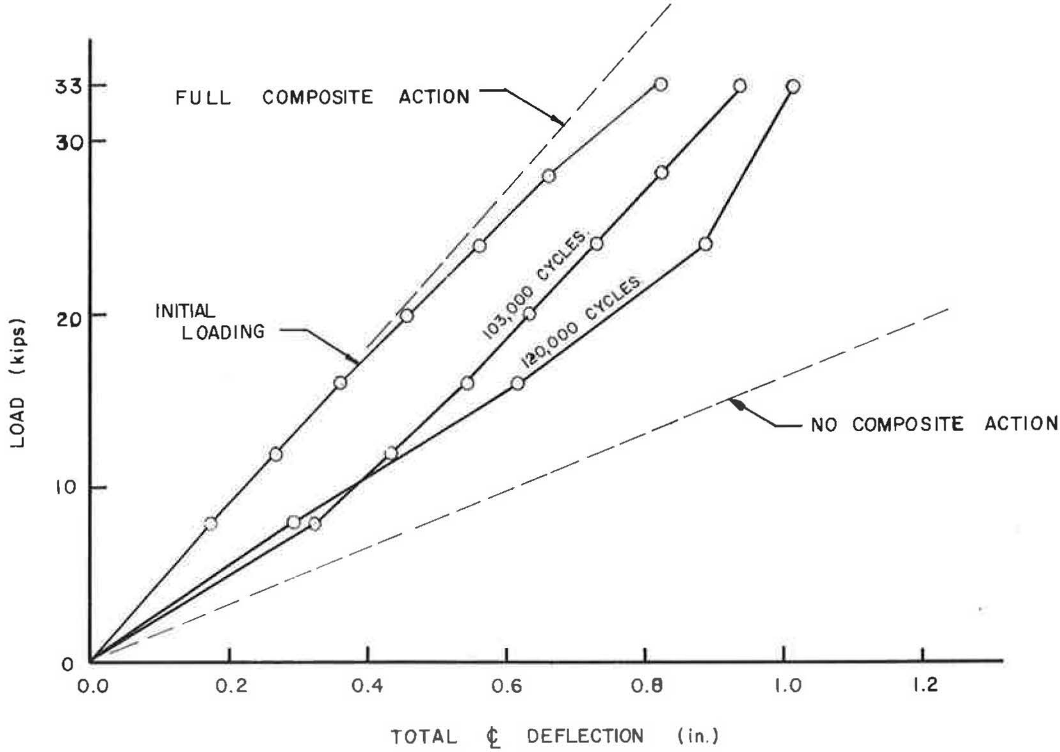


Figure 15. Total deflection vs load for specimen 2-C.

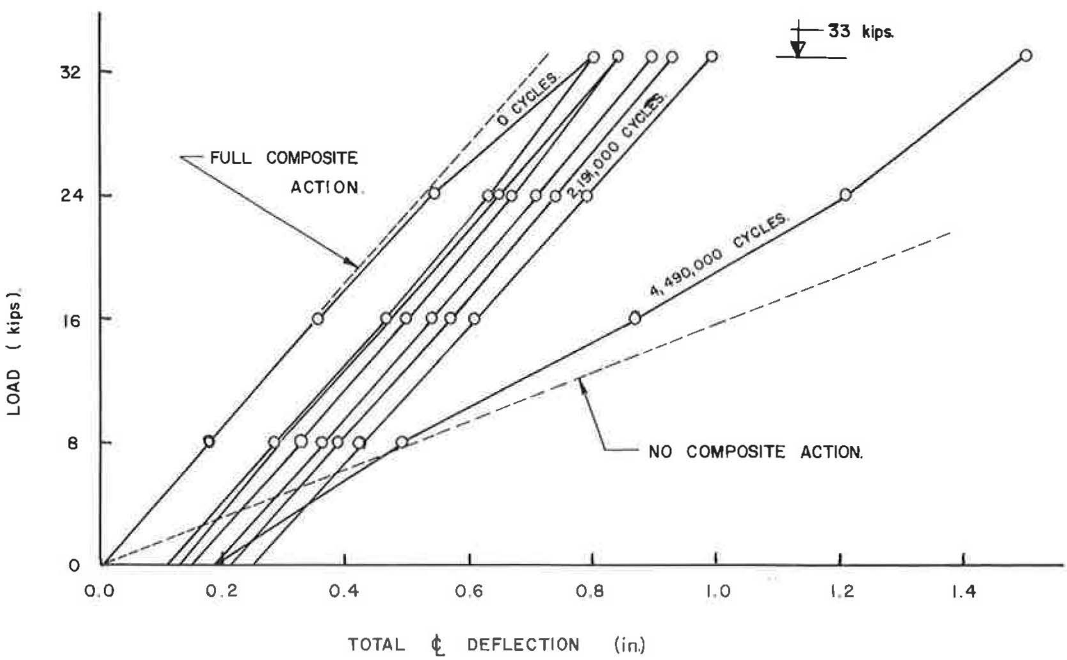


Figure 16. Total centerline deflection vs load for specimen 1-B.

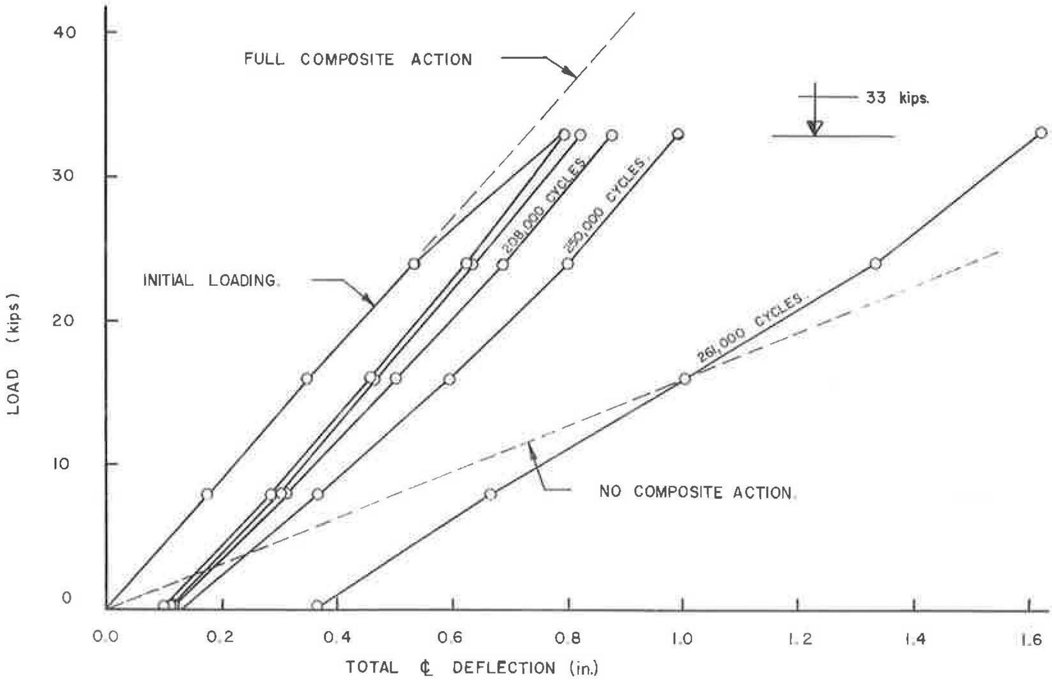


Figure 17. Total centerline deflection vs load for specimen 1-C.

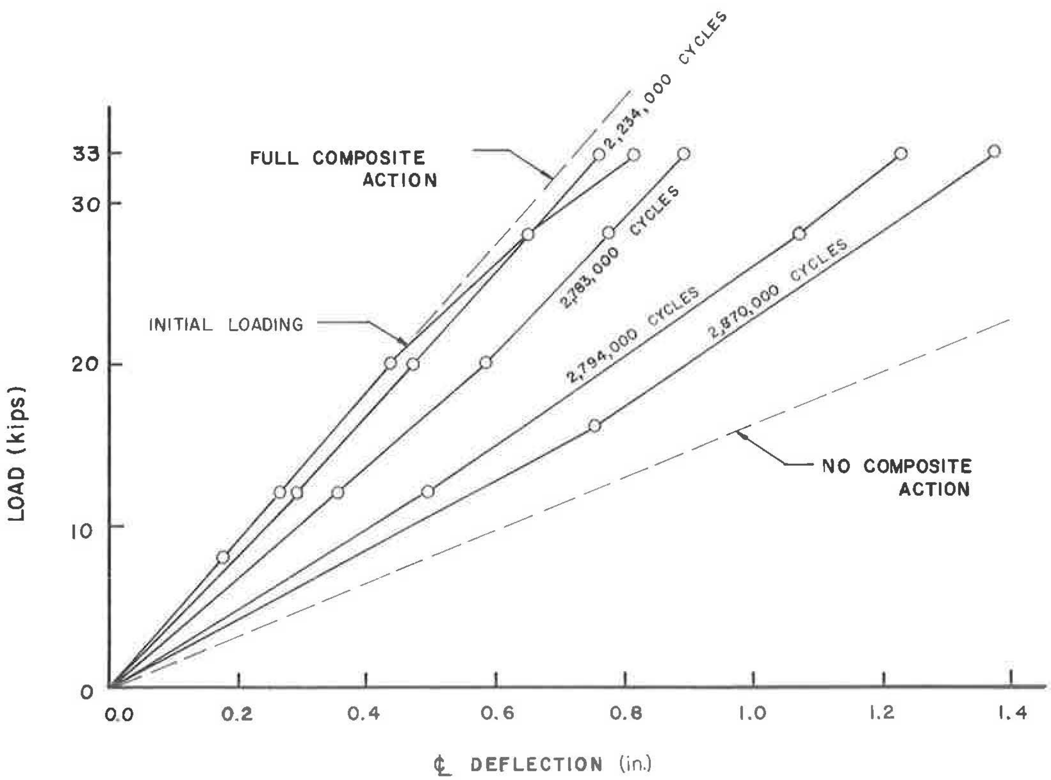


Figure 18. Centerline deflection vs load for specimen 1-D.

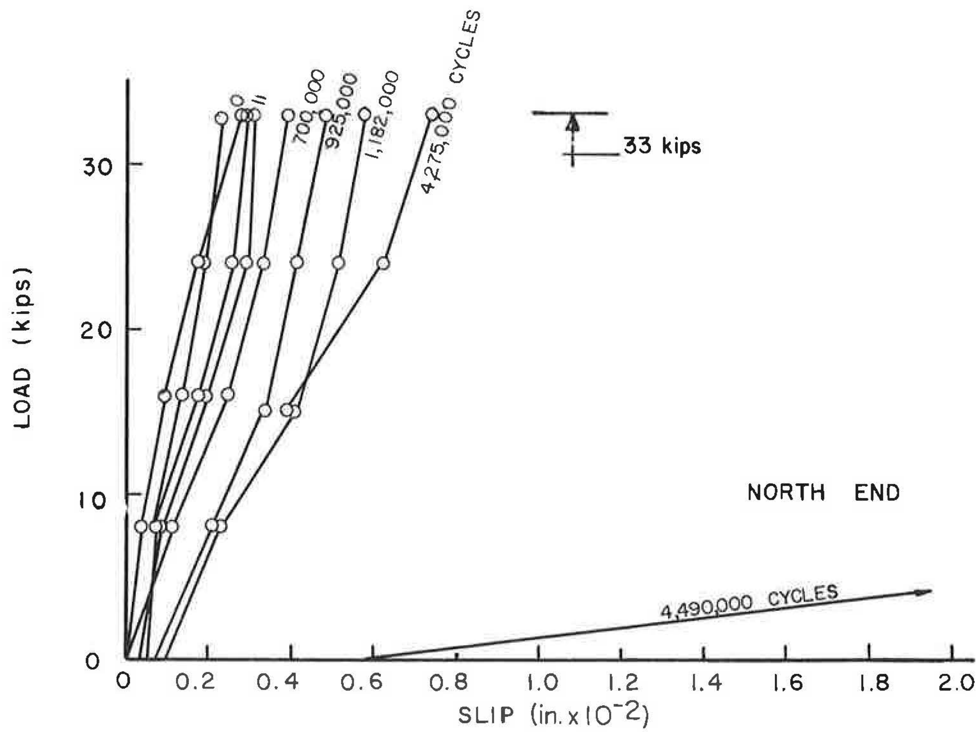


Figure 19. Slip vs load for specimen 1-B.

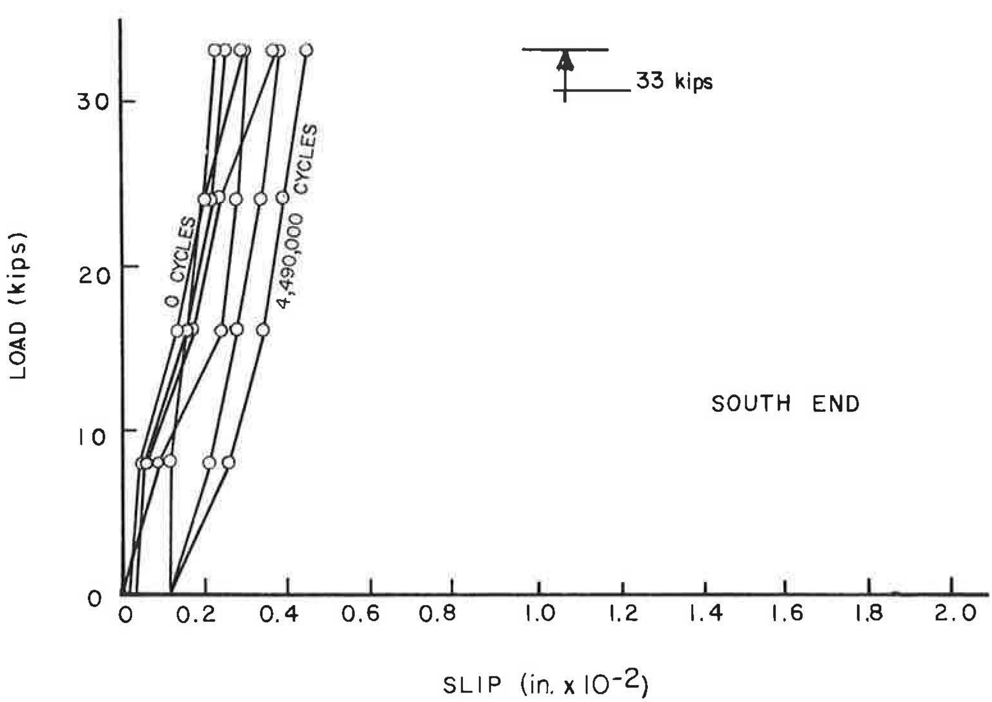


Figure 20. Slip vs load for specimen 1-B.

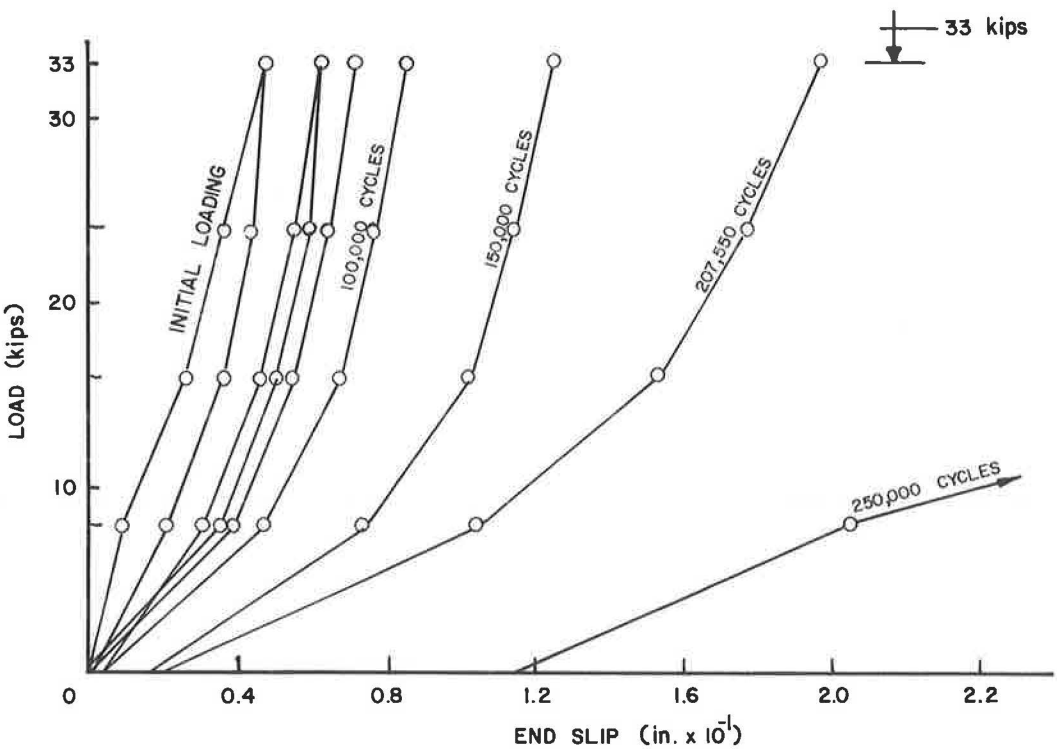


Figure 21. End slip vs load for specimen 1-C.

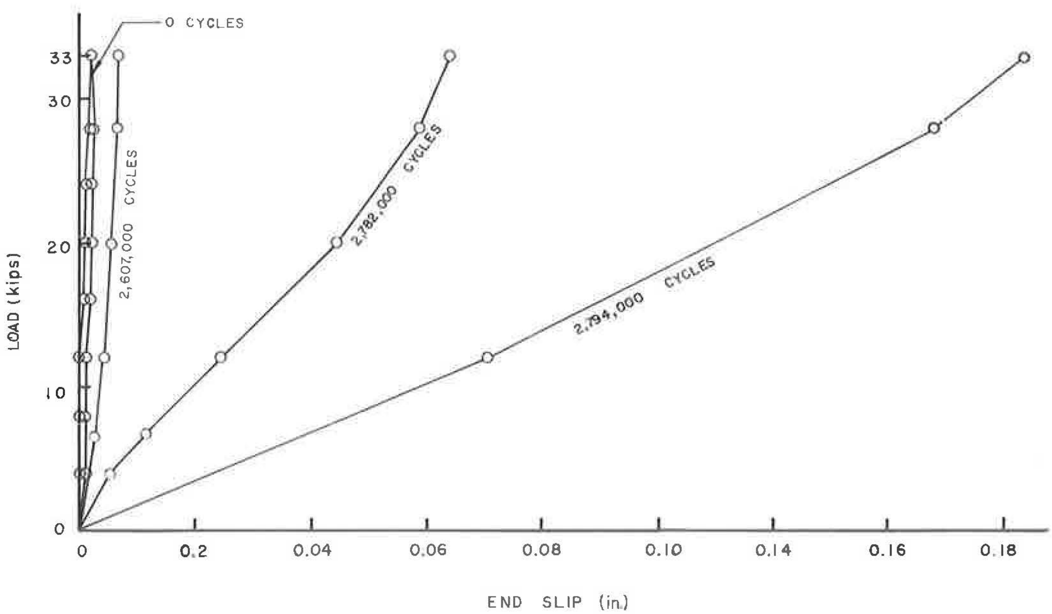


Figure 22. End slip vs load for specimen 1-D.

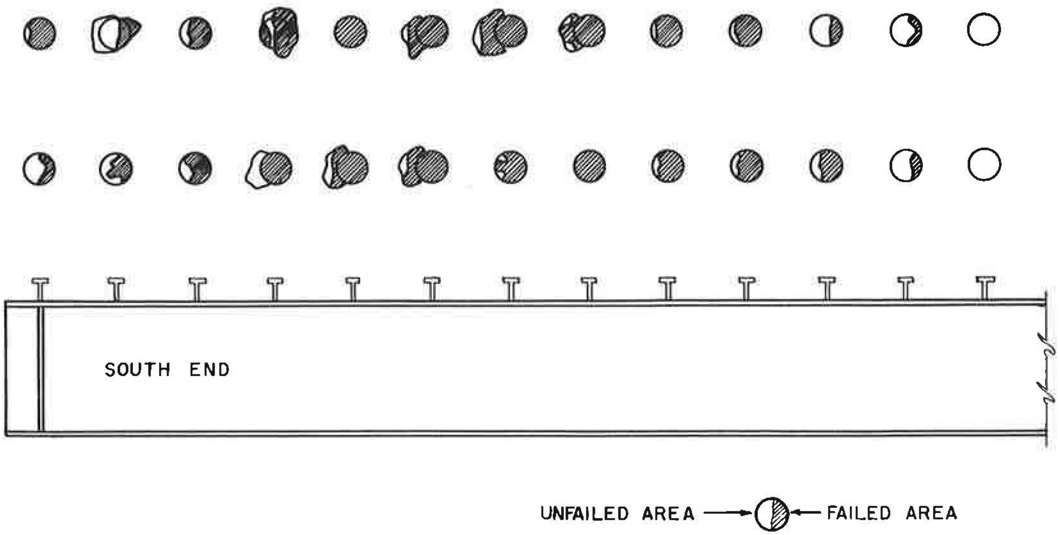


Figure 23. Representations of stud failures for specimen 1-C.

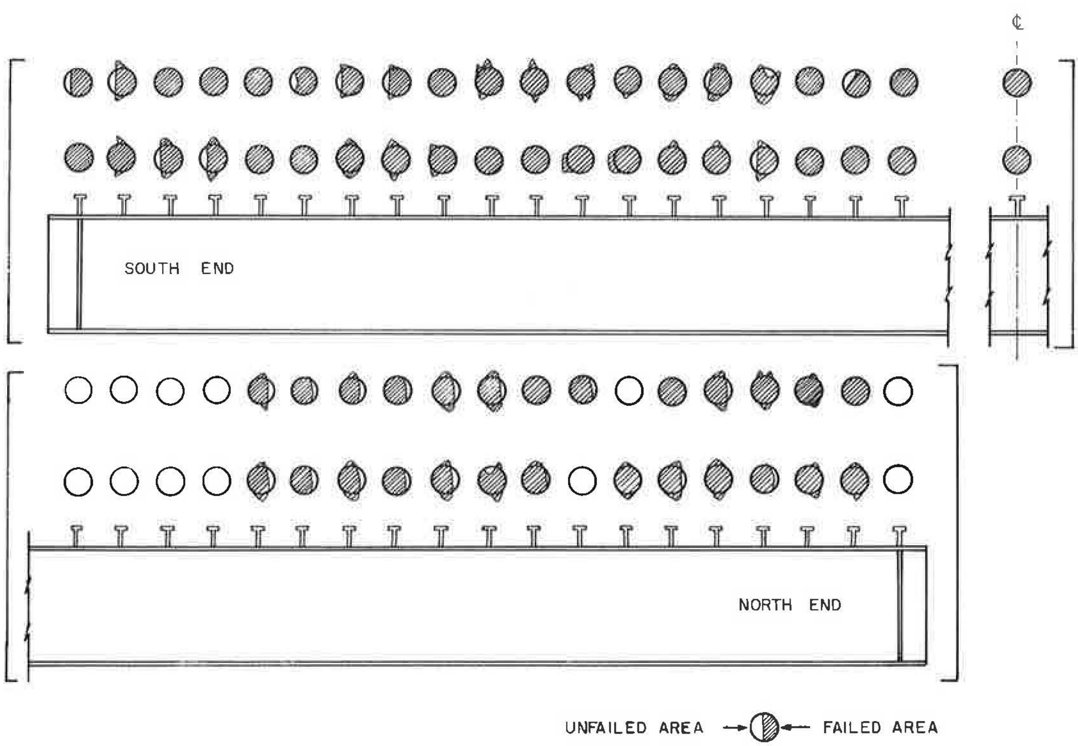


Figure 24. Representations of stud failures for specimen 1-D.

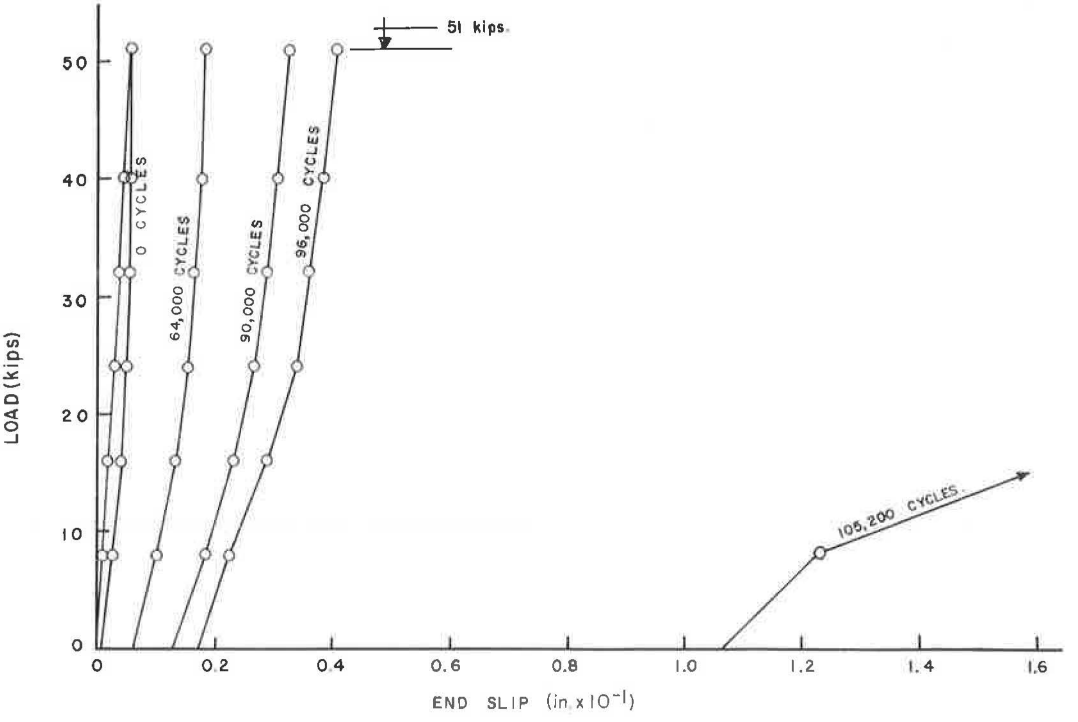


Figure 25. End slip vs load for specimen 1-A.

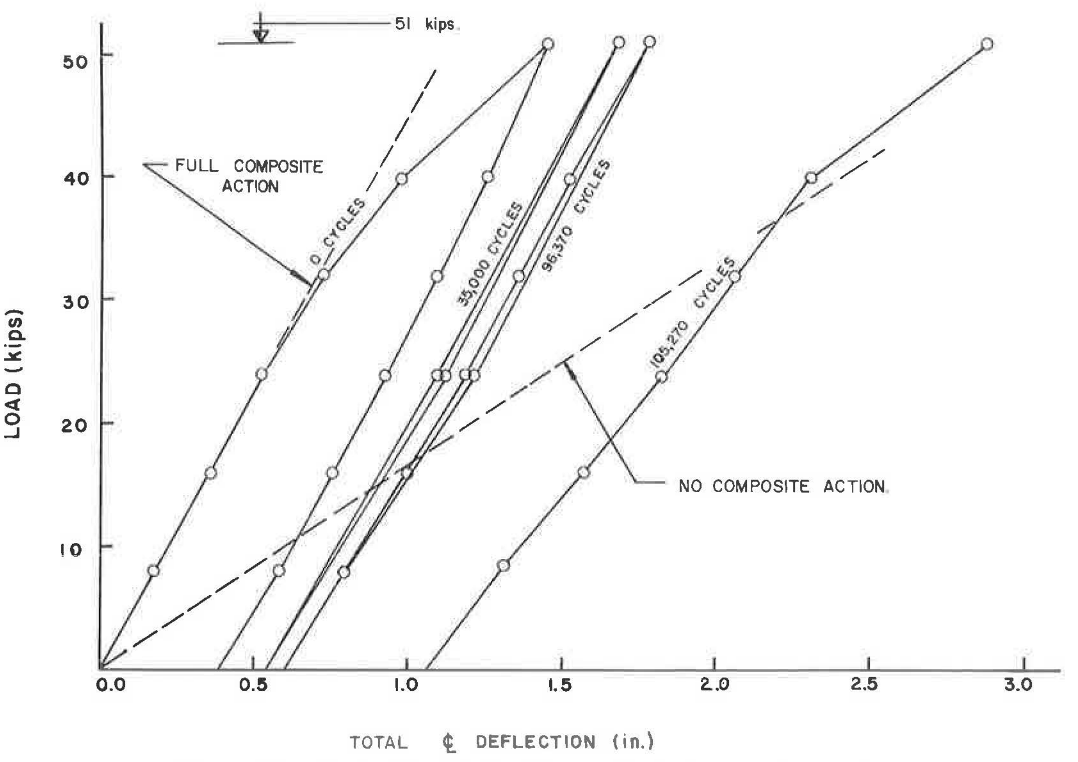


Figure 26. Centerline deflection vs load for specimen 1-A.

1-D had a fatigue failure in the tension flange of the steel section at 1,507,000 cycles. Although this failure and the subsequent repairs could have affected the structural integrity of the composite beam, testing was resumed after repairs.

Figures 23 and 24 show the condition of studs and/or flange after the concrete for beams 1-C and 1-D was removed. In both beams the south shear span (rocker end) showed the largest slip. Whereas all the studs of the south end sheared off completely, 12 (about 32 percent) at the north end were without any crack. These two sketches, which are typical, suggest that stud failure starts at or near the reaction point and proceeds toward the load points. In most specimens the pair of studs above the reaction was either without crack or if it had a crack, it was due to a defective weld.

Specimen A.--This beam was in all respects similar to 2-A with two exceptions: (a) its welds were superior and acceptable, and (b) it was subjected to 58 percent heavier loads. The maximum hydraulic cylinder load for beam 1-A was 51 kips vs 33 kips for 2-A. Such a heavy load resulted in yielding at the bottom flange and in shifting of the neutral axis.

The end slip for this beam is given in Figure 25, and Figure 26 shows the centerline deflection with residual deflections. This beam failed after 105,200 cycles.

ANALYSIS OF RESULTS

The following discussion is a comparison of the results of this study with other investigations and with current design specifications.

Criteria for Failure

Several criteria could be used in defining the fatigue life (cycles to failure) of a composite beam, including: (a) number of cycles when the first reduction in stud effectiveness occurs, (b) number of cycles when the first stud becomes completely ineffective, (c) number of cycles when the first pair of studs becomes completely ineffective, and (d) the point at which the rate of loss in composite action increases considerably. Of these, the last was chosen in this report for the obvious reason that even though studs failed in a progressive manner, no corresponding progressive increase in deflection or end slip was observed. In fact, end slip and deflection remained fairly constant throughout the test until just before the beam failed completely. Figures 27 and 28 show end slip, midspan deflection, and the neutral axis position as a function of the number of cycles of Group 1 and 2 beams. The neutral axis location was determined from strain readings at mid-span. It should be pointed out here that for specimens 1-D, 2-B, and 2-C, the deflection plotted is simply that measured as the load was increased from zero to the maximum during the static tests. The deflection for the rest of the specimens, however, represents total deflection, which includes residual deflection, from the beginning of the dynamic tests. This accounts partly for the difference in the deflections as plotted for specimens 2-A and 2-B, and 1-B, 1-C and 1-D.

The results as plotted in Figures 27 and 28 indicate that there is a definite point (number of cycles) which can be taken as the failure point and it should be used as the failure criterion instead of any other whose determination is neither easy and practical nor structurally significant.

S-N CURVE FOR 3/4-IN. STUD CONNECTORS

Loads on shear connectors were determined by applying elastic analysis to the transformed section and computing a total horizontal shear force in each shear span. This total shear force divided by the area of the studs in that particular shear span gave the average stud shear stress.

As a means of comparing the results of the present study with other fatigue tests, an S-N curve was plotted based on the results of Group 1 beams. This curve, shown in Figure 29a, was drawn as the "best fit" line between the four points using the least squares method. A second-order polynomial when fitted to these points indicated the stress at 10 million cycles to be about 13 ksi. (Specimen 1-D is included in these

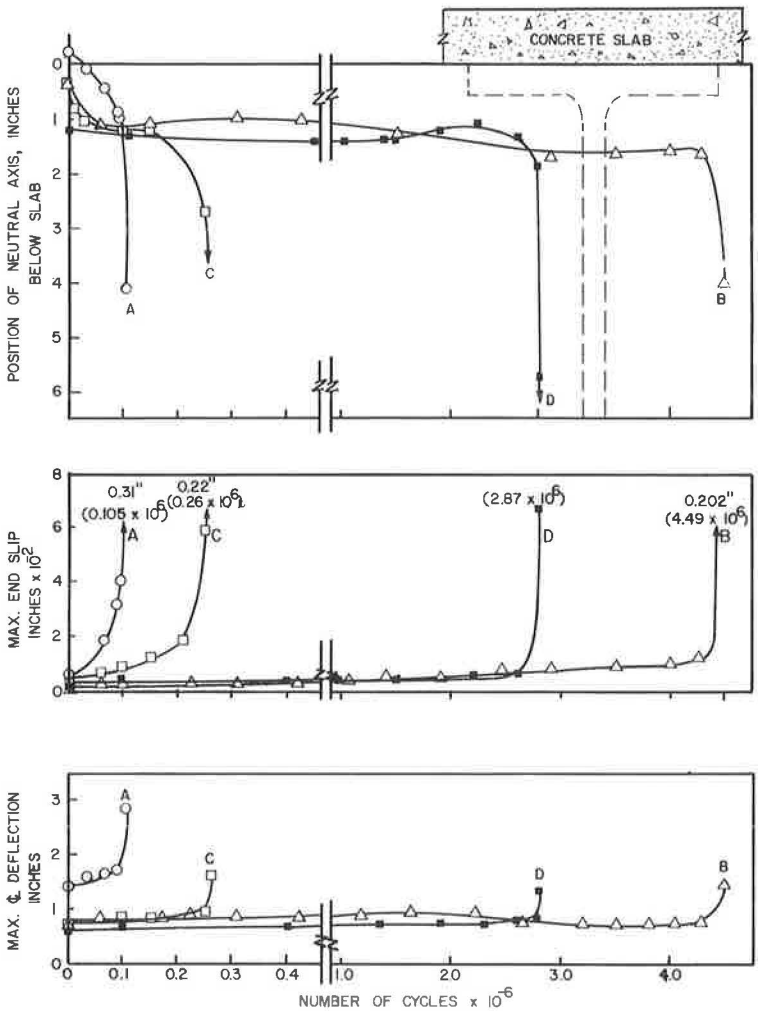


Figure 27. Deflection, end slip, and neutral axis position vs number of cycles—group 1 beams.

results even though its performance indicates that it was affected by the failure in the steel tension flange and by the subsequent operations during repairs. It would have been more logical if this beam were classified as belonging to Group 2.) Further, the curve is rather flat. Between 10,000 and 10 million cycles, the stress varies from about 20 to 13 ksi.

When the results of Group 2 were plotted, it became apparent that only beam 2-A reflected the effect of bad fabrication (defective stud welds). Admittedly, 2-A was the worst of the Group 2 beams. Possibly B and C of this group may not have been as defective as they looked. The S-N curve of Figure 29a is redrawn in Figure 29b and comparisons are made with the results of previous investigations at Lehigh University. The plotted points show that there is a "size effect" when ½- and ¾-in. studs are compared, and its magnitude in the 1 to 3 million cycle area seems to be around 3 ksi.

Assuming that the relationship as indicated by the curve is reasonably valid, a comparison of it with the AASHO specifications (8) indicates the factor of safety against fatigue for loading from zero to maximum stress as follows:

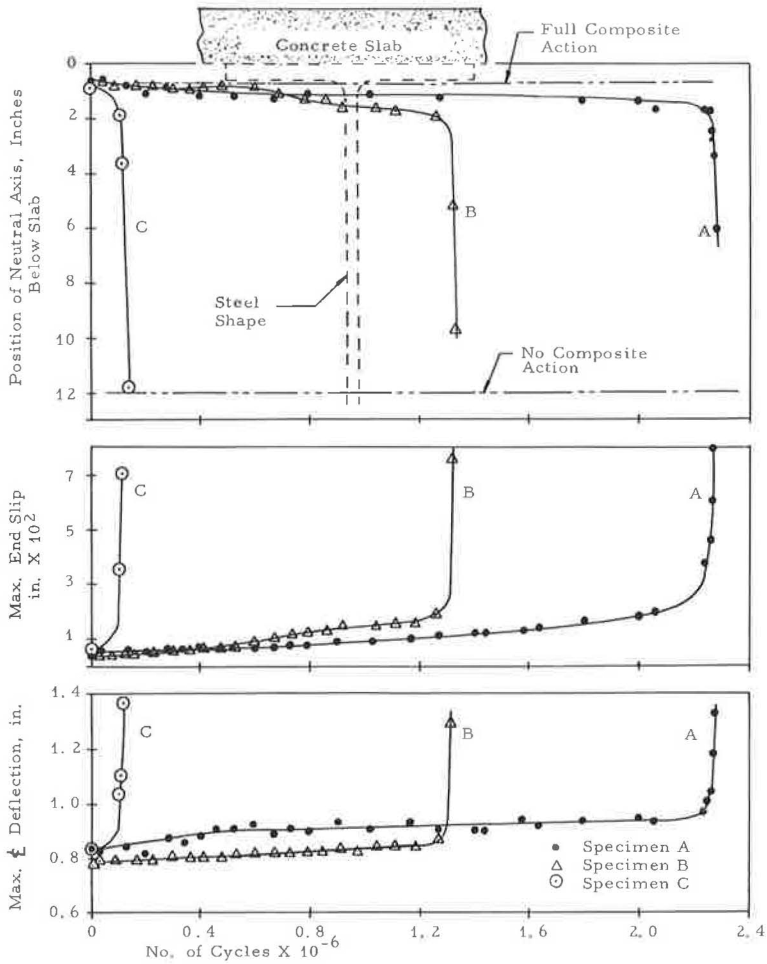


Figure 28. Deflection, end slip, and neutral axis position vs number of cycles—group 2 beams.

Assuming an average factor of safety = 3.70,

$$\text{Useful capacity} = Q_{uc} = 330\,(0.75)^2\sqrt{4400} = 12.3 \text{ kips}$$

$$\text{Allowable load} = 12{,}300/3.70 = 3{,}300 \text{ lb/stud}$$

Taking from the S-N curve in Figure 29 the stress value of 13.8 ksi for 2,000,000 cycles, we obtain:

$$\text{Load} = 13{,}800 \times 0.442 = 6.1 \text{ kips/stud}$$

Thus, the ratio of the load for expected failure at 2,000,000 cycles to the allowable design AASHO load is 6.1/3.3 = 1.85.

If similar calculations are made on the basis of the stress for 10 million cycles, we have the following:

$$\text{Strength of } \tfrac{3}{4}\text{-in. studs for } 10^7 \text{ cycles} = 13.0 \text{ ksi}$$

$$\text{Force per stud} = 13 \times 0.442 = 5.75 \text{ kips}$$

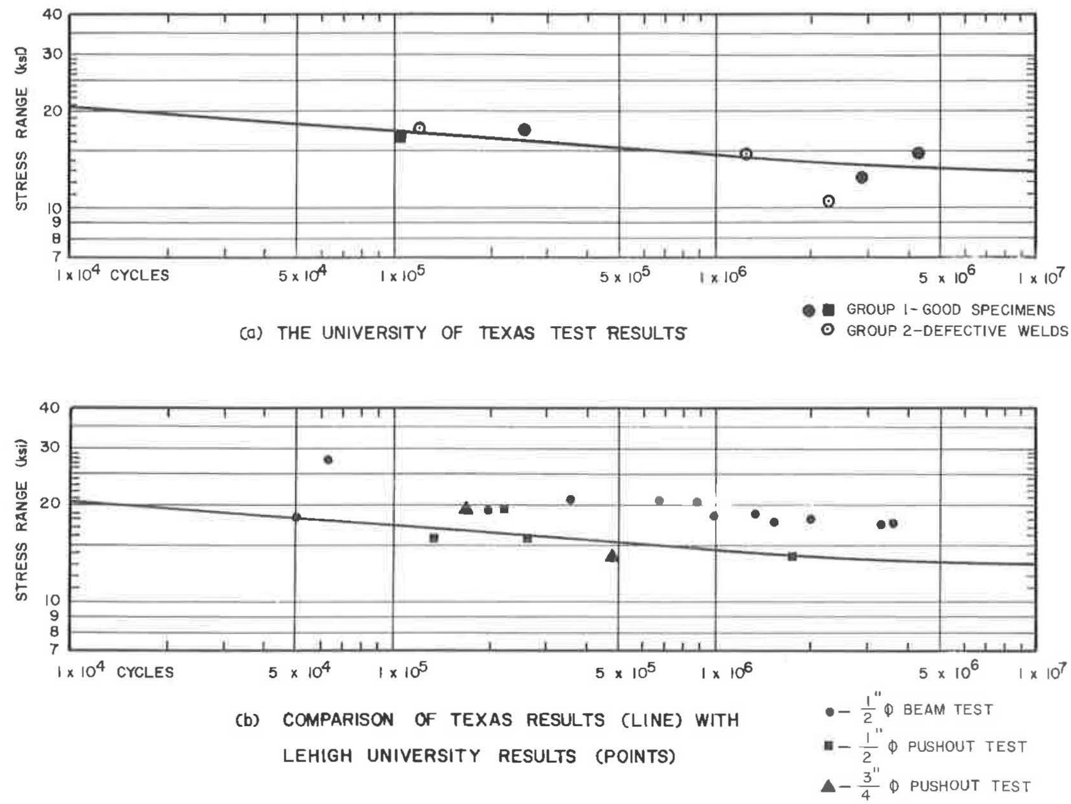


Figure 29. S-N curve for 3/4-in. stud connectors.

The factor of safety based on present AASHO specifications would be then 5.75/3.3 = 1.74. This is a rather high safety factor in view of the fact that even a very defective and totally unacceptable specimen such as beam 2-A was able to sustain 2,282,000 cycles with a stress range of 10.5 ksi. In view of this and if a factor of safety of 1.40 is agreed on, the allowable range of force per stud connector of ¾-in. diameter could be set at 4.1 kips.

CONCLUSIONS AND RECOMMENDATIONS

The small number of specimens tested makes the results of this study tentative in nature. However, augmented by observation reported by Lehigh University, the total body of data is now comparable to that of other investigations which have been used as the basis for design recommendations. Conclusions from this investigation are as follows:

1. The procedure of using strain gages to indicate the effectiveness of individual studs is reliable. It is possible to evaluate the relative effectiveness of individual studs.
2. Stud failure is progressive in nature. Individual studs showed a gradual decrease in effectiveness.
3. End slip and deflection measurements are not sensitive to individual stud failure. Most of the instrumented studs in one shear span always failed before end slip or deflection measurements showed a significant increase.
4. The fatigue life (number of cycles for the same stress) of the beams with ¾-in. studs tested in this investigation was shorter than that of the beams with ½-in. diameter

studs of earlier tests. The differences between the two investigations (Texas and Lehigh) are of the order of 3 ksi in stress range.

The question concerning the minimum number of studs required to provide an adequate factor of safety against stud fatigue failure has been partly answered. It seems reasonable that the AASHO specifications should be liberalized with respect to design of studs. Based on the results of the present study, it is recommended that the factors of safety presently used be liberalized.

ACKNOWLEDGMENTS

This research project was carried out with funds supplied by the American Institute of Steel Construction. The author is thankful for the support and cooperation given by Dr. T. R. Higgins, Director, Engineering and Research, AISC. The interest shown to this project by Dr. Ivan M. Viest is also appreciated.

REFERENCES

1. Viest, I. M., Fountain, R. S., and Singleton, R. C. Composite Construction in Steel and Concrete. New York, McGraw-Hill, 1958.
2. Viest, I. M. Review of Research on Composite Steel--Concrete Beams. Proc. ASCE, Jour. Struct. Div., June 1960.
3. King, D. C., Slutter, R. G., and Driscoll, G. C., Jr. Fatigue Tests of Composite Beams. Lehigh Univ., Fritz Eng. Lab. Rept. No. 285.3, March 1962.
4. Driscoll, G. C., Jr., Slutter, R. G., and King, D. C. Fatigue Strength of ½ Inch Diameter Stud Shear Connectors. Fritz Eng. Lab., Lehigh Univ. Unpubl. Lab. Rept., June 1963.
5. Türlimann, B. Fatigue and Static Strength of Stud Shear Connectors. Jour. ACI, Vol. 30, No. 12, June 1959.
6. Greer, D. C. (co-signed by A. W. Eatman). Letter from the Texas Highway Dept. concerning inspection of stud welding, Nov. 4, 1963.
7. Construction Bull. C-5. Texas: Texas Highway Dept., Austin.
8. Standard Specifications for Highway Bridges. 8th ed. Washington, D. C., AASHO, 1961.
9. Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings. New York, AISC, 1961.
10. Commentary on the Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings. New York, AISC, 1961.
11. Slutter, R. G., and Driscoll, G. C., Jr. The Flexural Strength of Steel and Concrete Composite Beams. Lehigh Univ., Fritz Eng. Lab. Rept. No. 279.15, 1963.