# Fatigue Strength of ½-Inch Diameter Stud Shear Connectors

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Composite steel and concrete beams were tested in fatigue at various stress levels. All twelve beams tested had ½-in. diameter welded studs as shear connectors. The beams were designed so that normal working stresses would be achieved at peak loads during repeated loading while the shear stress on connectors was sufficiently high to produce fatigue failure. Fatigue failure of connectors actually occurred in eleven of the beams.

Electrical resistance strain gages were used in eight of the test beams to detect when fatigue cracks were initiated in connectors. The use of such strain gages enabled the investigators to determine the extent of fatigue failure at any time during the testing. This information was compared with end slip and deflection data taken during the tests.

The criterion of failure was taken as the initial cracking of a pair of shear connectors. On this basis an S-N curve was obtained from the results of seven of the beam tests. A statistical analysis of these data was made and the 95 percent confidence limits of the data were obtained. The data on fatigue of stud connectors obtained by other investigators fall generally within these 95 percent confidence limits.

•COMPOSITE STEEL and concrete beams are being extensively used in structures which are subjected to fatigue loading. Various aspects of this problem as related to composite beams have been studied, but the fatigue strength of various types of full-size shear connectors has not been determined by a systematic investigation. For this reason, information on the behavior of shear connectors subjected to fatigue loading is not as extensive as the seriousness of the problem seems to warrant. The magnitude of the factor of safety which design specifications provide against fatigue failure of the concrete slab (1) or a built-up steel section (2) is generally known, but the magnitude of the factor of safety with regard to shear connectors is for the most part unknown.

Before the 1957 revision of the AASHO Standard Specifications for Highway Bridges (3) a considerable amount of research on composite beams was conducted at the University of Illinois (4, 5). Both static and fatigue tests were performed in these investigations involving beams with channel connectors. The AASHO formulas for the useful capacity of shear connectors were derived from the static behavior of beams based on limitations on the amount of slip between concrete slab and steel beam. Tests showed that by placing limitations on the magnitude of slip, fatigue failure of connectors could be prevented.

Most of the full-scale beam tests made before 1962 were conducted to verify the adequacy of the AASHO formulas. However, there has been some evidence that the AASHO specifications do not permit the maximum economy of design possible in com-

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posite construction. Before any revision of specifications can be undertaken, a thorough study of the fatigue strength of various types of shear connectors must be made.

In bridge construction today, the stud shear connector is the most commonly used type, but its fatigue behavior is not well understood. A research program was started in 1961 at Lehigh University to study the behavior of welded stud shear connectors subjected to fatigue loading.

The general objective of this investigation was to determine the fatigue strength of stud shear connectors and to determine if the design of beams could be based on this information. Fatigue tests of 12 composite beams are reported. Two groups of identical beams were tested with the only variable being the magnitude of loading on the beam. The results of these tests establish the fatigue strength of ½-in. diameter studs for one value of minimum stress.

## REVIEW OF PREVIOUS INVESTIGATION

A review of previous testing programs involving the fatigue of stud shear connectors is useful so that this information can be analyzed along with the new test results. The previous tests consist of three approaches to the problem of investigating the fatigue strength of shear connectors. These are fatigue tests of bare studs, of pushout specimens, and of composite beams.

### Tests of Bare Studs and Pushout Specimens

Studs welded to steel plates without being incased in concrete were tested with stress reversal by a load applied perpendicular to the stud (6). These tests were performed with the force applied to the head of a ¾-in. diameter by 4-in. long stud. Sufficient results were obtained at various stress levels to establish the S-N curve for this loading condition, given as the upper curve in Figure 1. These results were quite high, and there would apparently be little danger of fatigue failure of stud connectors in composite beams.

Pushout specimens consisting of concrete slabs 6 in. thick attached to the flanges of 8 WF 40 beams by four connectors in each slab were tested (7). The results obtained from these tests are summarized in Table 1. The term hooked refers to studs having a 90 deg bend at the top. The horizontal leg at the top of a ½-in. diameter stud was 1½ in. long. The maximum and minimum shear stresses were obtained by dividing the maximum and minimum loads applied to the specimen by the area of the shear connectors.

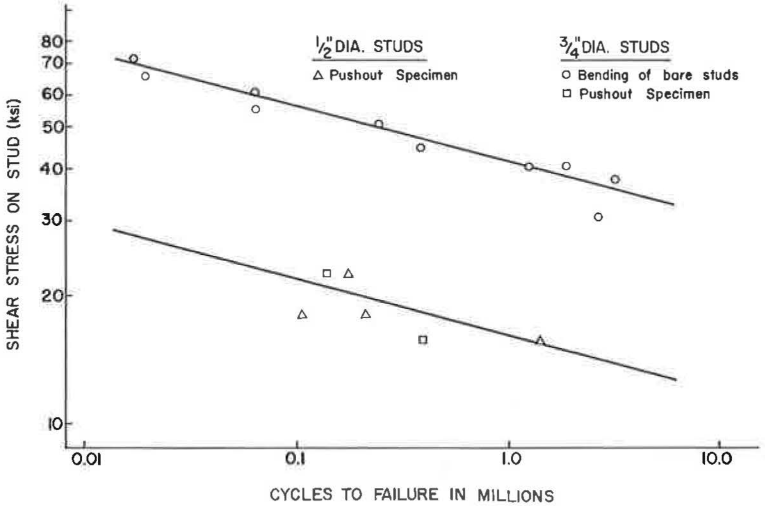



Figure 1. S-N curves for previous tests of stud shear connectors.

TABLE 1

SUMMARY OF FATIGUE TEST RESULTS BY OTHER INVESTIGATORS

| Specimen Number | Reference | Type of Stud Connector | Type of Test Specimen | Minimum Shear Stress on Stud (psi) | Maximum Shear Stress on Stud (psi) | Cycles of Loading | Results |
|---|---|---|---|---|---|---|---|
| 4 | 7 | 1/2" Dia. Hooked | Pushout* | 2900 | 22,300 | 223,200 | Stud Fracture |
| 5 | 7 | 1/2" Dia. Hooked | Pushout* | 2200 | 17,800 | 134,200 | Stud Fracture |
| 6 | 7 | 1/2" Dia. Hooked | Pushout* | 2200 | 17,800 | 261,000 | Stud Fracture |
| 7 | 7 | 1/2" Dia. Hooked | Pushout* | 1900 | 15,600 | 1,748,000 | Stud Fracture |
| 9 | 7 | 3/4" Dia. Headed | Pushout* | 2800 | 22,300 | 169,400 | Stud Fracture |
| 10 | 7 | 3/4" Dia. Headed | Pushout* | 1700 | 15,600 | 474,000 | Stud Fracture |
| Bridge | 8 | 1/2" Dia. Bent | Beam | 1850 | 15,700 | 256,800 | No Failure |
| B4 | 9 | 1/2" Dia. Bent | Beam | 1500 | 21,000 | 619,000 | No Failure |
| B4 | 9 | 1/2" Dia. Bent | Beam | 1500 | 24,100 | 122,400 | No Failure |

*Concrete slabs on these specimens were 28 inches high by 20 inches wide

The data of Table 1 are plotted in Figure 1. The S-N curve through these points was arbitrarily drawn parallel to the upper curve. A statistical analysis of these data would result in a flatter curve than the one shown. Because only a small amount of data are available and there were several variables involved, the approximate curve shown in Figure 1 was considered acceptable for planning the new test series. The important point to be made concerning Figure 1 is that there is a vast difference between the two curves. It is, therefore, imperative that the correct curve for composite beams be known. The two curves of Figure 1 were taken as the probable upper and lower limits of the test results in planning the tests.

It is of considerable interest in making these tests to determine if pushout test results are comparable to beam test results since the testing of composite beams in fatigue is an expensive and time-consuming process and the pushout test is more easily performed and could be effectively used in extending the research work into other areas of interest.

## Beam Tests

Two composite beams with stud shear connectors were tested in fatigue at Lehigh University (8, 9). The first member consisted of two 18 WF 50 steel beams with a concrete slab 18.0 ft wide by 6 in. thick attached to the top flange by ½-in. diameter studs on one beam and ¾-in. diameter studs on the other beam. This member was tested on a span of 30 ft. The second member having a span of 10 ft consisted of an 8 WF 17 steel section with a concrete slab 4 in. thick and 2 ft wide attached to the top flange with ½-in. diameter studs.

Both of these beams were subjected to fatigue loading, but no failure of connectors occurred during the tests. The results of these tests were limited except to verify that the AASHO design specifications are satisfactory from the point of view of limiting shear connector stresses to values which probably prevent fatigue failure.

The data from the two beam tests are included in Table 1. The maximum and minimum stresses on the studs are calculated stresses. The shear stress on the connector, $f_s$, was calculated by

$$f_s = \frac{VQS}{I_{tr} A_s} \tag{1}$$

where V is the applied shear force at the cross-section, Q is the first moment of the transformed concrete slab area, S is the spacing of studs having a cross-sectional area of $A_s$, and $I_{tr}$ is the moment of inertia of the transformed composite section. These data have not been plotted on Figure 1 since no failures were obtained. A comparison of the data from the beams with data from the pushout test specimens reveals that the beam test points would plot near the lower curve of Figure 1.

The data contained in Table 1 provide no basis for conclusions concerning the fatigue strength of stud connectors in composite beams. The only conclusion that can be drawn is that beam specimens must be tested at load levels which will produce fatigue failures of connectors. Such tests were conducted and the results are reported and analyzed in the subsequent sections of this report.

## EXPERIMENTAL PROGRAM OF THIS INVESTIGATION

### Scope

The investigation was limited to $\frac{1}{2}$-in. diameter welded stud shear connectors to match the specimen size to the capacity of the available loading equipment. An additional advantage of using $\frac{1}{2}$-in. diameter connectors was that more information was available for this size of connector than for any other. It was assumed that information obtained from these tests could be extrapolated to larger sizes of stud shear connectors, and that these extrapolated values would be verified by later tests.

### Preliminary Beam Tests

Before begining a full-scale series of fatigue investigations, it was decided that some preliminary tests should be made: (a) to produce fatigure failure of connectors in a beam; (b) to develop a method to determine exactly when a connector failed in fatigue; and (c) to develop a more comprehensive instrumentation and testing procedure for future tests.

Description of Specimens.—Each of four beams for the preliminary tests consisted of a 2-ft wide by 3-in. thick concrete slab cast onto an 8 WF 17 steel beam as shown in Figure 2. The shear connection consisted of $\frac{1}{2}$-in. diameter hooked welded stud connectors. The spacing of these connectors is also shown in Figure 2. The section properties of the four specimens are given in Figure 3.

The testing of the specimens took place between 28 and 79 days after pouring. The average concrete strength at the time of testing for BF-A and BF-B was 3,030 psi and that for BF-C and BF-D was 3,500 psi.

Instrumentation.—The instrumentation consisted of electrical resistance strain gages at midspan under the top and bottom flanges of the steel section, a midspan deflection gage, and slip measuring devices at both ends and near the quarter points. The location of the strain gages is as shown in Figure 4. The locations of the deflection gage and four slip gages are shown in Figure 5. The slip gages consisted of 0.001-in. dial gages for the dynamic readings and 0.0001-in. dial gages for the static readings.

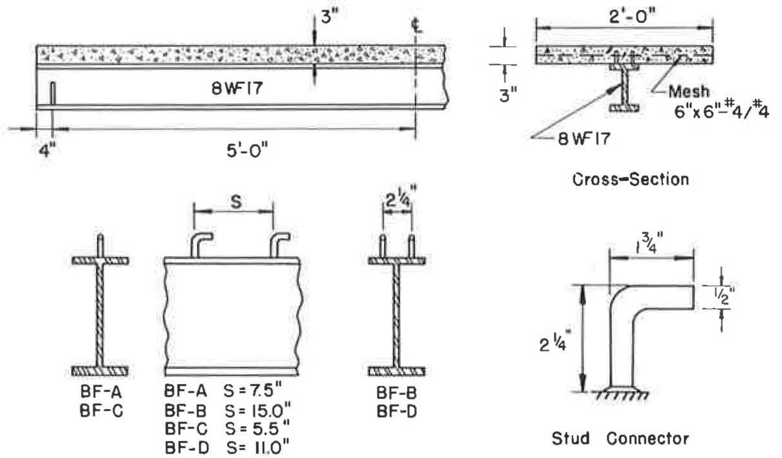
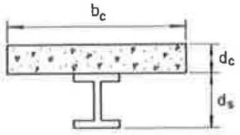


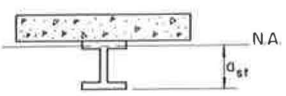

Figure 2. Dimensions of test specimens BF-A through BF-D.




Concrete Slab
$b_c$ = 24 in.
$d_c$ = 3 in.
$f'_{c_{design}}$ = 3,500 psi
n = 10

Steel Beam (8 WF 17)
$d_s$ = 8.00 in.
$A_s$ = 5.00 sq. in.
$I_s$ = 56.4 in.$^4$
$f_{y_{design}}$ = 33,000 psi

Composite Section
$a_{st}$ = 7.48 in.
I = 156.0 in.$^4$




Studs (L-connector)
diameter = ½ in.
height = 2.25 in.
area = 0.196 sq. in.

Figure 3. Section properties for BF-A through BF-D.

Test Procedure.—The specimens were moved to the loading frame and testing was begun after the specimens had been wet-cured for 2 wk and air-cured for a minimum of 2 wk. Each specimen was initially loaded to a static value sufficient to break the bond between beam and slab. The testing arrangement is shown in Figure 5.

While being loaded statically, all strain gage deflection and slip gage readings were taken at intervals of 1.5 kips per jack. After this initial static test each specimen was loaded dynamically at 250 cycles/min. Static tests were taken at intervals until failure occurred. Periodically, dynamic end slip and deflection readings were taken.

## Primary Beam Tests

In the preliminary beam tests stud fatigue failures could be produced for the first time in a beam specimen. However, these tests supplied only four points for plotting the S-N curve for ½-in. diameter stud connectors. Obviously, many more points would be required firmly to establish the position of the S-N curve.

Also, even after the conclusion of the preliminary beam tests, a method had not been perfected for determining when a connector actually failed in a beam under a fatigue loading. The points taken as failure were determined on the basis of a visual observation that vertical movement of the slab with respect to the beam was taking place. However, it was observed that at the number of cycles designated as failure, several shear connectors were actually fractured.

As a result of these problems the primary beam tests concentrated on obtaining additional points for the S-N curve and on perfecting a method by which the initial failure of a connector in a beam could be determined.

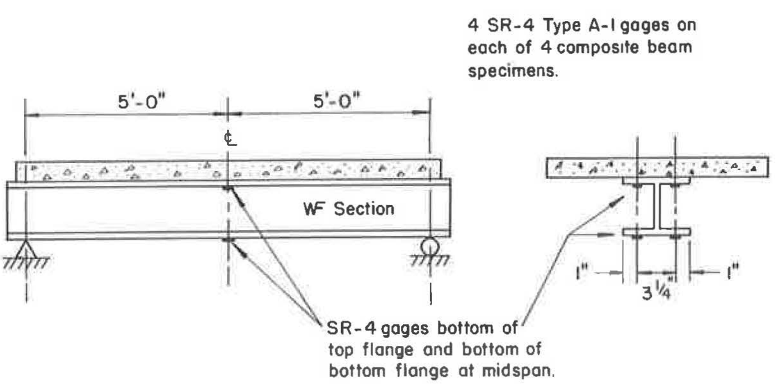



Figure 4. Strain gage locations for BF-A through BF-D.

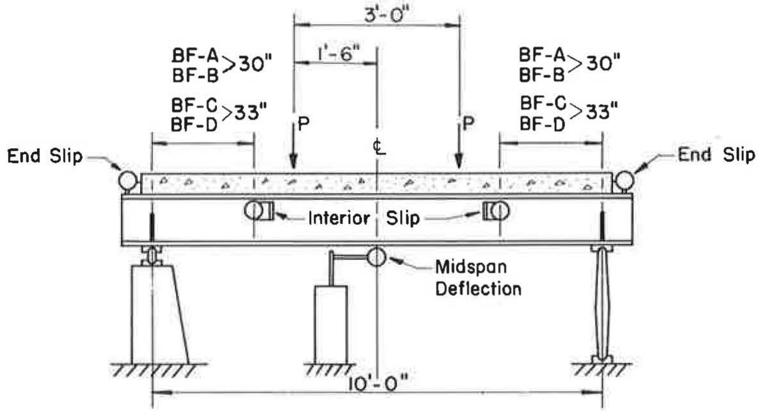



Figure 5. Test setup for BF-A through BF-D.

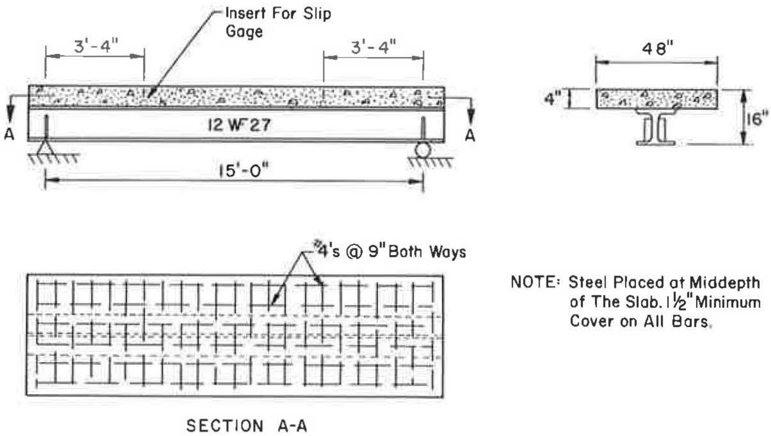



Figure 6. Typical beam fatigue specimens BF-1 through BF-8.

Description of Specimens.—The primary test program included eight identical steel and concrete composite beams. Each beam consisted of a concrete slab 4 ft wide and 4 in. thick connected to a 12 WF 27 steel beam by means of ½-in. diameter welded stud shear connectors. The rolled section was of ASTM A7 steel. The concrete slabs were cast at Fritz Engineering Laboratory using transit-mixed concrete proportioned for a 28-day compressive strength of 3,000 psi. Four test cylinders were poured with each test beam.

The shear connectors were ½-in. diameter headed studs which varied in length after welding from 2 in. to 2⅛ in. The studs were welded by a stud-welding process at a local fabrication shop. The welding was typical of general shop welding in quality. Connectors were arranged in pairs on the eight test beams. Details of the specimens are shown in Figure 6. The concrete slab reinforcement consisted of No. 4 bars at 9 in. center-to-center in both the longitudinal and transverse directions. The transverse slab reinforcement was supported 1½ in. from the bottom of the slab, and the longitudinal reinforcement was supported by the transverse steel. The arrangement of the 40 shear connectors, identical in all eight test members, is shown in Figure 7. Section properties and design strengths of the composite beams are given in Figure 8.

In the selection of the size of the test specimens, it was desirable to choose a size of member such that the dynamic loading correction would not become appreciable

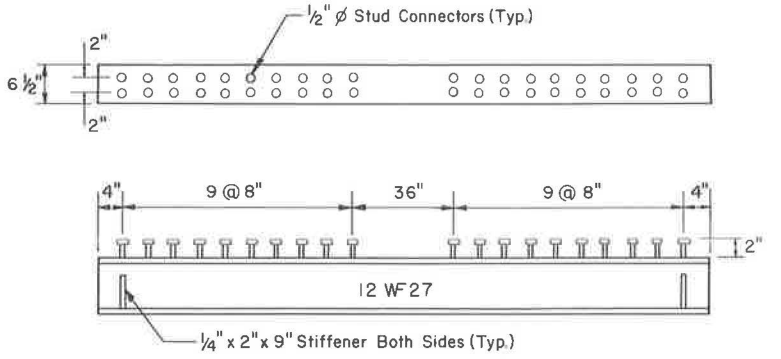



Figure 7. Stud shear connector arrangement for BF-1 through BF-8.

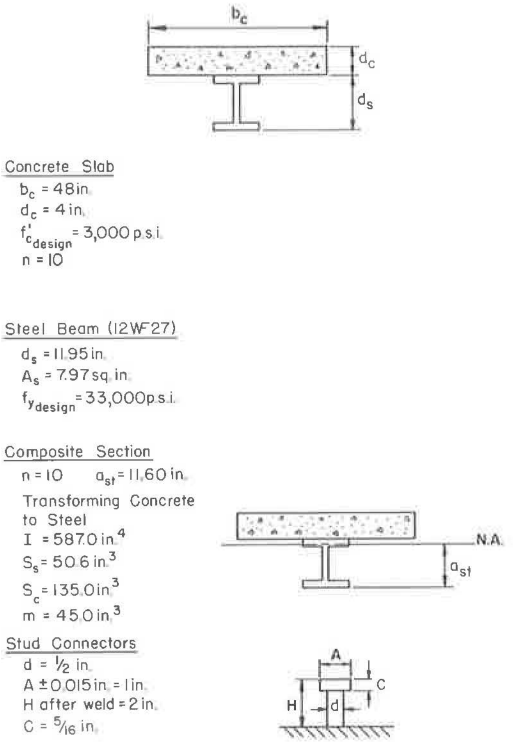



Figure 8. Section properties for BF-1 through BF-8.

during the test even though the effective stiffness of the member might decrease. This was an important consideration since it was desirable to test some of the beams until there was little or no composite action remaining. The size of the member made it necessary to use ½-in. diameter connectors to have a sufficient number so that the change in properties of the beam with cycles of load would be gradual and could, therefore, be studied carefully.

No studs were placed between the loading points because the loading points were placed close enough together so that the hydraulic jacks being used to apply the test load were sufficient to prevent separation of the slab and beam. This was done so that at all times it could be determined exactly which connectors were effective in transferring shear stresses. In the preliminary beam tests, it had been observed that connectors located between loading points were being forced to carry shear by means of the slab reinforcing steel. The effectiveness of these connectors in transferring shear was difficult to evaluate. Actually it probably varied depending on the magnitude of slip and the condition of the connectors near the ends of the member.

The number of connectors supplied in these test members was sufficient to develop the static ultimate moment capacity of the member. It had been established in a previous investigation of the static strength of composite beams that this minimum amount of shear connection should be provided to avoid reduction of the ultimate moment capacity by shear connector failure (10). The magnitude of the bottom flange steel stresses was limited to magnitudes less than the yield stress so that fatigue failure of the steel section would not occur.

Instrumentation.—The instrumentation consisted of dial gages at midspan to measure deflection and at both ends and 3 ft 8 in. from each end to measure slip, as well as numerous electrical resistance strain gages. The dial gages located at the ends and at midspan for measurement of deflection were 0.001-in. gages. The other two dial gages were 0.0001-in. gages. The location of electrical resistance strain gages varied from test to test, depending on the data required.

The midspan deflection gage was used in adjusting the dynamic load at the beginning of each test. Since the bending stiffness of the beam changes because of bond failure at the beginning of the test, the magnitude of the applied load changes. The dynamic loading correction also changes as bending stiffness varies. Therefore, the midspan deflection was held constant until bond failure was complete, generally by 5,000 cycles, determined by visual inspection. After complete bond failure, the load was held constant and the midspan deflection was allowed to vary as the test continued. The change in deflection with cycles became an indication of loss of interaction in the member. The change in deflection was difficult to detect with a 0.001-in. dial gage until after a substantial number of connectors had failed. A more sensitive gage could not be used because of the necessity of disconnecting the instrument during dynamic loading.

Electrical resistance strain gages were placed on the bottom of both the top and bottom flanges at midspan on all beams. Two methods of determining initial connector failure by electrical resistance strain gages were studied during the testing of beam BF-1. The first method consisted of instrumenting cross-sections of the beam on each side of a pair of connectors. Although connector failure could be detected by comparing data from these gages, the method was not satisfactory because the magnitude of the changes in strain due to connector failure were too small.

A second method of detecting connector failure was based on the assumption that the connector forces caused local bending stresses in the top flange of the beam as indicated in Figure 9. This method proved to be quite sensitive to changes in the condition of connectors, and could be used to detect the initial growth of a fatigue crack. The best location for these gages was determined experimentally, and is indicated in Figure 10 which shows the location of all strain gages used in the tests.

The slip gages were used to measure the movement of the slab relative to the steel beam and to serve as a general indication of connector failure. The slip gages also indicated when connector failure began seriously to affect interaction. The interior gages were removed during dynamic testing. The stems of end dials were isolated from contact with the specimen during dynamic tests, but these dials were not removed from the member.

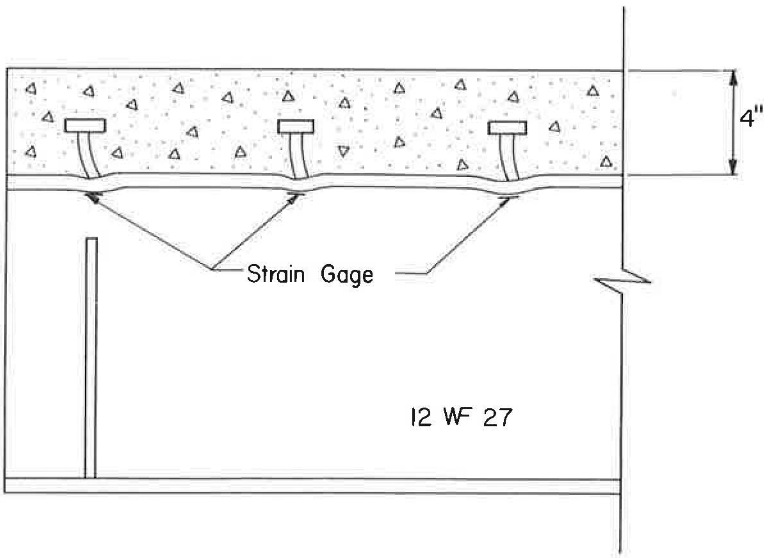



Figure 9. Distortion of top flange of steel beam due to shear connector load.

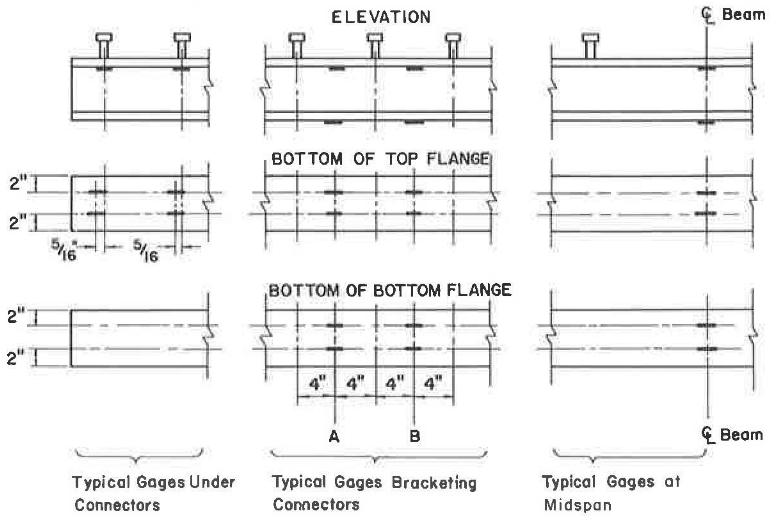



Figure 10. Typical strain gage locations for BF-1 through BF-8.

TABLE 2

INSTRUMENTATION USED WITH EACH SPECIMEN

| Specimen | End Slip Gages | Interior Slip Gages | Midspan Deflection Gage | Number of Electrical Resistance Strain Gages Used | | |
|---|---|---|---|---|---|---|
| | | | | Midspan | Opposite Connector | Bracketing Connector |
| BF-A | x | x | x | 4 | | |
| BF-B | x | x | x | 4 | | |
| BF-C | x | x | x | 4 | | |
| BF-D | x | x | x | 4 | | |
| BF-1 | x | x | x | 4 | 4 | 14 |
| BF-2 | - | - | x | 4 | 10 | - |
| BF-3 | x | x | x | 4 | 8 | - |
| BF-4 | x | x | x | 4 | 8 | - |
| BF-5 | x | - | x | 4 | 8 | - |
| BF-6 | x | - | x | 6 | 8 | 24 |
| BF-7 | x | - | x | 4 | 17 | - |
| BF-8 | x | - | x | 4 | 8 | - |

For members BF-1 through BF-4, strain gages with a gage length of 13/16-in. were used. Starting with member BF-5, strain gages with a gage length of 1/4-in. were tried for measuring the local stresses near connectors. These were found to be slightly more sensitive than the larger gages. Local stresses being measured were found to be confined to an area having a diameter only about twice that of the connectors.

Once the behavior of the strain gages opposite connectors was determined, it was not necessary to use as many gages to detect initial failure of connectors. After tests of a few members were completed it was found that the slip and deflection data were not as significant as the strain gage data in studying the behavior of individual connectors. The interior slip gages were omitted in some tests. The major difficulty with these gages is the fact that the sensitivity of a 0.0001-in. dial gage is required to detect the minute

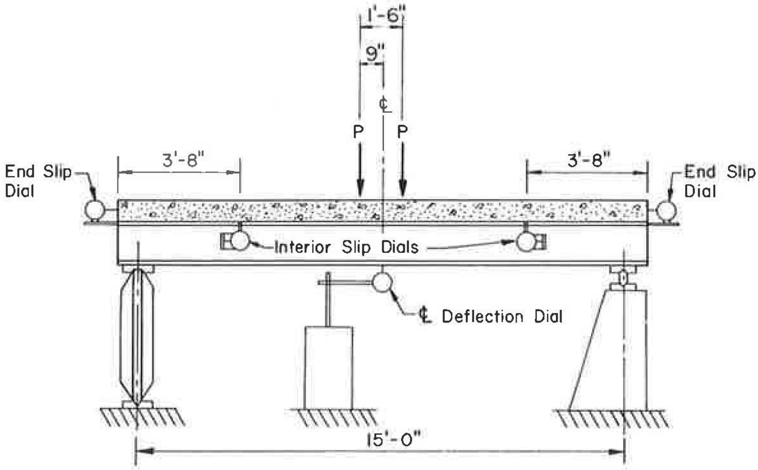



Figure 11. Test setup for BF-1 through BF-8.

TABLE 3

CONCRETE AGE AND STRENGTH AT TIME OF TESTING

| Beam | Concrete Age at Start of Test | $f'_c$ (psi) |
|---|---|---|
| BF-A | 28 | 3030 |
| BF-B | 30 | 3030 |
| BF-C | 36 | 3500 |
| BF-D | 45 | 3500 |
| BF-1 | 33 | 3330 |
| BF-2 | 33 | 3290 |
| BF-3 | 39 | 3350 |
| BF-4 | 39 | 3490 |
| BF-5 | 37 | 3310 |
| BF-6 | 58 | 3108 |
| BF-7 | 70 | 4060 |
| BF-8 | 84 | 3980 |

changes in slip caused by cracking of connectors, but these gages are too delicate to be used during dynamic loading.

The instrumentation used on each of the eight test beams is summarized in Table 2. The general arrangement of electrical resistance strain gages is shown in Figure 10. The locations of slip and deflection dial gages are shown in Figure 11.

Test Procedure.—Each beam was simply supported on a span of 15 ft and loaded by hydraulic jacks located 9 in. on each side of the centerline. The arrangement for testing of members is shown in Figure 11. Testing was started at least 28 days after the concrete slabs were cast. In some cases the concrete was older than 28 days when testing began. Concrete slabs were moist-cured for 7 days and then air-cured until time of testing. The concrete strength and age of the eight specimens are given in Table 3. Four concrete test cylinders were poured with each beam. Two of these cylinders were tested when dynamic loading was started. The other two were tested at the end of the test. The concrete strength given in Table 3 is the average of the four cylinders tested.

Initially each specimen was loaded statically to the maximum load to be applied dynamically. None of the members were overloaded statically. If the bond between the steel beam and the concrete slab was broken throughout the length of the member due to the initial static test, the deflection measurements were used to determine the correct dynamic load. If the bond was not broken by the initial static test, cycling was begun using a theoretically determined load until bond was completely broken.

The maximum load to be maintained during dynamic testing was determined from previous results. A second static test was made as soon as bond failure was complete. Generally it required about 5,000 cycles to break bond, but on one member 7,000 cycles were required. The midspan deflection was measured on the second static test and was used in adjusting the dynamic loading equipment for the correct jack load. Thereafter, the loading equipment settings were held constant and deflection of the member was allowed to change.

Throughout an entire test, static tests were run at regular intervals. During each static test, the member was loaded in increments of 2 kips per load point to the maximum test load. All dial gages and electrical resistance gages were read at each load increment whenever it was judged from the behavior of the specimen that the data would be significant. In some of the tests, complete readings were taken only at zero load and maximum load.

All specimens were loaded at the rate of 250 cycles/min. The minimum load was the smallest that could be applied without separation of beam and loading jack at any time during the load cycle. Generally this minimum load was approximately 10 percent of the maximum load.

After the completion of each test, the concrete slab was removed from the steel beam and a visual inspection of the connector failures was made. Photographs were made of connector failures and cracked connectors. The final visual inspection was used as a check on the information gained from electrical resistance strain gage data. In several instances, this final inspection verified strain gage data which were in doubt at the completion of the test. These inspections were important in establishing confidence in the technique used to detect connector failures.

## RESULTS OF BEAM TESTS

During the course of the preliminary and primary beam tests, it was possible to obtain data for eleven points with which to establish an S-N curve for ½-in. diameter studs. The S-N curve obtained was derived using only data from seven of the eight primary beam tests in which fatigue failure of connectors occurred. This curve was obtained by a regression analysis of the data using the following mathematical model:

$$\log N = A + B\ (S_{max.} - S_{min.}) \tag{2}$$

in which

A, B = empirical constants,
$S_{max.}$ = maximum shear stress,

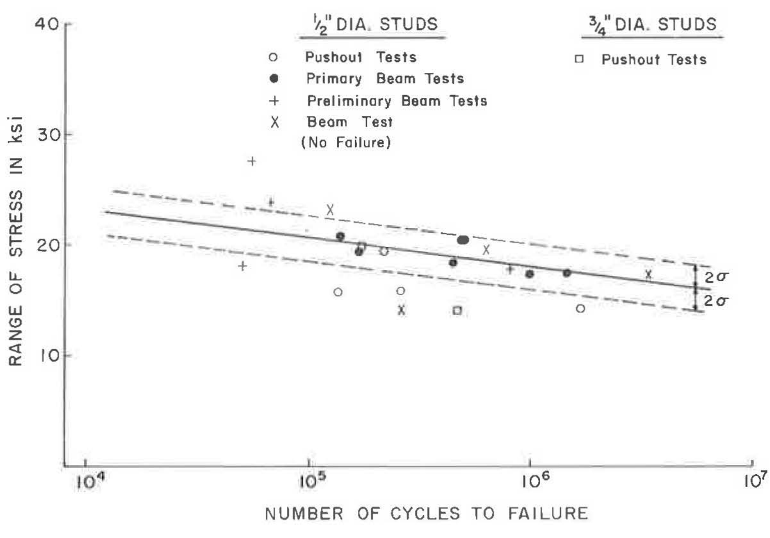



Figure 12. S-N curve for primary beam tests.

$S_{min.}$ = minimum shear stress, and
$N$ = number of cycles to failure.

The resulting curve is plotted in Figure 12 with the range of stress as ordinate and the number of cycles to failure as abscissa. Data points from all beam and pushout tests are shown. The failure criterion for determining the value of N differs for each group of tests. That for pushout specimens was complete failure of connectors on one or both flanges. The failure criterion for the preliminary beam tests and primary beam tests is described in the following sections. A statistical analysis of the seven data points from the primary beam tests resulted in an unbiased standard deviation of $\sigma = 1.027$ ksi. The 95 percent confidence limits of the data are shown in Figure 12.

Data points from pushout tests are shown along with other test results in Figure 12. Generally the pushout test results fall below the curve, but the curve is much closer to the pushout results than it is to the results obtained from tests of bare studs as can be seen by comparison of Figure 12 with Figure 1. It appears that pushout tests can be used in evaluating the fatigue strength of shear connectors.

Although Figure 12 presents the general results of the tests, much of the additional data taken and many of the visual observations made during the tests are of interest. The additional information has considerable bearing on the interpretation of the results given in Figure 12 and will be presented before making a complete evaluation of the S-N curve. The results of the preliminary beam tests are considered separately. The subsequent sections of the report are concerned mostly with the primary beam tests but some of the information pertains to the preliminary tests also.

## Preliminary Beam Tests

The details of these test specimens have been already presented, and section properties are given in Figure 3. The behavior of these members as fatigue failure of connectors took place was similar to the behavior of composite beams with channel shear connectors tested at the University of Illinois (5). The most difficult problem in connection with performing these tests was to determine when a connector somewhere on the specimen first developed a fatigue crack or when it became completely fractured.

In all members, failure began with the end pair of connectors at the expansion end of the member. Shortly thereafter failure occurred at the opposite end of the member. Failure of connectors then progressed rather gradually from both ends toward the center. From the start of a test until a sufficient number of connectors had failed so that a noncomposite member remained, there was no sudden change in applied loads, strains, slip, or deflection.

The most complete data were obtained on specimen BF-D because the shear connector stress on the other three test specimens was higher and failure took place before very much data were obtained. Beam BF-D was loaded so that the maximum stress on the connectors was equal to the useful capacity of this type of connector as specified in Section 1.9.5 of the 1961 AASHO specifications (3).

The test results of preliminary and primary beam tests are summarized in Table 4. The minimum stress on the connectors was always approximately 10 percent of the maximum stress, and hence the S-N curve of Figure 12 is actually based on data from tests in which the stress range was approximately 90 percent of the maximum stress.

As each member was cycled between the minimum and maximum loads given in Table 4, the first obvious indication of failure was an audible banging of the slab on the steel section. For the preliminary beam tests, this determined the number of cycles to failure recorded in Table 4. This is also the failure criterion used in plotting the preliminary beam test data points in Figure 12.

The visual inspection method of detecting failure is not precise, and an analysis of data on slip, strains, and deflection was also used in attempting to determine N. Since N could only be determined approximately by using all of the information available, a failure zone was defined as being the probable range of N within which failure occurred. A failure zone for beam BF-D is indicated in Figures 13 and 14. The left edge of the failure zone was determined by the first indication of failure which could be discerned from the data, and the right edge was established by the first positive proof that failure had occurred.

TABLE 4

SUMMARY OF TEST RESULTS

| Specimen | Minimum (kips/jack) | Maximum (kips/jack) | Calculated Stud Stress* Minimum | Calculated Stud Stress* Maximum | Static Test Interval (kc) | Cycles to Failure+ (kc) | Total Number of Cycles during Test (kc) |
|---|---|---|---|---|---|---|---|
| BF-A | 1.6 | 13.5 | 5730 | 23,900 | 50.3 | 50.3 | 50.3 |
| BF-B | 1.3 | 11.5 | 4900 | 32,600 | 50.3 | 55.4 | 63.9 |
| BF-C | 1.1 | 11.0 | 3600 | 27,700 | 100.0 | 78.0 | 198.5 |
| BF-D | 0.7 | 7.0 | 2160 | 20,100 | 100.0 | 820.0 | 2,008.0 |
| BF-1 | 1.2 | 14.2 | 1880 | 22,200 | 50.0 | 490.0 | 880.0 |
| BF-2 | 1.2 | 14.2 | 1880 | 22,200 | 50.0 | 480.0 | 680.0 |
| BF-3 | 1.2 | 12.4 | 1880 | 19,400 | 100.0 | 980.0 | 1,556.0 |
| BF-4 | 1.4 | 12.4 | 2190 | 19,400 | 100.0 | -- | 3,315.0 |
| BF-5 | 1.2 | 14.5 | 1880 | 22,600 | 50.0 | 140.0 | 354.0 |
| BF-6 | 1.2 | 13.5 | 1880 | 21,100 | 50.0 | 168.5 | 1,009.0 |
| BF-7 | 1.2 | 13.0 | 1880 | 20,300 | 50.0 | 450.0 | 1,344.0 |
| BF-8 | 1.2 | 12.5 | 1880 | 19,500 | 50.0 | 1,445.0 | 3,522.0 |

*For BF-A to BF-D, shear stress was determined by dividing the compressive force in the slab at midspan by the number of connectors in the shear span. For BF-1 to BF-8, shear stress was determined by Eq. 2.1

+ Failure for BF-A to BF-D was based upon slip and deflection data, but failure of BF-1 to BF-8 was based on the average number of cycles to produce fatigue crack in a pair of studs

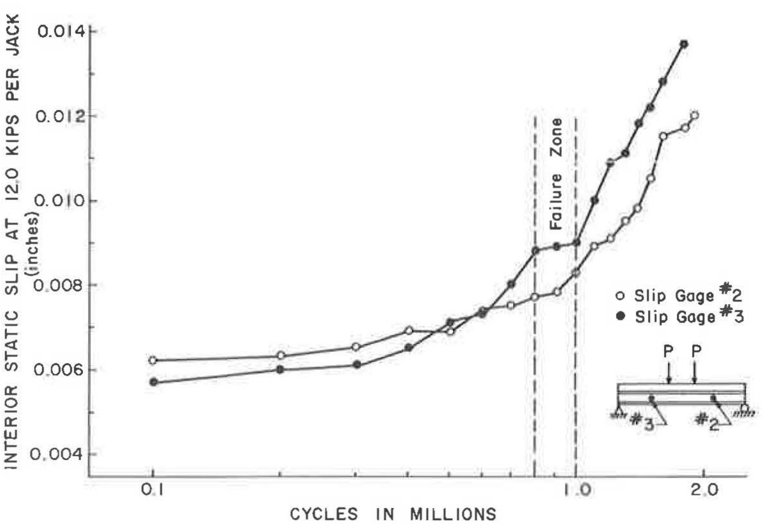



Figure 13. Interior static slip vs cycles for BF-D.

Failure of connectors in specimen BF-D was observed at 820,000 cycles. A plot of maximum slip measured at the two interior slip gages (see Fig. 5) vs cycles of loading from start to completion of the test is shown in Figure 13. This maximum slip was measured in a static test with a load of 12 kips per jack on the beam, which was the maximum static load applied in all of the static tests on beams of this series. The failure point does not correspond closely with any definite change in the slope of the two

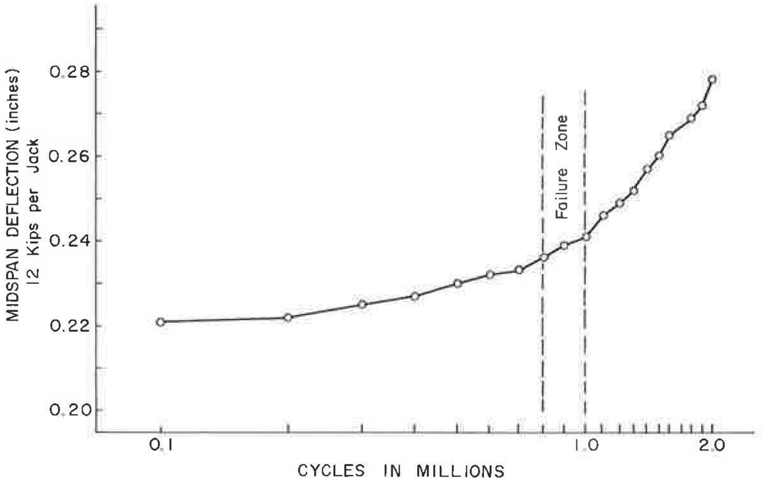



Figure 14. Midspan deflection vs cycles for BF-D.

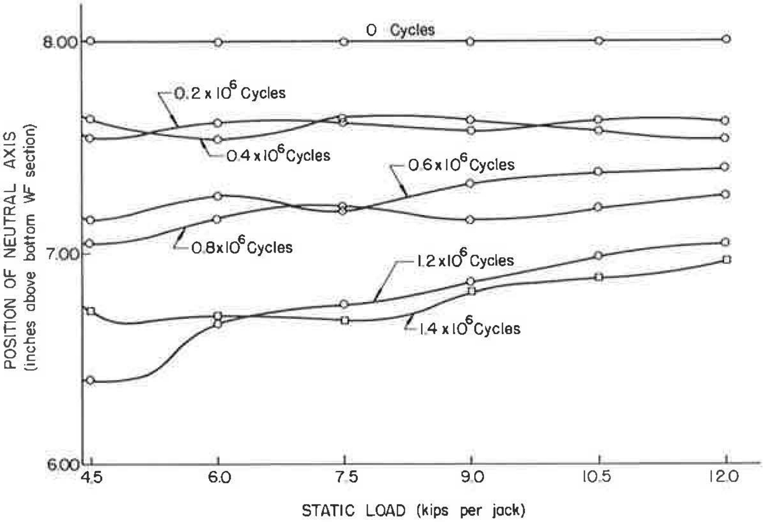



Figure 15. Movement of neutral axis under static load as fatigue failure progresses for BF-D.

curves. Figure 13 shows that the slip increased gradually as cycling progressed before failure of connectors as well as after failure. The magnitude of slip obtained in a static test was dependent on the length of time which the beam was allowed to rest before making the static test. For this reason the point-to-point curves are not smooth. This is also one of the reasons why the slip data were not considered to be very conclusive in determining failure of connectors.

A plot of the position of the neutral axis vs applied load at various numbers of cycles for specimen BF-D is given in Figure 15. These curves indicate a loss of interaction throughout the test, but a gradual shift of the neutral axis. Even when a curve was plotted for each static test, the point of failure of connectors could not be determined with any degree of certainty. There was a large shift in the neutral axis between zero cycles and the next static test because of failure of bond, and this was typical for all specimens.

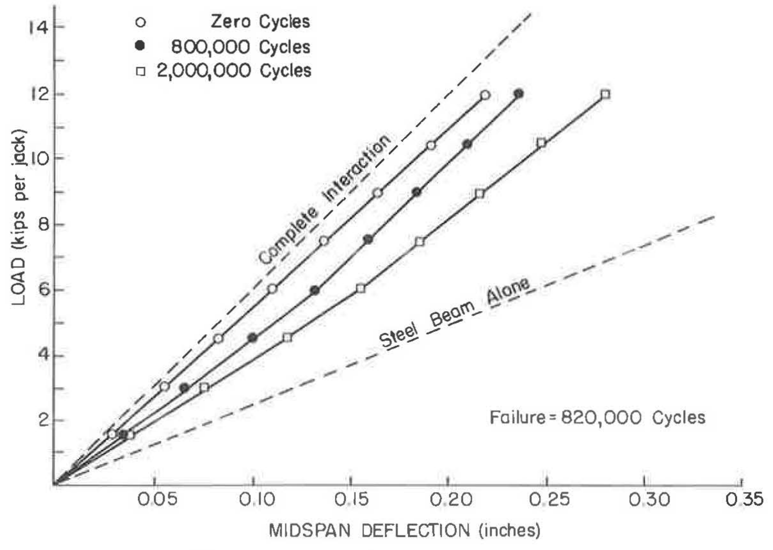



Figure 16. Load vs deflection curves for BF-D.

The static deflection at a load of 12 kips per jack obtained in each static test is plotted in Figure 14 vs the number of cycles of loading plotted on a log scale. There is no distinct change in the slope of this curve in the vicinity of 820,000 cycles. It would not have been possible to determine when failure occurred from this curve alone.

The load vs deflection curves of specimen BF-D at the start of the test, at 800,000 cycles, and at the end of the test are shown in Figure 16 along with the theoretical curves for a composite beam with complete interaction and the steel beam alone. The departure of the initial curve from the theoretical curve for complete interaction is due to the fact that these members were designed with a weak shear connection to insure that failure would take place in the shear connection rather than in the bottom flange of the steel beam. The number of shear connectors was about 57.5 percent of that required to develop the static ultimate strength of the member. The plotting of load vs deflection curves at intervals of 100,000 cycles was not useful in pinpointing the initial failure of connectors because of the gradual loss of interaction throughout the test.

After two million cycles of loading the test was stopped. As can be seen in Figure 16 the final load vs deflection curve for specimen BF-D is situated about midway between the two theoretical limits of complete interaction and bare steel beam. The concrete slab was removed and it was discovered that six of the eight shear connectors per shear span were fractured at one end and seven were fractured at the opposite end of the member. Some of these were actually broken when the slab was removed, but only about 10 percent of the cross-sectional area of these studs remained uncracked after fatigue loading. It is interesting to note that the fracture of about 80 percent of the total shear connector cross-sectional area in the shear spans resulted in approximately an 18 percent increase in deflection as compared with the original curve, whereas complete loss of interaction would result in an increase of 105 percent in deflection.

One of the reasons why it may have been difficult to determine when a connector failed was the fact that shear connectors between the load points carried some of the horizontal shear forces after the end connectors failed. Presumably, as connectors failed, connectors near midspan carried more and more of the horizontal force. This explains why BF-D performed somewhat like a composite beam after 2 million cycles even though only about 20 percent of the shear connector area in the shear spans remained effective.

In specimens BF-A, BF-B, and BF-C, the concrete actually cracked across the full width of the slab in each shear span near the load points. These members then performed as a member with a composite section between load points and a bare steel beam in the shear spans.

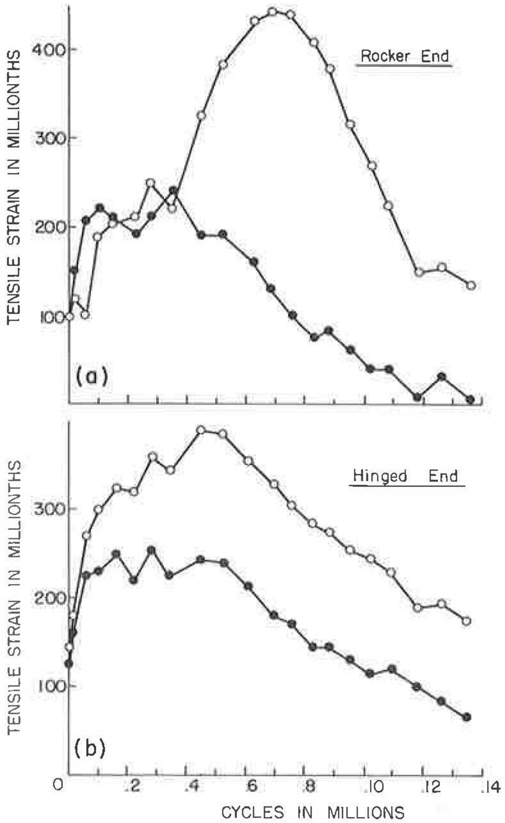



Figure 17. Strain readings of distortion gages at maximum load vs cycles for end pairs of connectors of BF-7.

The primary beam tests were planned to avoid the major difficulties encountered in the preliminary tests. The primary test specimens were designed with a knowledge of stress which would cause fatigue failure of connectors. It was also known that very little difference in stress range would be required to obtain a failure at 1 million cycles as compared to that required to produce a failure in 100,000 cycles from the S-N curve plotted using the preliminary test results.

The primary beams were designed with no connectors between load points so that there would be no difficulty in determining the stress on connectors at any time during the tests. It was felt that designing the members without connectors between load points would reduce scatter in the data obtained from the tests.

It was concluded from the preliminary tests that a better means of determining connector failure than the measurement of strains at midspan, slip, and deflection must be found to obtain suitable test results in the primary beam tests. The magnitude of slip measured in the tests led to the conclusion that the shear force transmitted by the connector must cause considerable bending stresses in the top flange of the steel beam. This notion led to the method of determining when failure of connectors took place described earlier and in the following section.

## Instrumentation for Determination of Connector Failure

The tests of the primary beams were started before a method of determining connector failure was perfected. Therefore, beam BF-1 was used as an experimental beam, and strain gages were placed at various points on the bottom of the top flange in an effort to measure the effect of the horizontal forces transmitted to the top flange of the steel beam by the shear connectors. Strain gages directly under connectors and gages on the cross-section on each side of a connector were used as shown in Figure 10. The latter method was not successful because of the small difference in strain between the two cross-sections.

The strain gages placed directly under the connector produced very satisfactory results. Usually the gages were placed on the side of the connector nearest the end of the beam to record tensile strains. If the gages were mounted on the opposite side, compressive strains were recorded.

During a test, readings on these strain gages were taken at each load increment of a static test. As the test proceeded, the strain readings at the maximum static load was plotted as ordinate and the number of cycles of loading was plotted as abscissa. This was done for several connectors at each end of the member. Typical curves obtained from these readings are shown in Figure 17. Curves of strain at the maximum applied static load of 13 kips per jack vs number of cycles for the pair of connectors on each end of BF-7 are shown.

The curves for the end connectors were chosen for the purpose of illustration because the strain in the top flange being measured is due almost solely to the flange dis-

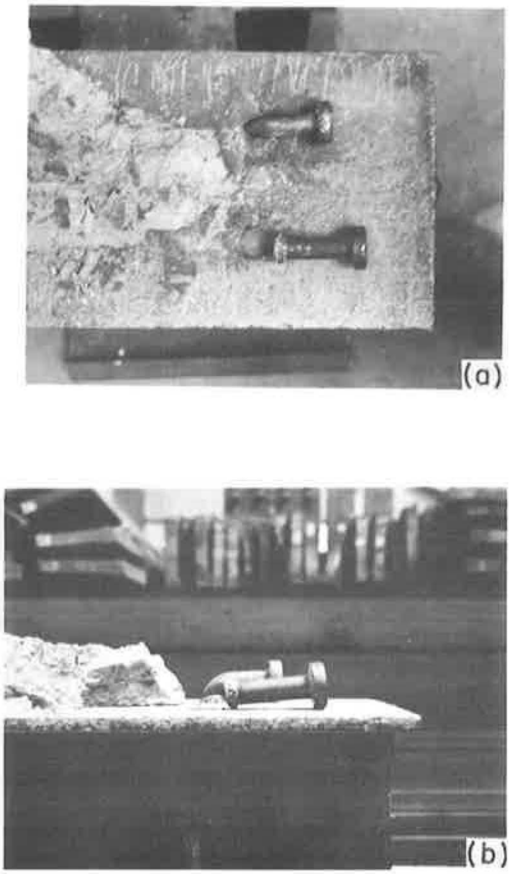


Figure 18. Stud fatigue failure adjacent to uncracked connector.

tortion since the strain gage is actually located between the end of the beam and the support. For studs located nearer mid-span, the strain in the top flange will be equal to the strain corresponding to the compressive stress in the top flange due to moment at that point plus the strain due to the flange distortion. In this case, it is necessary to subtract the compressive strain due to bending from the strain readings.

## Interpretation of Local Distortion Strains

Qualitatively the curves of local distortion strains vs number of cycles are not difficult to interpret. The strains begin to decrease after a fatigue crack begins to propagate, and when the strain reading decreases to zero or to the strain due to bending moment at that cross-section the connector has failed completely. Of the four curves shown in Figure 17, only one actually decreased to zero before the test was stopped. Most of the connectors continued to transmit a small amount of horizontal shear even though they were completely fractured because the fracture took place in the base metal and a mechanical connection capable of transmitting some horizontal load existed after failure of the connector.

Inspection of beams by removal of the concrete slab after the test checked the validity of this interpretation. Inspection also revealed that the strain reading on the downward portion of the curve divided by the maximum recorded strain was approximately equal to the proportion of uncracked shear connector area. It was possible to determine the percentage of shear connector area which remained uncracked in a shear span if the connectors had distortion gages by making a static test.

One occurrence particularly proved the validity of the distortion readings. The strain readings under the end studs of specimen BF-3 indicated that one connector had completely failed and the other stud of the pair was still 100 percent effective. The concrete was broken away to check this result because it did not seem possible. However, inspection verified the findings from the distortion strain readings as shown in Figure 18. One stud was completely fractured and the other stud could be completely bent over with a hammer without cracking the stud. This example, along with the other inspections of beams after testing, caused complete confidence in the use of the distortion gages for the prediction of shear connector failures.

The curves of Figure 17 were typical of most of those obtained. The strain readings increased to a maximum before decreasing. The curves in Figure 17b are typical for connectors which began to fail very early in the test, but most strains increased considerably above the initial reading before decreasing. It was observed that the increase of strain above the initial reading was less if the concrete in the slab was older. This increase in strain would seem to be due to inelastic deformation of the concrete around connectors so that the horizontal force was applied to the top flange primarily as a shear force initially but with more and more bending action as the test proceeded. The magnitude of distortion strain often decreased on these gages if the specimen was allowed

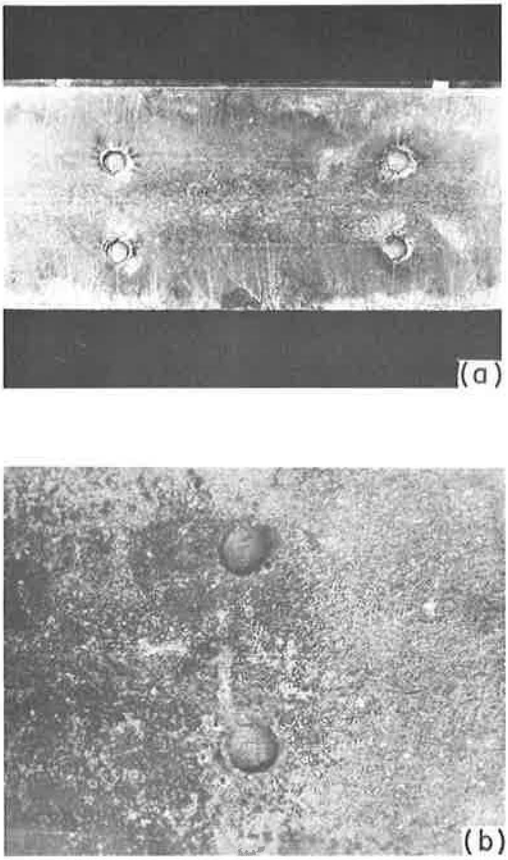


Figure 19. Typical stud fatigue failures in heat-affected zone of base metal.

TABLE 5

NUMBER OF CONNECTORS FRACTURED AT END OF TEST

| Specimen | Number of Connectors completely fractured at expansion end | Number of Connectors completely fractured at fixed end |
|---|---|---|
| BF-A | 7 | 7 |
| BF-B | 6 | 6 |
| BF-C | 9 | 8 |
| BF-D | 7 | 6 |
| BF-1 | 14 | 14 |
| BF-2 | 12 | 0 |
| BF-3 | 1 | 0 |
| BF-4 | 0 | 0 |
| BF-5 | 14 | 20 |
| BF-6 | 16 | 16 |
| BF-7 | 2 | 12 |
| BF-8 | 8 | 10 |

to rest. This suggests that the creep properties of the concrete and the method of conducting the tests may be important factors in the fatigue strength of connectors.

Typical Study Failures

The distortion strain readings reached a magnitude of 700 $\mu$in./in. tension in some members. Usually the maximum compressive strains were somewhat smaller. This corresponds to a stress of nearly 21 ksi on the bottom of the top flange. Presumably a similar stress exists on the top surface. This magnitude of stress in the base metal, combined with a shear stress of similar magnitude on the stud, may help to explain why a typical stud failure was a failure along the heat-affected zone in the base metal. Typical failures are shown in Figures 18 and 19.

In all cases checked, fatigue failure was initiated at the side of the connector toward the end of the beam, which would seem to be the wrong side for a failure to start. This mode of failure is the main reason why the position of the distortion gage shown in Figure 10 was found to be the most sensitive location for detecting failure of the connector.

Some connectors were found to have fatigue cracks on both sides and sound portions near the center. In these cases, the crack toward the end of the beam was equal to or larger than the other one. An explanation of why the fatigue fracture begins apparently on the wrong side of the connector was not developed in this program; however, shrinkage of the concrete slab which initially stresses the connectors in the opposite direction to the applied load could be the cause. As a result of shrinkage, the flange stress range on the outside face of the stud could be tension so that the distortion caused by loads resulted in fluctuating tensile rather than compressive stresses.

Of the 141 studs which failed in fatigue in the primary beam tests, all except two failed as shown in Figure 19. The near end of the beam is to the left in Figure 19. In Figure 19a the portion of the shear connectors which failed statically when the slab was removed is visible in the right-hand pair of connectors. Two studs failed in fatigue through the heat-affected zone of the stud above the bead of the weld. It is not known which side of these studs was cracked initially. Failures of the type shown in Figure 19 were also experienced in the preliminary beam tests. However, because some of the welds were observed to be porous along the failure zone, it was felt that the welds might be of inferior quality. Analysis of the steel of the preliminary beams revealed that the chemical composition of the base metal was not ideal for good stud welding. By comparison with the results of the primary tests, the preliminary beam test results seem to be satisfactory as regards both the type of failure and the general test results.

In Table 5, the number of shear connectors completely fractured due to fatigue loading is reported for each shear span of the member. Most of the tests were continued until a substantial number of connectors had failed. Many other connectors were partially cracked. In the case of specimen BF-3, the test was stopped to check instrumentation as soon as a fractured connector was detected. Beam BF-4 was the only member in which none of the connectors failed, and inspection of this beam after removal of the slab did not reveal any fatigue cracks.

## Failure of End Connectors

In both the preliminary and primary beam tests, all connectors which failed first were located in the vicinity of the end of the beam. Usually the end pair of connectors failed first, but occasionally the failure took place first in the second pair of connectors from the end of the beam.

There seem to be several factors inherent in the testing procedure which may be partially responsible for the fact that connectors near the end of the member fail first. In several members, torsional vibration of the specimen occurred due to slight eccentricities in either the specimen or the loading. This difficulty could be corrected so that no visible torsional vibration took place, but end shear connectors may still have been overstressed by the tendency for the member to twist on each cycle of load.

Bond failure took place in the first 5,000 to 10,000 cycles of loading. This failure started at the end of the member and progressed toward midspan. End shear connectors, therefore, were the first to undergo an increase in stress due to bond failure.

Strain measurements made on specimen BF-6 showed that end shear connectors are stressed higher than interior connectors after bond failure was complete as well as during the time that bond failure was taking place. The variation of load per stud along the length of the member was studied by placing strain gages at cross-sections on each side of two pairs of connectors. Connectors located at 24 and 64 in. from one end of the member were chosen because each pair was located a sufficient distance from local stress conditions at the support and load point.

The force transmitted by a pair of these connectors was determined by calculation of the compressive force in the concrete slab on each side of the pair of connectors being considered. The force on a pair of studs was determined as the difference between the magnitude of the compressive force on each side of the connectors. The strain gage readings were used to calculate the compressive force in the slab at each point required, including the cross-section at midspan.

The average force per connector was taken as the compressive force in the slab at midspan divided by the number of connectors in half of the length of the member. A comparison of the results of this investigation of BF-6 is given in Table 6. The average stress on connectors was obtained by calculating the compressive force in the concrete slab at midspan from strain gage readings with the assumption that this force was dis-

TABLE 6

STUD STRESS AT TWO SELECTED POINTS OF BF-6

| Cycles | Stud Stress (ksi) 24 inches from Beam End | Stud Stress (ksi) 64 inches from Beam | Average Stud Stress (ksi) $= \frac{\text{force in slab at } \mathbb{C}}{\text{area of studs}}$ |
|---|---|---|---|
| 9,000 | 21,400 | 15,300 | 17,900 |
| 49,000 | 23,100 | 16,700 | 18,500 |
| 415,000 | 21,500 | 19,300 | 18,600 |
| 667,000 | 19,200 | 22,800 | 18,900 |

tributed uniformly over the area of all connectors in the shear span. In considering the information contained in Table 6, it must be realized that the bond was only partially destroyed at 9,000 cycles and that the first crack in an end stud occurred at 49,000 cycles.

From the comparison of the measured and theoretical stud stress, it will be realized that although the conventional elastic design assumptions may be satisfactory from a design point of view, these assumptions are as much as 25 percent in error for the prediction of the actual stress on studs near the end of a member. It seems worthwhile to consider that this 25 percent error exists with the most elementary loading condition, and that the magnitude of error may be even larger with a more complicated loading condition.

The important fact concerning the results presented in Table 6 is that the difference between the stress on connectors near the end and near the center is not due to friction or bond. Since the results are obtained from strain measurements on the cross-section, any shear transfer due to friction or bond would merely be included in the apparent force per connector. Hence, the total shear force transferred per pair of connectors is higher near the end of the beam. This is also verified by the fact that slip readings are higher near the ends than near midspan on such a member. Higher stresses on end connectors are likewise predicted by the theory of incomplete interaction (4).

## Rate of Loss of Interaction

Another important observation made on the performance of composite beams is the rate at which loss of interaction between concrete slab and steel beam occurs. The first decrease in interaction takes place as a result of bond failure. As cycling continues, slip at the ends of the beam tends to increase.

It is necessary to be cautious in considering the condition of incomplete interaction with regard to fatigue tests. It appears that a time effect exists due to repeated loading, and that a portion of the increase in slip is due to inelastic deformation of concrete around shear connectors. This is undoubtedly the case since rather high bearing stresses exist. Slip and deflection, therefore, increase as cycling continues. However, these increases are not indicative of connector failure or changes in the stresses in the cross-section at midspan. Loss of interaction will be discussed only in terms of changes in the compressive force in the concrete slab at midspan.

It has been found that after bond failure, a composite beam loses interaction at the same rate as the rate of decrease in the total stud area. The measure of the effectiveness of the composite beam in this case is the magnitude of the total compressive force in the concrete slab at midspan. If this total force after some number of cycles is only 90 percent of its value at 0 cycles (when there is almost complete interaction or 100

TABLE 7

AVERAGE STUD SHEARING STRESS FOR BF-6

| Cycles | Total Force in Slab at ℄ (kips) | Effective Number of Studs | Average Shearing Stress (psi) |
|---|---|---|---|
| 49,000 | 72.5 | 20.0 | 18,500 |
| 156,000 | 69.7 | 19.8 | 18,000 |
| 266,000 | 68.9 | 19.4 | 18,100 |
| 370,000 | 68.7 | 19.0 | 17,900 |
| 415,000 | 68.4 | 18.7 | 18,600 |
| 470,000 | 68.1 | 18.3 | 19,000 |
| 564,000* | 65.1 | 17.3 | 19,100 |

*Only the first 564,000 cycles are given because of lack of reliable data beyond this point

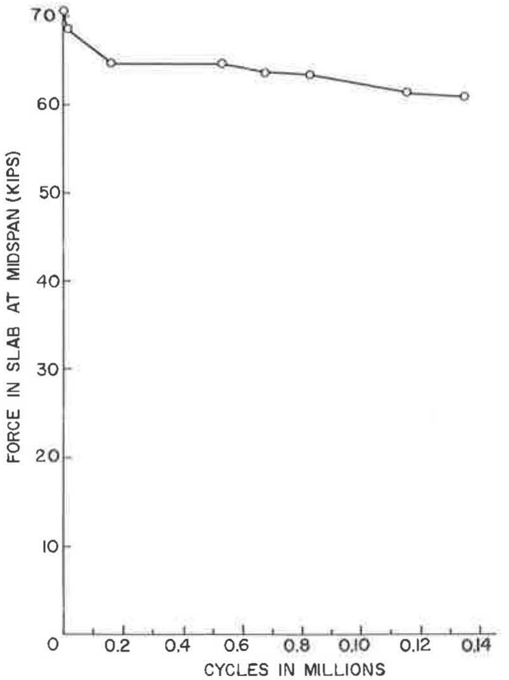



Figure 20. Force in slab at midspan vs cycles for BF-7 at maximum load.

percent effectiveness), then the composite beam is considered to be only 90 percent effective as a composite beam.

It was possible to determine from strain readings, in a manner which has been described, the amount of stud area that remained uncracked at any number of cycles. When 10 percent of the total stud cross-sectional area was gone, leaving 90 percent of the total stud area, the compressive force in the slab was also only 90 percent as large as it was before stud failure. That this is true can be shown in the following manner. For a given composite beam (specimen BF-6 in this case) the total force in the slab at the midspan was calculated from strain readings after the member had been cycled for different lengths of time. At the same number of cycles that this force was calculated, the effective stud area (total stud area minus cracked area) was also calculated from distortion strain readings. If the effectiveness of the slab and the studs decreases at the same rate, then the total force in the slab divided by the effective stud area should remain constant, regardless of the number of cycles. The result of such a calculation is shown in Table 7. The compressive force in the slab at midspan is given for various values of N, and the effective area of studs as determined by strain gage readings is given in terms of the number of studs for corresponding cycles of loading. The average shear stress given in this table was calculated as in Table 6 by dividing the compressive force in the slab at midspan by the unfractured shear connector area in the shear span. The amazing fact is that this average stress is nearly constant throughout the test. The differences which do occur are of smaller magnitude than the probable error in determining the results.

The loss of interaction as determined by measurement of the compressive force in the concrete slab at midspan was found to be directly proportional to the loss of the effective area of shear connectors. Since the stress on the uncracked area of the shear connectors did not increase during the test, the loss of interaction in a member was a gradual process. The rate of decrease of the compressive force in the slab vs cycles of loading is shown in Figure 20 for specimen BF-7. From Figure 17 it appears that the first stud became cracked at about 350,000 cycles and the fourth at about 720,000 cycles. However, at 1,400,000 cycles the force in the slab is still approximately 86 percent of its original magnitude.

## Progressive Failure of Studs

It has been illustrated in Figure 20 that the rate of loss of interaction and, therefore, the rate of stud failure is very gradual. However, with the addition of corrosion effects in the field, the rate of stud failure could be increased. For this reason the determination of the initial failure is significant, and our attention must be focused on initial failure as a design criterion.

Of the eight members tested in the primary beam tests, connectors failed in seven beams. The rate of failure of connectors was gradual. From Table 4 it can be seen that the stress range on specimen BF-7 was about the average value of stress range among the seven beams with connector failure. Thus, the rate of failure of connectors illustrated by Figure 20 is about the average rate for the seven beams.

The tests demonstrated that if one pair of connectors failed in a member, failure of all connectors would eventually result if loading were continued. It has been shown that the average stress on the uncracked connectors remains nearly constant. However, this does not mean that the stress on one particular stud remains constant. In Table 6 the stress on the pair of studs located 64 in. from the end of the beam is shown to increase as failure of end connectors proceeds. The data in Table 6 indicate that the stress on connectors in the shear span becomes more uniform as the loading proceeds. This may be the reason why the reduction in the stiffness of the beam with fatigue loading progresses slowly.

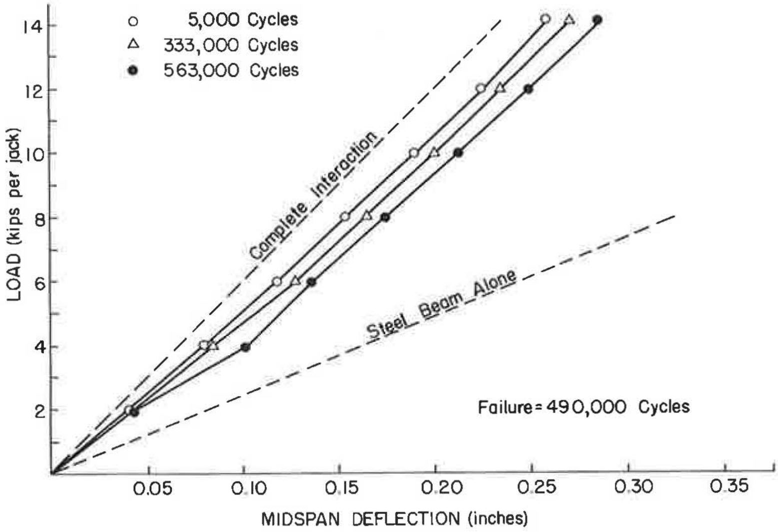



Figure 21. Load vs deflection curves for BF-1.

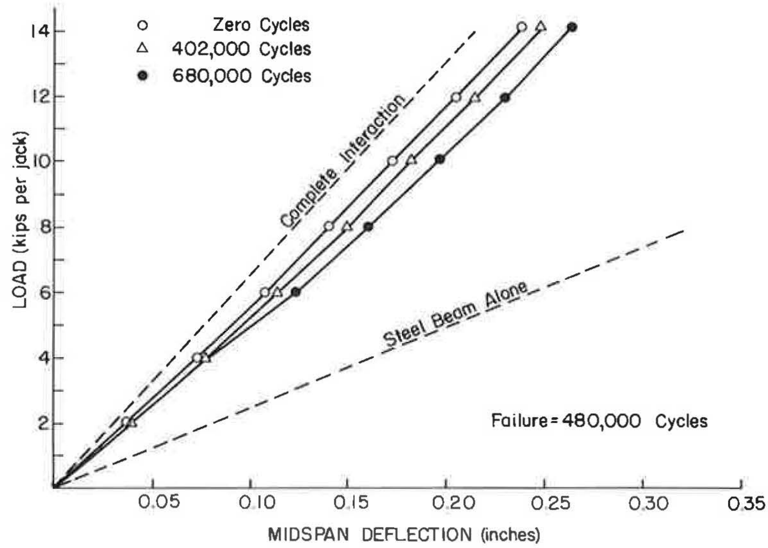



Figure 22. Load vs deflection curves for BF-2.

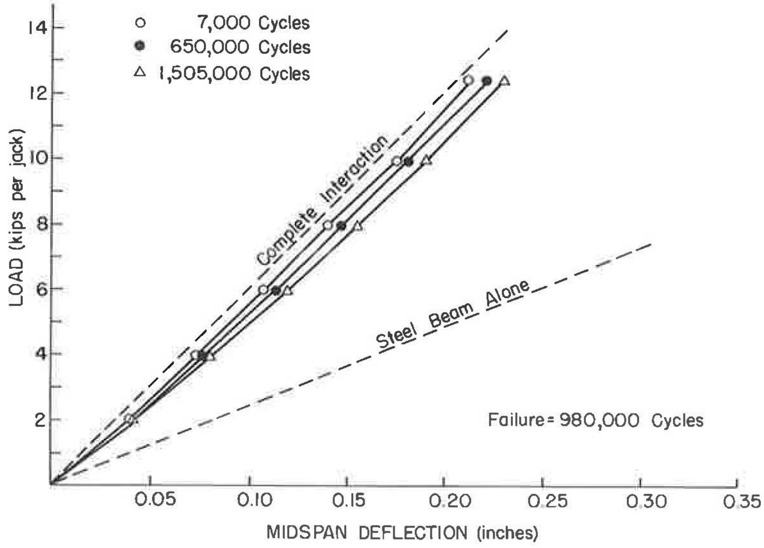



Figure 23. Load vs deflection curves for BF-3.

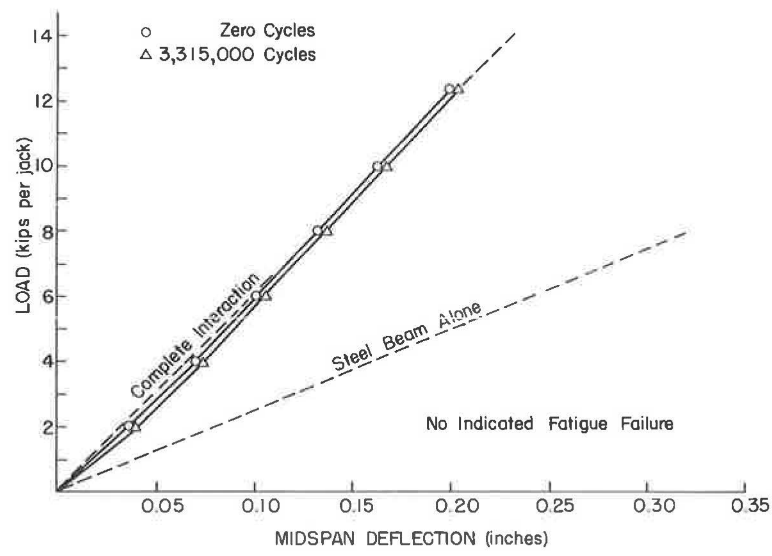



Figure 24. Load vs deflection curves for BF-4.

## Deflection of Members

The deflection of members at maximum cycling load increased from the first cycle to the completion of the test in the primary beam tests in about the same manner as that shown for specimen BF-D in Figure 15. In the early stages of loading, the increase in deflection would seem to be due to bond failure and polishing of the slip plane due to movement of the slab with respect to the steel beam on each cycle. A second stage of deflection increase might be due to inelastic deformation of concrete around connectors. Finally, the increase in deflection becomes due to failure of connectors. It is not possible to separate these stages on a curve such as Figure 15.

A comparison of the load-deflection curves from static tests of the primary beam specimens is of interest. In Figures 21 through 28 are shown load vs deflection curves

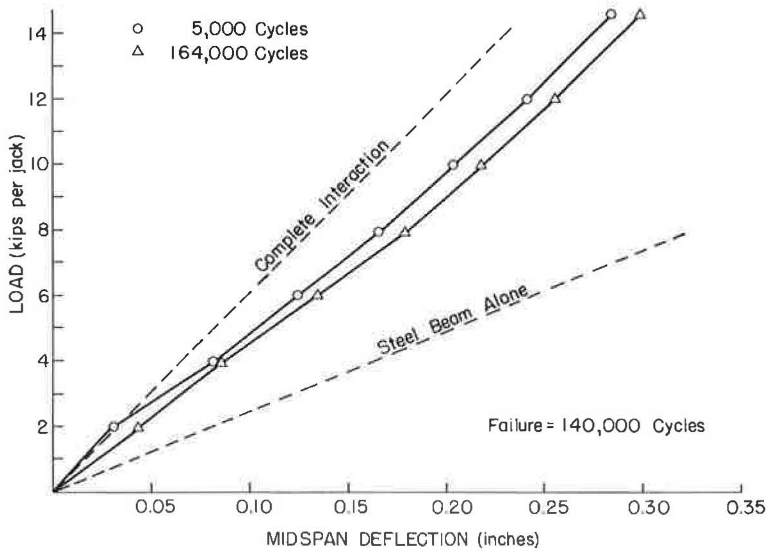



Figure 25. Load vs deflection curves for BF-5.

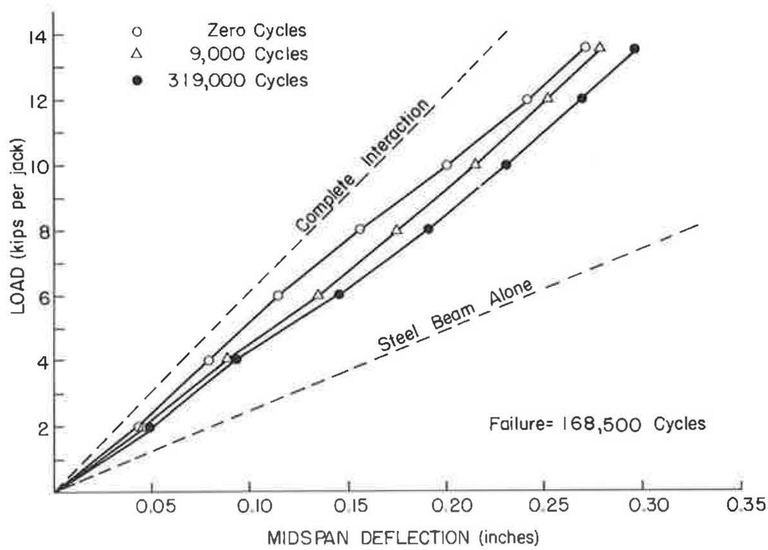



Figure 26. Load vs deflection curves for BF-6.

taken at various numbers of cycles of loading. On each figure two dotted lines are shown for the limits of composite action. The upper dotted line represents complete interaction and the lower dotted line represents the steel beam alone. Deflection due to shear has been taken into account in establishing the upper dotted line. The criterion for establishing the number of cycles to failure shown is discussed in the following section of the report. The curves shown in Figures 21 through 28 represent only a portion of the load vs deflection data taken during the tests.

Comparison of the eight sets of curves in Figures 21 through 28 shows some correlation between deflection data and fatigue failure. The change in deflection curves between the zero cycle curve and a curve before failure is less for members in which connector failure takes place after a larger number of cycles. In Figure 24, for in-

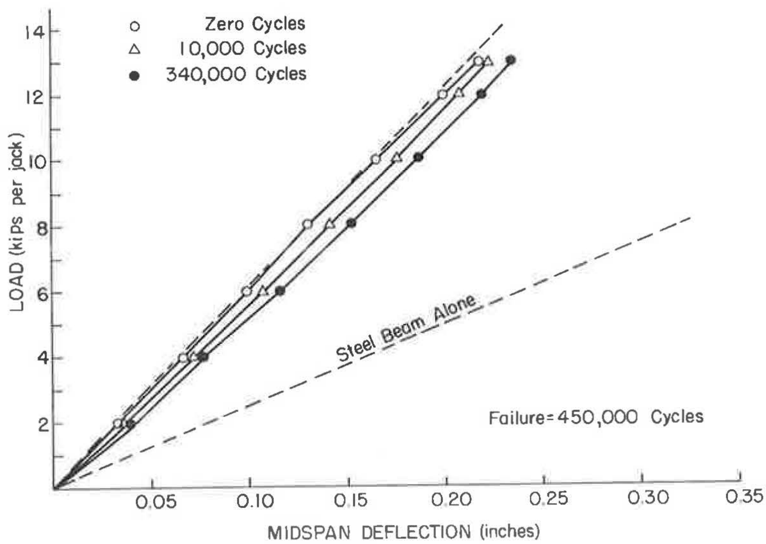



Figure 27. Load vs deflection curves for BF-7.

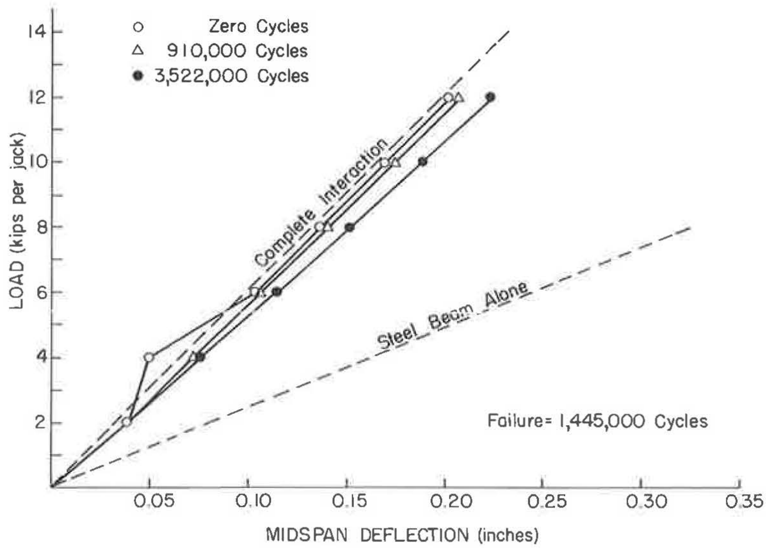



Figure 28. Load vs deflection curves for BF-8.

stance, there was hardly a measurable change in the various load-deflection curves from start to finish of the test, and in this member none of the connectors failed.

The amount which the initial curve departs from the theoretical curve for complete interaction was different for various members. In the case of the two members with the best fatigue endurance, the initial and theoretical load-deflection curves nearly coincide. A study of these curves reveals that the increase in deflection which takes place after failure of connectors is not significant. In several members, the deflection increase with cycles of loading is about the same before failure as it is after failure.

The data obtained from slip readings seem to have a significance equal to that of the load-deflection data. The relationship of connector failure, slip, and deflection is shown for beam BF-6 in Figure 29. Significant changes in the slope of the slip and

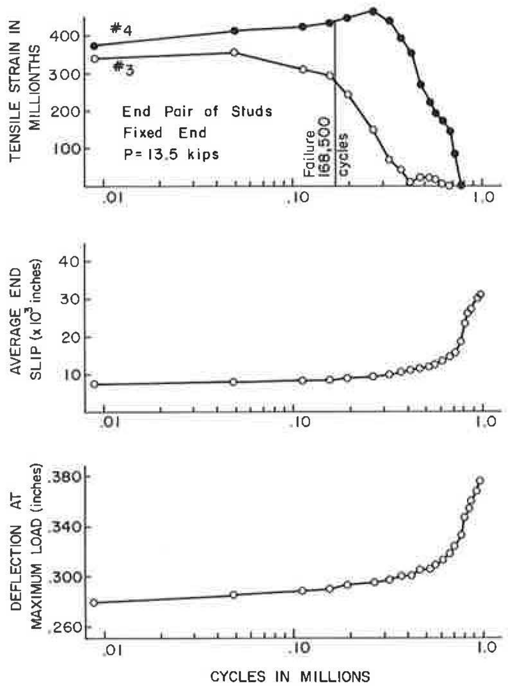



Figure 29. Slip, deflection, and distortion gage readings for BF-6.

deflection curves occurred after the beginning of connector failure, but the changes in slope of these curves were not large and were found to be time dependent. If one had only the slip and deflection curves as evidence it would be impossible to determine initial failure in many of the tests.

## S-N Curve for ½-In. Diameter Stud Connectors

The points on the S-N curve of Figure 12 for the preliminary beam tests and push-out tests had to be plotted using the number of cycles to failure as observed. It is realized from a careful study of the results of the primary beam tests that the method of observation of connector failure in these early tests is not precise. Therefore, the apparently large amount of scatter in the test data may be partly due to the lack of precision in observation. For this reason, the S-N curve of Figure 12 was drawn by considering only the primary beam tests.

Even the use of the distortion gages for the detection of shear connector failure does not completely simplify the plotting of an S-N curve because of the nature of the failure. A decision was necessary on whether the number of cycles to failure should be based on first cracking of a connector, first complete failure of a connector, or some other basis. After study of the data, it was decided that the value of N should be based on the average number of cycles for cracking of the first pair of connectors. This basis is rather arbitrary, but it seemed to provide the best basis for the following reasons:

1. Up to the point of failure, the beam should be capable of developing the static ultimate strength;
2. Complete failure of a connector or pair of connectors was not considered satisfactory because usually many connectors were cracked before the first pair to become cracked finally failed;
3. Cracking of connectors was detected more positively and before any change in slip or deflection; and
4. The cracking of a single connector may not be significant, but the cracking of a pair of connectors seemed to indicate that the member will fail completely in the shear connection if loading continues.

The values of stress plotted in Figure 12 and recorded in Table 4 are those calculated by the elastic formula for horizontal shear stress, as previously stated. The actual average shear stress on connectors was determined at failure by computing the compressive force in the concrete slab at midspan from strain gage readings and dividing this value by the area of shear connectors in half of the beam. These computed values of average stress were found to differ from the value recorded in Table 4 by less than 5 percent in the majority of members. A maximum difference of 10.8 percent was found in member BF-7. Both theoretical and measured values along with the percentage difference are given in Table 8 for all of the primary beam tests. As shown previously, the actual shear stress on an individual connector in these members may exceed the value in Table 4 by more than 10 percent.

The S-N curve of Figure 12 contains all published data on the fatigue strength of ½-in. diameter stud connectors. The authors feel confident that this curve adequately

TABLE 8

COMPARISON OF THEORETICAL AND MEASURED STUD STRESS AT BEGINNING OF TEST

| Specimen | Average Shear Stress $b\,\frac{VQ}{I}$ (psi) | Average Shear Stress from Strain Measurements at Midspan (psi) | Percent Difference |
|---|---|---|---|
| BF-1 | 22,200 | 21,300 | 4.1 |
| BF-2 | 22,200 | 21,000 | 5.0 |
| BF-3 | 19,400 | 19,200 | 1.0 |
| BF-4 | 19,400 | 19,500 | 0.5 |
| BF-5 | 22,600 | 21,700 | 4.0 |
| BF-6 | 21,100 | 20,300 | 3.8 |
| BF-7 | 20,300 | 18,100 | 10.8 |
| BF-8 | 19,500 | 19,100 | 2.1 |

TABLE 9

BOTTOM FLANGE STEEL STRESS

| Specimen | Initial Bottom Flange Stress (psi) | Average Bottom Flange Stress (psi) | Number of Cycles at this Stress |
|---|---|---|---|
| BF-1 | 22,500 | 24,000 | 880,000 |
| BF-2 | 21,800 | 22,800 | 680,000 |
| BF-3 | 19,900 | 19,500 | 1,556,000 |
| BF-4 | 18,400 | 19,500 | 3,315,000 |
| BF-5 | 25,600 | 27,000 | 354,000 |
| BF-6 | 21,000 | 22,800 | 1,009,000 |
| BF-7 | 18,600 | 19,500 | 1,344,000 |
| BF-8 | 19,300 | 19,800 | 3,522,000 |

represents the fatigue strength of ½-in. diameter connectors in beams for design purposes. Some of the variables which affect the fatigue strength of connectors, such as the effect of minimum stress, rate of loading, flange thickness, and concrete strength, have not been thoroughly investigated and should be the subject of future investigations.

From Figure 12, the failure stress would be greater than 15.9 ksi or 3.12 kips per connector for 97.5 percent of the specimens. The AASHO useful capacity of ½-in. diameter studs in 3,000 psi concrete is 4.51 kips per connector. Since fatigue strength and useful capacity are unrelated terms, different values are to be expected. It is interesting to note, however, that the useful capacity is not a conservative approximation of the fatigue strength. Therefore, it is not advisable to modify present shear connector design procedure by merely using a more liberal value of the factor of safety which is used in deriving allowable connector loads from the useful capacity.

The magnitude of the failure stress serves to point out that fatigue failure of connectors is a severe problem. It is important that S-N curves such as Figure 12 be obtained for other sizes of connectors. These results also indicate that the design of connectors should undoubtedly be based on range of stress rather than on maximum stress as in the case of present specifications.

## Bottom Flange Stress

The primary test specimens were designed in such a way that fatigue failure would not take place except in the shear connection. However, the members were also designed so that the bottom flange steel stress would be equal to or greater than 18 ksi throughout all tests. Stresses in the concrete slab were sufficiently low that data on the slab did not provide any worthwhile information regarding fatigue failure. However, bottom flange steel stresses were high enough during some of the tests that the data are worth including in the report.

The magnitude of the bottom flange stress changed during any one test as the compressive force in the concrete slab decreased. For this reason both the initial bottom flange stress and the average bottom flange stress are given in Table 9.

There were no fatigue failures observed in the bottom flange of the test specimens. The most severe fatigue loading condition from the point of view of possible failure of the bottom flange was in beams BF-1 and BF-5. Beam BF-1 endured 880,000 cycles with an average maximum bottom flange stress of 24 ksi, and beam BF-5 endured 354,000 cycles with the average maximum flange stress at 27 ksi.

## CONCLUSIONS

The information obtained from tests of composite beams containing ½-in. diameter stud connectors leads to the following conclusions concerning fatigue failure of connectors and the effect of connector failure on the performance of a composite beam:

1. The average shear stress at which ½-in. stud connectors failed in fatigue at 1,000,000 cycles of loading was 18.2 ksi;
2. Fatigue failure of connectors was progressive in nature and began at connectors near the ends of the member;
3. A composite member can be considered effective long after initial cracking of studs, but complete failure will eventually occur if a pair of connectors becomes cracked;
4. Fatigue failure of ½-in. diameter studs with good welds usually occurred in the base metal of the beam to which they were attached;
5. Measurements indicate that end connectors were stressed approximately 25 percent higher than the average connector stress when connectors are designed elastically; and
6. The occurrence of fatigue cracks in shear connectors can be detected by using electrical resistance strain gages mounted near connectors on the bottom of the top flange.

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