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A BRIEF SUMMARY OF FATIGUE TEST RESULTS ON COMPOSITE BEAMS

by

R. G. Slutter

Fritz Engineering Laboratory
Report No. 285.8

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A BRIEF SUMMARY OF FATIGUE TEST RESULTS ON COMPOSITE BEAMS

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R. G. Slutter

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Fritz Engineering Laboratory
Lehigh University
Bethlehem, Pennsylvania
February 17, 1964

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The report which follows is a brief summary of the results of research work being conducted at Lehigh University as part of a research project entitled "Fatigue of Composite Beams". This project is being sponsored by the American Institute of Steel Construction and Gregory Industries, Inc. Since it will not be possible for a member of the Lehigh University staff to be present at the forthcoming meeting of the Joint ACI-ASCE Committee on Composite Construction, this report is being mailed to members of the committee prior to the meeting.

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During the past year eight composite steel and concrete beams were tested for the purpose of determination of the fatigue strength of 1/2 inch welded stud shear connectors in beams.

The members tested consisted of a concrete slab 4 inches by 48 inches connected to a 12 W 27 steel beam by forty 1/2-inch diameter stud connectors. The concrete mix was designed for 3000 psi and the strength at time of test averaged about 3500 psi. These tee beams were supported on a span of 15'-0". The members were loaded by two hydraulic jacks located 9 inches on each side of midspan, and the load was spread laterally by distributor beams as shown in Fig. 1.

The number and spacing of shear connectors is shown in Fig. 2. Shear connectors were not placed in the center portion of the member so that all connectors would be located in the shear spans. This made it possible to determine with a better degree of certainty the magnitude of the shear stress on each connector.

Because of difficulties in determining when connectors actually failed in previous beam tests, a method of measuring the local bending stresses in the top flange of the steel beam by strain gages was devised. The horizontal force applied to the top flange was found to induce bending stresses in the top flange of appreciable magnitude. By experimentation with the location and type of electrical resistance strain gage to be used it was possible to provide gages, later referred to as distortion gages, which were quite sensitive to changes in the connector force.

By the use of distortion gages it was possible to determine when a connector first became cracked and when it completely failed. It was found that these gages were most effective when placed slightly toward the end of the beam from the connector. One of the reasons for this is that a fatigue crack first started on this side of the connector.

Typical failure of connectors consisted of a crack which formed in the base metal and presumably progressed along the heat affected zone of the weld in the base metal. Complete failure resulted in the removal of a crater of material from the flange of the steel beam. Distortion gage readings revealed that often many connectors were cracked before the first connector failed completely. Generally failure began with the end connectors and progressed toward midspan.

The failure of the shear connectors was taken as the average number of cycles required to start a fatigue failure in any pair of connectors.

The distortion gage readings versus cycles of loading are shown in the top curves of Fig. 3 for the initial failure in specimen BF-6. Based upon observations made during testing, it was concluded that if any pair of connectors began to fail, all connectors would fail eventually if the testing were continued long enough. It was revealed in the tests that the cracking of a single connector did not necessarily mean that complete failure would take place, but in all cases failure of a pair resulted in a progressive failure which would eventually have resulted in failure of all connectors.

The slip and deflection readings are also shown in Fig. 3 for specimen BF-6. It can be seen in this figure that the beginning of fatigue failure does not make any recognizable change in either the deflections or the end slip readings. For this reason the distortion gage readings rather than the deflection and slip data were used for evaluation of connector performance.

From strain gages on the steel section at midspan, it was possible to determine approximately the compressive force in the concrete slab during the testing. It was found that the magnitude of this force was proportional to the percentage of uncracked stud area in the half of the beam where failure proceeded more rapidly.

At regular intervals during the testing, a static test was made on a specimen to obtain the data being presented. In Fig. 4 the applied load versus midspan deflection curves are shown for Beam BF-6. This set of curves is typical of curves obtained from beams in which connectors failed in fatigue. Load-deflection curves prior to failure are shown as solid lines whereas the load-deflection curve after failure is shown as a dashed line. The uppermost and lowermost dashed lines represent the limits of composite action.

Curves corresponding to Fig. 4 for a beam in which no fatigue failure of connectors occurred revealed only a very slight change in deflection for 3,000,000 cycles of loading. This would seem to point toward some correlation between deflection and connector failure. However, in the other seven specimens there seemed to be little relation between the spacing of these curves and the number of cycles prior to connector failure.

All of the valid test results pertaining to the fatigue failure of connectors are shown in Fig. 5 where stress and the number of cycles to failure are plotted on a log scale. The points from the recent beam tests are plotted with the number of cycles to failure determined on the basis described above. Other test results are plotted using the number of cycles to failure reported by the investigator. The S-N curve represents the mean of beam test results obtained by considering only the seven failure points of the recent tests. The results of beam tests appear to be quite consistent compared to pushout results. However, it seems that pushout test data could be used to establish a S-N curve which would be conservative. Shear stress for Fig. 5 was calculated by the following equation

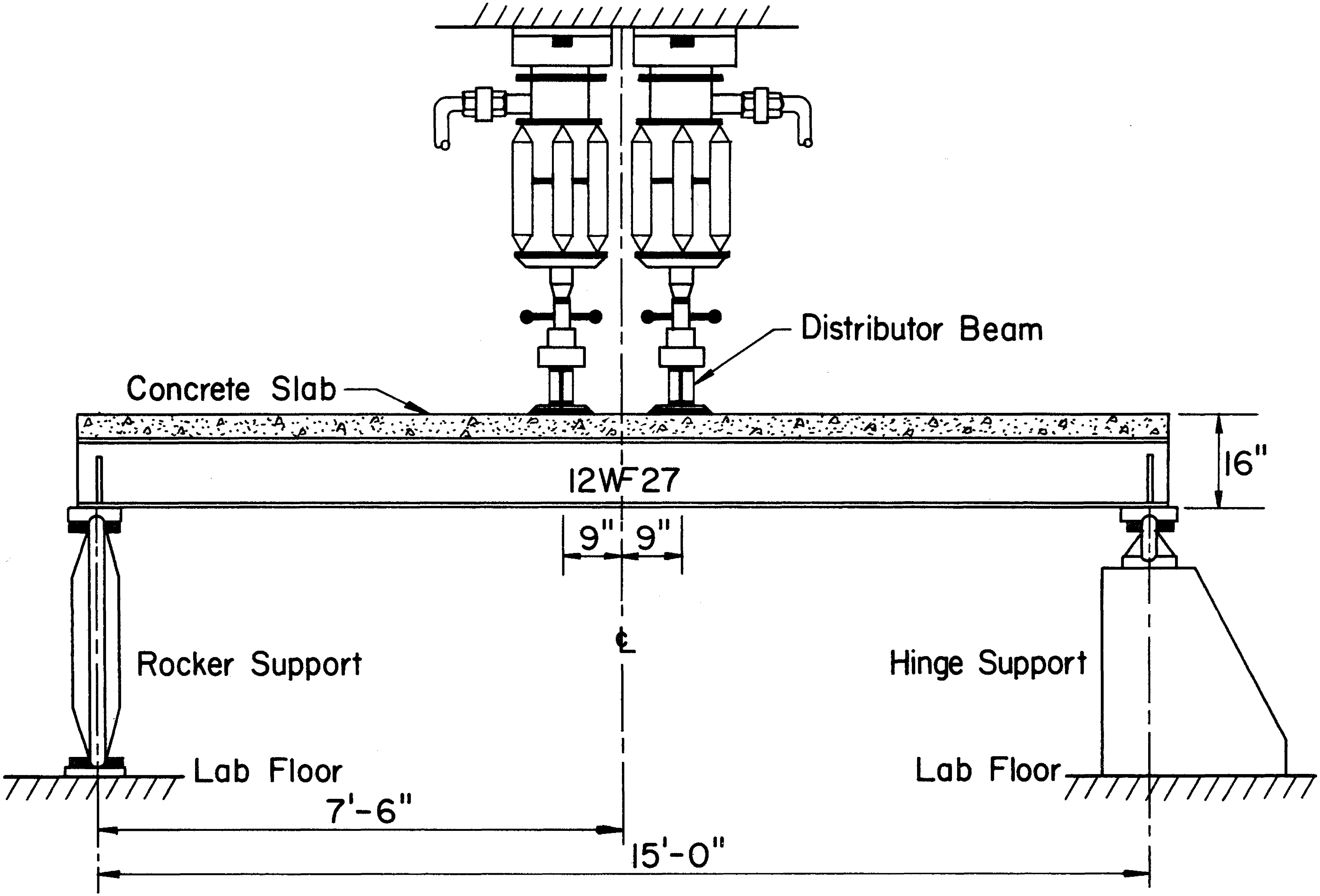
$$S = \frac{V\ Q\ s}{I\ A_s}$$

where V is the applied shear
Q and I are the first and second moments of area
s is the spacing of connectors
A is the cross-section area of the stud

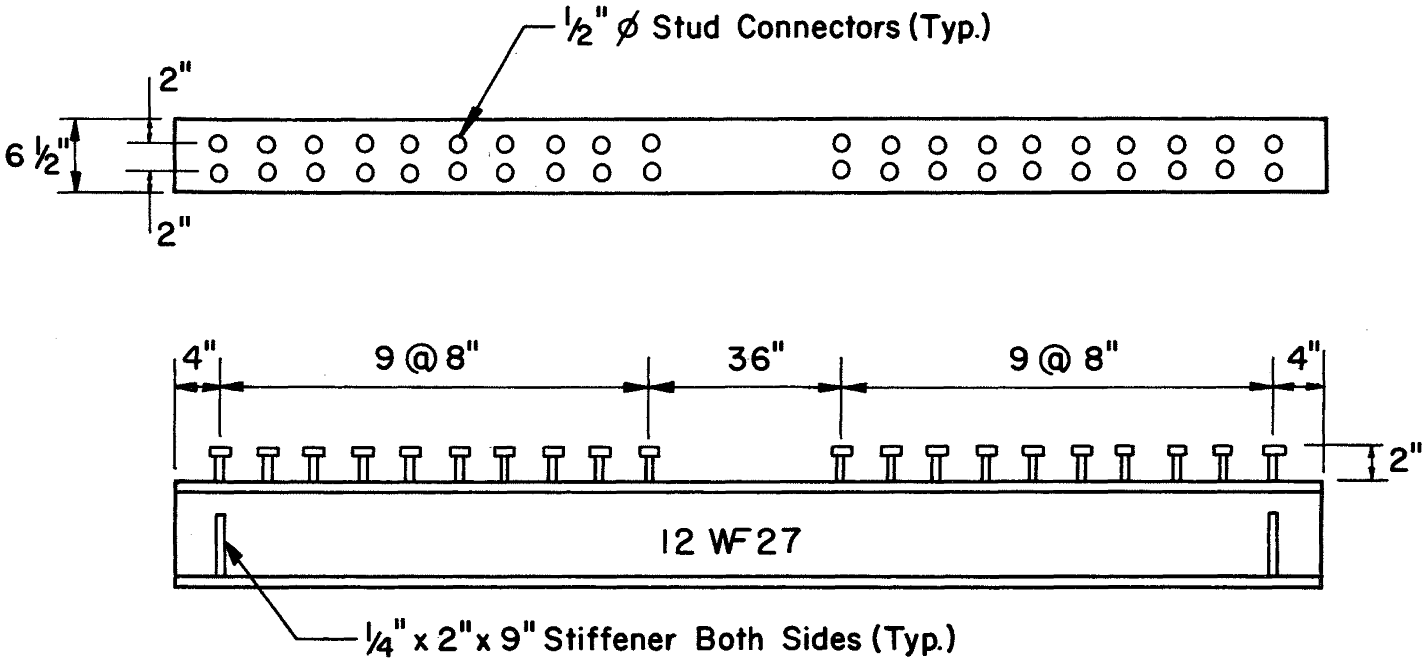
The stress for failure at 1,000,000 cycles of loading on the S-N curve of beam tests is 20.7 ksi or 4.06 kips per stud. The useful capacity for this type of connector in 3,000 psi concrete is 4.52 kips per stud. The useful capacity is therefore not a conservative value for fatigue strength at 1,000,000 cycles.

Since the S-N curve of Fig. 5 is very flat it does not seem that a large factor of safety such as 4.0 used as the upper limit in the AASHO Specifications is justified. Once sufficient data is available for establishing a S-N curve for the type of connectors used in highway bridges, a factor of the order of magnitude of 1.25 could probably be used. More test results on larger diameter connectors than those presented herein are necessary before this approach can be considered.

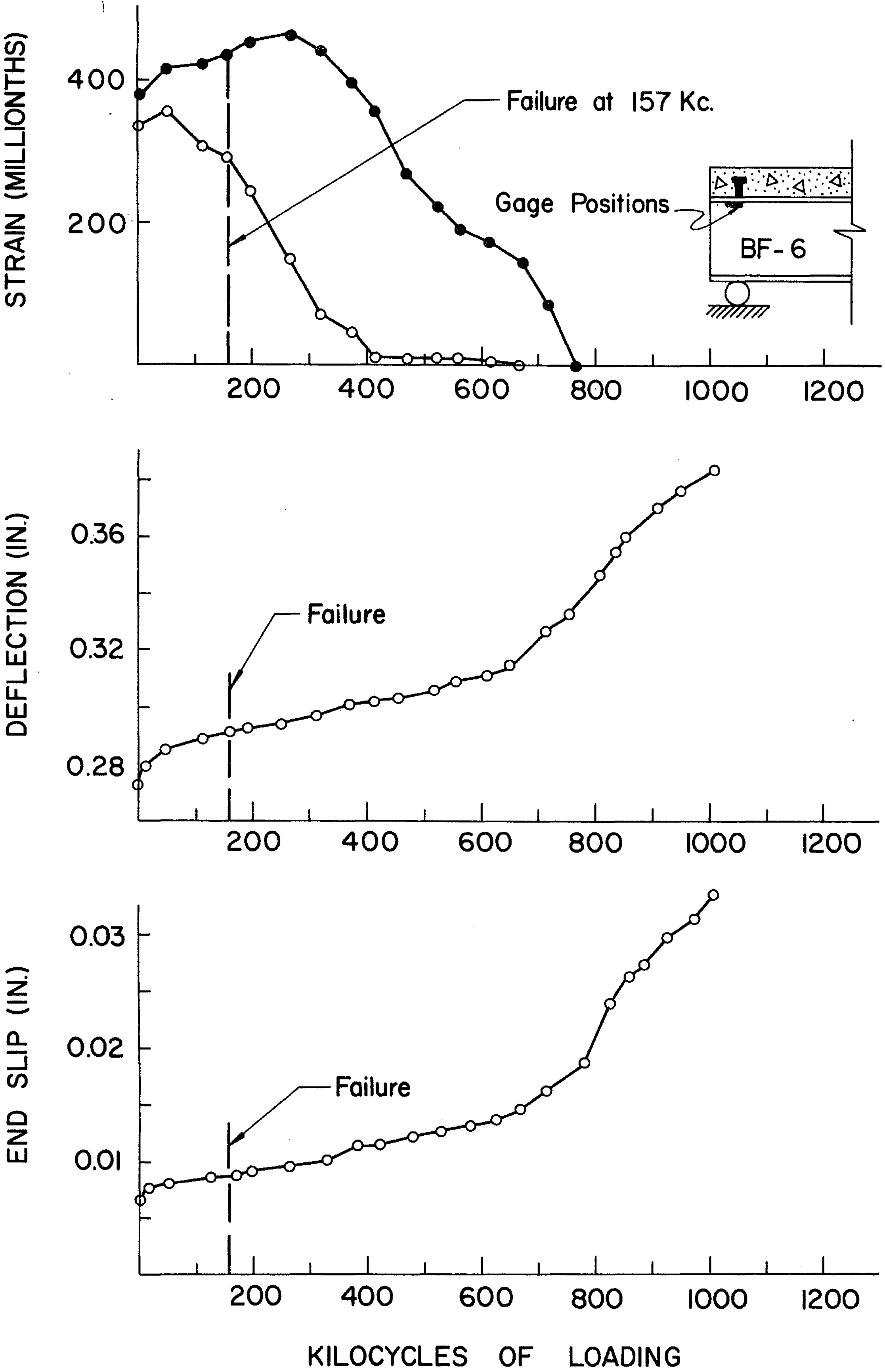
FIGURES



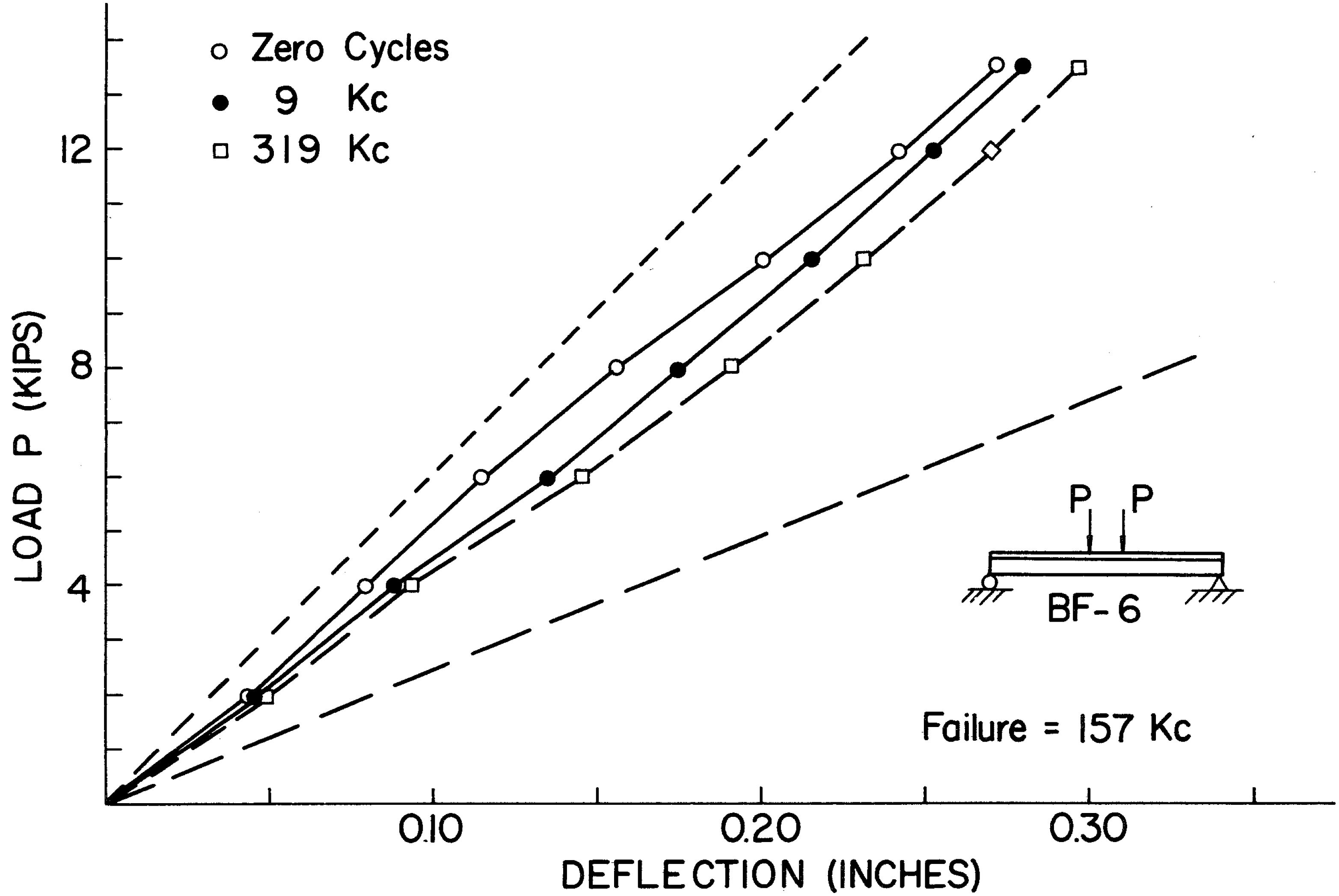
ARRANGEMENT FOR FATIGUE TEST OF COMPOSITE BEAMS



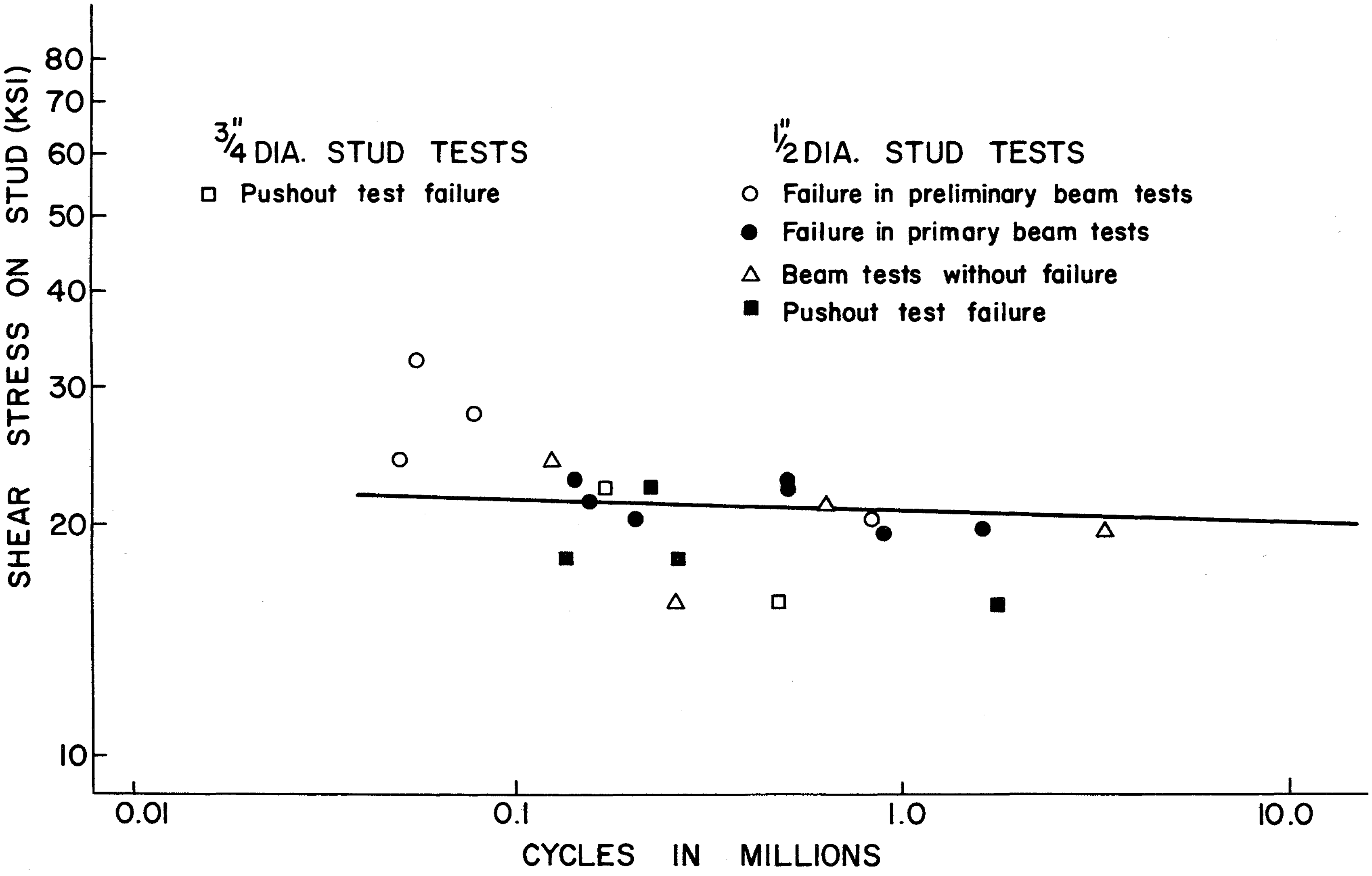
SPACING OF SHEAR CONNECTORS



COMPARISON OF SLIP AND DEFLECTION WITH DISTORTION GAGE READINGS FOR BF-6



LOAD VERSUS DEFLECTION CURVES FOR SPECIMEN BF-6



S-N CURVE FOR FAILURE OF 1/2 INCH DIAMETER SHEAR CONNECTORS IN COMPOSITE BEAMS