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Shear Connector Design For Highway Bridges

THE FATIGUE STRENGTH OF SHEAR CONNECTORS IN STEEL-CONCRETE COMPOSITE BEAMS

by
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A B S T R A C T

An experimental investigation was undertaken at Lehigh University to determine the fatigue strength of mechanical shear connectors for composite steel and concrete beams. The results of this investigation were used to formulate a design procedure for proportioning shear connectors in highway bridge beams so that the resulting shear connection would contain the minimum number of shear connectors consistent with the performance requirement for such members.

Factorial experiments were designed to provide information regarding the effect of minimum stress, stress range, and maximum stress on the fatigue behavior of 3/4 and 7/8 inch diameter studs and 4 inch 5.4 lb. channel connectors. A pushout specimen consisting of one concrete slab attached to a steel section by connectors was developed for the experimental program. The testing program included 43 specimens with 3/4 inch diameter studs, 9 specimens with 7/8 inch diameter studs and 16 specimens with 4 inch 5.4 lb. channel connectors.

The test results were analyzed by several mathematical models to obtain the best fit for the data of each experiment. An experimental relationship between the number of cycles to failure and the stress range on the area of the weld was found to provide the best representation of the data for all types of connectors.

It was also found that there was no significant difference between the curves fitted to the three groups of data, and that a single S-N curve could be satisfactorily fitted to all the data obtained in the investigation using the same mathematical model provided that the stress reversal data were neglected. In all cases the stress reversal condition resulted in improved cycle life for a given stress range as compared to the cycle life obtained at other levels of minimum stress.

A design procedure based on the results of the experimental investigation was developed. The procedure was based on proportioning shear connectors for the range of shear stress on the connector and the fatigue strength of the connector for a given cycle life. The shear connection is further required to be adequate to develop the flexural ultimate strength of the beam.

1. INTRODUCTION

Composite steel and concrete members are extensively used in structures subjected to repeated loading. Because of its relatively large stiffness for superimposed loads, this type of member is used extensively in bridges. Interest in this type of construction seems to have originated from observations of existing bridges which were designed and constructed as non-composite structures but which appeared to have much greater stiffness than the stiffness of the steel beams alone. The first investigations of composite beams by the Truscon Steel Company in the United States in 1921, by the National Physical Laboratories in England also in 1921, and the Dominion Bridge Company of Canada beginning in 1922 were concerned with the increase in stiffness afforded by bond and friction between the concrete slab and steel beams.[(1)]

The idea of using a mechanical shear connector to connect the slab and beam was first investigated in Switzerland in the early 1930's. Other investigations in Europe and the United States quickly followed, but the use of this type of construction lagged far behind the experimental research. Progress in construction was hampered by the lack of design specifications on the one hand and the lack of suitable economical shear connectors on the other. In the United States composite construction first became economically feasible in bridges. An additional impetus for this type of construction was probably provided by the

necessity of preventing the shifting of the concrete slab on steel girders in the construction of modern bridges. The real impetus came with the adoption of the 1944 AASHO Specification.(2)

Finally, the more comprehensive 1957 ASSHO Specification(3) provided for the proportioning of the cross-section and the design of the shear connectors. This document with some revisions is still in use. It is recognized that the current design procedure for the shear connectors is conservative and that additional studies are desirable to lower the construction costs.

1.1 BACKGROUND OF SPECIFICATIONS

Composite beams subjected to repeated loading below their ultimate strength can fail by (1) crushing of the concrete slab in compression, (2) fracture in the tension flange, or (3) fracture of the shear connectors. A weak shear connection could lead to either of the latter two types of failure. The failure of the shear connection would lead first to excessive deflections as the stiffness of the composite member decreased from that of a composite section to the stiffness of the slab and beam acting separately. Depending on the geometry of the cross-section, steel stresses might be raised high enough to produce fatigue failure of the bottom flange.

Because of the serious consequences of a shear connection failure due to repeated loading, the 1957 AASHO Specifications were very conservative with respect to both design of the shear connection and the allowable stresses permitted for the tension flange of the steel section.

Building specifications and codes permitted higher flexural stresses in the steel section of composite beams than were permitted in non-composite beam sections because it was recognized that the ultimate strength of the composite section is significantly greater than the plastic moment of the non-composite section. Because the performance of the composite beam under repeated applications of load was considered critical, higher stresses were not allowed in bridge specifications.

The formulas given in the 1957 AASHO Specification resulted from the criteria that the shear connectors should not be allowed to yield for any moment less than the ultimate strength of the member.[4] Extensive investigations at the University of Illinois[5,6] involving both static and fatigue behavior indicated that this approach would insure that the loss of interaction between slab and beam would be small for loads up to ultimate load and that fatigue failure of connectors would not occur.

Although fatigue tests were conducted to investigate the problem, the design procedure prescribed by the specification was based on static loading conditions. The fatigue studies had shown the fatigue strength to be adequate when the static loading criteria were used for design. This current (1966) procedure consists of calculating the maximum horizontal shear stress in kips per inch along the beam by elastic theory,

$$S = \frac{V\,Q}{I} \tag{1.1}$$

where V is the total dead load plus live load shear force on the composite section, Q is the statical moment of the transformed area of the concrete

slab, S is the horizontal shear in kips per inch, and I is the moment of inertia of the composite cross-section. The shear connector spacing is then determined by dividing the allowable load per shear connector by S. This allowable load is obtained by calculating the useful capacity of a connector (the load at which it will yield) and applying a suitable factor of safety which is given by formulas in the specification.

This current design procedure is undesirable for the following reasons: (1) Bridge engineers have recognized that it is conservative as a result of experience and available test data. (2) Since the current design procedure is based primarily on static considerations, it is not possible to ascertain what effect an arbitrary reduction in the factor of safety will have on the performance of the connector. (3) Connectors near the midpoint of positive moment regions are subjected to a range of shear nearly twice the maximum static shear.

Shear connector design based upon static loading has led to other practices and notions which are not compatible with actual conditions of stress resulting from moving loads on bridges. It is common practice to omit shear connectors from the negative moment regions of continuous beams, and this has also been suggested for the end portions of simple span beams where the steel beams are able to carry the bending and shear stresses without the aid of the concrete slab.

The improper use of shear connectors may result in (1) fatigue failure of connectors, (2) excessive cracking in the concrete slab, (3) complete separation of slab and beam due to differential temperatures, and (4) excessive local bending stresses in the concrete slab.

Although the consequences of improper design of the shear connection are not likely to be disasterous, the increased maintenance and repair costs associated with these problems may become considerable in view of the number of structures in service.

The AASHO Specifications form the basis of design in the western hemisphere and to some extent have been used abroad. European codes have developed from work done mainly in Germany and Switzerland. These codes were generally developed on the basis of a more rigid type of shear connector rather than the types commonly used in the United States. The result is a more precise analysis of horizontal shear forces and a more complicated design procedure.

The problem of possible fatigue failure of connectors is reflected in European codes in the relatively conservative values for shear connector loads. Other provisions deal with the secondary forces such as those resulting from shrinkage of the concrete. Such forces are not considered in the AASHO Specification. Generally, it can be stated that the more flexible the shear connection the less the need for an exact analysis of forces and stresses. This helps to explain some of the differences between the two approaches as represented, for example by the AASHO and German Bridge Specification.

1.2 OBJECT AND SCOPE

The object of this investigation is (1) to develop a rational basis for the design of connectors, (2) to provide sufficient supporting data on the fatigue strength of connectors, and (3) to develop a design

procedure which is applicable to the design of both simple span and continuous members.

This investigation was preceded by two pilot studies employing fatigue testing of composite beams with welded stud connectors to study the behavior of beams and connectors under repeated loading. The first of these investigations, conducted at Lehigh University,[7] included fatigue tests of 12 simple span members containing 1/2 inch diameter stud connectors. The second investigation, at the University of Texas,[8] included 7 full-scale beam tests with 3/4 inch diameter connectors. These two investigations showed that the fatigue strength is a function of the connector load.

For this investigation a statistical experiment was developed to aid in the assessment of the significance of the major test variables. Three common types of shear connectors were included: 4 inch 5.4 lb. channels, 3/4 inch studs, and 7/8 inch studs.

Forty-three specimens containing 3/4 inch studs, 16 specimens containing channel connectors, and 9 specimens containing 7/8 inch studs were tested. The results of these tests and all other data were analyzed to obtain suitable guide lines for a more liberal design of connectors for various cycle life requirements.

A design procedure for shear connectors was developed based on the fatigue and ultimate strength requirements. By considering both, the design procedure is suitable for all conditions of continuity, method of construction, and dead load to live load ratios.

1.3 PREVIOUS RESEARCH

In the early tests of beams where bond and friction alone provided the shear connection, tests were run to find if vibratory loading would destroy bond. The earliest tests of this type were conducted by MacKay, Gillespie, and Leluan in 1923.[9] In these tests the steel beam was encased in concrete, and these and later tests resulted in the establishing of a tentative value for allowable bond stress. These tests are of little interest today because encased beams are seldom used in bridge construction.

A formula for the strength of spiral shear connectors was developed by Voellmy of the Swiss Federal Institute for Testing Materials in 1935.[10] This formula was based on the maximum stress in the spiral bar caused by a combination of normal, shearing, and bending stresses. The formula seemed to be conservative when compared with the results of four repeated load tests, but spirals came into use in both Europe and the United States without further testing in fatigue although many static tests were made.

Fatigue tests of pushout specimens and beams with rigid connectors consisting of flexurally stiff rolled sections and hooks were made in Switzerland by Ros.[11] Weld failures were observed in three beams and four pushout specimens, and it was noted that good welding was quite important. A design method was recommended based on limiting the bearing stress on the concrete to 55 percent of the prism strength. Since the fatigue failures were associated with higher bearing stress, the problem of fatigue was dismissed, but very good welds and high quality concrete were considered essential where fatigue loading can occur.

Fatigue tests of flexible connectors were first conducted at the University of Illinois.[5] This investigation also included static tests of three beams and 64 pushout specimens. Eighty-five fatigue tests were conducted on 3-foot-span model beams containing small channel connectors. The variables studied included web thickness, flange thickness, mortar strength, and the orientation of connectors. The criterion of failure was a slip of 0.01 inch between slab and beam at the support.

The conclusions reached in this investigation are of interest because most were later verified by tests of full-size composite beams.[7,8] The following summarizing statements are taken from Sect. 42 of Reference 5.

> (1) The fatigue failure of the shear connection of a composite Tee-beam is progressive in character. The end connectors fail first and the damage progresses toward the load.
>
> (2) A large and rapid decrease in the degree of interaction takes place only after several connectors have been broken. This decrease in interaction is accompanied by a decrease in the load-carrying capacity of the beam.
>
> (3) In a channel shear connector with the flanges stiffer than the web, the maximum steel strains occur at the fillet of the flange welded to the beam and the fracture occurs on a section extending from the fillet to the edge of the weld on the back face of the channel.
>
> (4) An increase in the thickness of the channel web produced an increase in the fatigue strength of the composite beams. However, the increase in fatigue strength was very much less than the increase in section modulus of the channel web.
>
> (5) The fatigue strength of composite beams with channel shear connectors was increased by an increase in the compressive strength of the mortar slab. The effect of the strength of mortar was smaller for high strengths than for low strengths.
>
> (6) The results of the repeated-load tests confirmed the concept of a dowel-like action of channel shear connectors. An exception was the anomalous behavior of the beams having connectors with various flange thicknesses.

(7) In spite of the fact that an attempt was made to destroy the natural bond between the mortar slab and the beam in all specimens, there were several beams in which the bond was effective throughout the initial static tests and during part or even all of the repeated-load tests. The fatigue life of these beams was increased appreciably over that of the beams without bond and in some of these beams there was no failure of the shear connectors whatsoever.

The first of these conclusions describes the type of failure usually experienced in all fatigue tests of composite beams with flexible connectors. Although it has been shown theoretically that the interior connectors may be stressed more highly than end connectors, the end connectors are always critical.

The second conclusion points out that the initial connector failure in a beam is difficult to detect, but it is this initial connector failure which is necessarily the basis for design. As indicated in Ref. 5, the failure of the first connector marks the beginning of eventual beam failure.

The third and fourth conclusions confirm that it is economical to use the lightest standard channel as a shear connector. Little is gained by increasing the thickness of flange or web. The fifth and sixth conclusions are discussed in detail in Chapter 4. The experiences related in the seventh conclusion have also been encountered in other investigations.

A fatigue test of a full-scale double tee-beam was conducted at Lehigh University and reported by Thurlimann.[12] This test demonstrated that fatigue failure of stud shear connectors is not a problem when connectors are designed by the 1957 AASHO Specifications. In the test

it was shown that 1,000,000 cycles of design stress followed by 290,000 cycles of 125 percent of design stress and 250,000 cycles of 150 percent of design stress neither produced fatigue failure nor reduced the ultimate strength of the member.

Later fatigue tests of pushout specimens by Thurlimann(13) resulted in recommended design stresses for 1/2 inch diameter connectors of 2700 lbs. or a unit stress of 13,800 psi for loading of approximately zero to maximum. This value was based on a cycle life of 600,000 cycles and a factor of safety against failure of 1.25. The test of a single beam by Culver and Coston(14) containing 1/2 inch diameter connectors indicated that the above values are conservative. In this test 1,022,900 cycles of load producing stresses which varied from 1.5 to 15.0 ksi were followed by 505,400 cycles varying from 1.5 to 18.1 ksi, followed by 619,900 cycles varying from 1.5 to 21.2 ksi, and finally 122,400 cycles varying from 1.5 to 24.1 ksi. This sequence of loading did not produce connector failure, but relatively large end slips were observed with the higher loadings. However, the performance of the beam remained essentially linear in spite of relatively large slips. This test indicated that a limitation on slip is unnecessary if fatigue failure does not occur in a member.

The fatigue strength of bare stud shear connectors under stress reversal was ascertained by Sinclair(15) prior to the present investigation. In this investigation 3/4 inch studs welded to a plate 4-1/2 in. x 7-3/4 in. x 3/4 in. were loaded by a force parallel to the plate at a point 4 inches from the surface. The tests were made without concrete encasement of the connector and the load on the stud was

primarily bending rather than shear as in actual service. The shear connectors exhibited very high fatigue strengths in stress reversal under these loading conditions.

Fatigue tests of 12 beams with 1/2 inch diameter connectors were conducted at Lehigh University in 1962.[7] The objectives were to study the performance of beams as connectors failed to devise a means of detecting failure. This investigation produced the first fatigue failures of stud connectors in beams. A method of detecting initial failure of connectors was also developed.

In previous beam tests the initial failure of connectors was not detected and failure of the composite beam was studied by means of measurements of slab movement or beam deflection.[5] When the slab was removed after considerable increases in slip and deflection had been observed, it was found that many connectors had failed prior to beam failure. This situation might mean that the ultimate strength of a member is reduced by connector failure prior to any appreciable effect on elastic performance.

The S-N curve obtained from these beam tests was found to be slightly above the fatigue failure results obtained from pushout tests of 1/2 inch and 3/4 inch connectors. These tests further substantiated that the properties of the composite beam would not be impaired prior to fatigue failure of connectors. The distribution of forces on connectors was studied using the instrumentation provided for detecting connector failure. Some variation of connector force was observed in regions of constant shear, due to the distribution of bond and friction forces.

However, the measured stress on the end connectors was in good agreement with the stress calculated by elastic theory.

Seven full size beams with 3/4 inch diameter connectors were tested at the University of Texas.[8] The apparent fatigue strength of these beams was less than the strength obtained with 1/2 inch diameter connectors at Lehigh. A difference of about 3 ksi in unit shear stress for the two sizes of connectors was found for corresponding cycle lives.

The data from these previous investigations for stud connectors are summarized in Table 1 and the range of shear stress versus number of cycles to failure is plotted on a semi-log scale in Fig. 1 with stress range as the ordinate and number of cycles to failure as the abscissa. Even with different types of tests, different failure criteria and different sizes of studs involved the scatter of the data is no greater than for typical fatigue data. All of the test data plotted in Fig. 1 was obtained for conditions of near zero to maximum. Stress range rather than maximum stress was used for plotting the data, however, it was not conclusively shown by any of these investigations that stress range was indeed the significant parameter as it differed very little from maximum stress.

Static tests have shown that complete interaction can only be achieved in beams when the number of shear connectors is great enough so that bond is not broken between the slab and beam.[5,7] These studies have indicated that incomplete interaction is apt to exist in all members used in practice. The effect of the lack of complete interaction on the behavior of beams has been studied by several investigators [5,16,17] and

shown to be minor. The loss of interaction due to slip has no significant effect on stress in the cross-section so long as the shear connection is adequate to develop the flexural strength of the member.

It was also shown in one investigation that the ultimate strength of the composite member is not affected by slip when an adequate shear connection is provided.[18] Weak or inadequate shear connectors resulted in a reduction in the flexural strength of the beam.[6,18]

The minimum number of shear connectors required to ensure flexural strength is considerably less than the number provided in a beam by the concept of useful capacity.[18] This lower limit is based on resisting the maximum force in the slab by connectors stressed to their ultimate strength.

2. TEST SPECIMENS AND TESTING PROCEDURES

2.1 TYPES OF SPECIMENS

2.1.1 General

An evaluation of the interplay of all aspects of bridge design requires the testing of bridges such as was carried out at the AASHO Road Test.(19) In general, the field testing of bridges is of limited usefulness in repeated loading because actual designs are too conservative and are based on design procedures which do not result in a balanced condition with regard to the various components. Also, the instrumentation used may not be indefinitely reliable and the cost and time consumed by such tests is extremely high. As a proof test of a given design such tests are invaluable, but as a means of evaluating a large number of variables a more systematic approach is needed.

Each pair of shear connectors between the support and midspan of a simple span beam is subjected to a different value of maximum stress, minimum stress, and stress range. If the steel beam was shored during construction, the loading on each connector is different than if the beam was not shored. It is necessary to study the limits of these stress conditions to find which conditions govern and which variable or variables can be neglected. A number of tests must be made so that the importance of all factors can be evaluated statistically.

2.1.2 Beam Specimens

Beam specimens may closely approximate the performance of a bridge deck. However, it has already been pointed out that the laboratory loading condition varys greatly from actual bridge loading conditions. The laboratory tests normally produce connector failure near the end of the test member.

Static beam tests show that minimum slip between slab and beam occur when the spacing of shear connectors is in proportion with the maximum shear at any point. Any arrangement of shear connectors may be used, but the actual force on each connector is difficult or impossible to determine. A redistribution of forces takes place when flexible connectors are used, which makes it difficult to determine the actual state of stress in the connectors. Instrumentation installed on shear connectors is apt to disturb the distribution of forces since the mechanism of load transfer by connectors is a combination of tension, bending, and shear on the connector.

Another difficulty observed in beam tests is that forces in the concrete slab may be distributed by longitudinal reinforcing steel or non-linear variation of concrete compression stresses from one group of shear connectors to another. For instance, it was found that connectors located between two symmetrically placed loading jacks in a region of constant moment actually participated in the transfer of horizontal shear forces across the interface. For this reason it was deemed advisable to place jack loads sufficiently close together to permit the elimination of connectors in the center portion of the test beam. This procedure was followed in the studies reported in References 7 and 8.

Perhaps the most serious limitation of beam tests is the relatively large size specimen required to test full size connectors. The slab and beam must be large enough to produce a sizeable compressive force in the slab and the span must then be long enough so that reasonable loads are required to produce the slab force. The end result is an expensive specimen which cannot be tested at a fast rate of loading because of its low natural frequency of vibration.

2.1.3 Pushout Specimens

The typical pushout specimen which has been in use since 1933 for tests of shear connectors consists of two concrete slabs connected to each flange of a steel wide flange or I section by means of the shear connectors. The specimen is supported by the ends of the slabs and load is applied to the top of the steel beam with the load transferred to the slabs by the connectors. This specimen is representative of end connectors in a beam where the compressive force in the slab on one side of the connectors is zero.

Specimens of this type must be fabricated with considerable care if valid test results are to be obtained. Even when every precaution is taken in constructing the specimen, results may be unsatisfactory because of variation in the properties of the concrete slabs or differences in the quality of the concrete surrounding the shear connectors. Eccentricities of loading on the connectors which cannot be compensated for sometimes occur in testing. For this reason it is always necessary to test a group of specimens and discard data from a specimen which is obviously not typical.

The test result from a pushout specimen can be expected at best to be conservative because there is no possibility for redistribution of forces among connectors as there is in a beam. It is tacitly assumed that each slab supports half of the load applied to the steel beam. In practice this is not easily achieved and in fact the proportion of load carried by each slab may vary during the testing of a single specimen. It appears from all existing data that the load versus slip characteristics from pushout specimens and beam specimens come nearer to being identical for loads below useful capacity than for higher loads.[20]

The results of pushout tests are easier to evaluate than beam test results because the average shear connector force is obtained directly from the load on the specimen by dividing by the number of connectors involved in the test. The comparison of results from this type of pushout test with beam test results given in Fig. 1 indicate that the pushout specimen is equivalent to or produces results which are slightly more conservative than beam test data.

2.2 DESCRIPTION OF TEST SPECIMENS

2.2.1 General

The conventional pushout specimen discussed in Art. 2.1.3 is not readily adaptable for fatigue testing with stress reversal loading. For this reason a single slab specimen was used.

It is desirable to cast the slabs of pushout specimens horizontally as a bridge deck is cast to avoid variation in concrete properties across the width of slab and prevent the formation of voids around connectors. Specimens with two slabs require two separate

concrete pours, which is undesirable because the concrete properties in the two slabs may differ. This problem was eliminated by using a specimen with only one slab.

2.2.2 Details of Specimens

The main experiment for each type of connector used a specimen consisting of a 2'-6" length of 8 W 40 steel beam with a concrete slab 6 inches thick and 2'-3/4" long by 1'-8" wide attached to it as shown in Fig. 2. Shear connectors were placed at two locations. Eight 1-1/16 inch diameter holes in the opposite flange were used for bolting the specimen to a suitable support for testing, and this connection was designed as a friction connection for all loading conditions.

The concrete slab thickness was chosen as the minimum slab thickness that might be used on a composite bridge deck. The percentage of reinforcing steel used in both directions in the slab was typical of bridge slab design. Each specimen contained a cage of reinforcing steel which was welded of closed loop bars to ensure that all steel was fully anchored. The reinforcing steel cages were carefully positioned so that there was a minimum of 1-1/2 inches of concrete cover around all shear connectors.

Pilot tests were made with stud and channel shear connectors to evaluate the specimen configuration. At higher loads the 8 W 40 section permitted some rotation through web bending due to small eccentricities of the flanges, bolts, and shear connectors with respect to the web and in some cases slight errors in the centering of the slab on the web. The problem was corrected by welding 3/8 inch end plates on all steel sections before testing.

The steel section selected for the specimen was chosen on the basis of past experience with pushout tests. It was necessary to choose a compact section to avoid excessive bending moments which might affect test results at high loads. The flange thickness was required to be typical of lighter bridge members.

2.2.3 Fabrication of Specimens

In addition to those fabricated with 8 W 40 sections, two specimens were constructed with 8 W 31 steel sections and two were constructed with 8 W 67 steel sections. These specimens were used to study the effect of flange thickness on the fatigue life of connectors.

Sixty-six specimens were fabricated by a local steel fabricator and five additional specimens with channel connectors were fabricated at Fritz Engineering Laboratory. Forty-four specimens had 3/4 inch diameter studs, 10 specimens had 7/8 inch diameter studs, and 17 specimens had 4 inch 5.4 lb. channel connectors.

The 8 W 40 steel sections were all from the same heat of steel. Shear connectors were welded and inspected following the procedure outlined in a draft of "Recommendations for Materials and for Welding for Steel Channel, Spiral, and Stud Shear Connectors," proposed by Sub Committee I of the ASCE-ACI Committee on Composite Construction dated July 10, 1964. The stud shear connectors were ordered specifically for the project and all were made from the same material. All channel connectors except the four supplementary test specimens fabricated at Fritz Engineering Laboratory were from the same material.

The concrete slabs were cast in groups of ten except for the first pour of pilot specimens in which only eight specimens were cast. A single form for a group of ten specimens was constructed and was supported on the ends of the steel beams. The use of this type of form made it possible to obtain specimens with ends square and true. The specimens were cast in seven pours using the same concrete mix. The concrete slabs were moist cured using burlap and polyethylene covering for seven days followed by air curing until the time of testing. The testing of pilot specimens was begun twenty-one days after casting. All other specimens were at least twenty-eight days old at testing.

2.2.4 Properties of Materials

The steel beams and channel stock for shear connectors were ASTM A36 steel. The reinforcing steel used was deformed bars of intermediate grade conforming to the ASTM A15 Specification. The steel rod material from which the stud shear connectors were made conformed to the ASTM A108-51T Specification.

Tensile tests for material properties were made on shear connector materials. As mentioned previously, one fabricated specimen containing each type of shear connector was used for obtaining these specimens. In the case of stud connectors a section of flange 4 inches by 4 inches was cut from the 8 W 40 with a connector in the center of the plate. The plate was clamped to the head of a testing machine and a tensile force was applied to the head of the stud. Eight specimens containing 3/4 inch diameter studs and four specimens containing 7/8 inch diameter studs were tested to failure in this manner. Four of the 3/4 inch diameter connectors were from the main experiment and the other

four from the six supplementary test specimens. All data on properties of materials are summarized in Table 2. A pair of electrical resistance strain gages mounted on opposite sides of the connector and connected in series was used to obtain the stress-strain relationship for the connector material.

Tensile coupons having a 2 inch gage length were cut from the webs of channel connectors, and pulling tabs were welded on the ends. The results of these tests are also reported in Table 2.

The concrete mix used for all specimens is given in Table 3 along with the slump for each pour. For the main experiment involving 3/4 inch diameter shear connectors, two concrete cylinders were made for each specimen and were tested at the beginning of the fatigue loading of the specimen. For the 7/8 inch diameter stud experiment and the experiment for channel connectors only one concrete cylinder was tested for each fatigue specimen.

The concrete mix was designed for a strength of 3500 psi at twenty-eight days. The actual concrete strengths for the main experiment of each type of connector were 4300 psi with a standard deviation of 340 psi for 3/4 inch studs, 4470 psi with a standard deviation of 77 psi for 7/8 inch studs and 6045 psi with a standard deviation of 90 psi for the channel connectors. The concrete strengths are summarized in Tables 6 through 9.

The material properties of the steel sections and reinforcing steel are summarized in Table 4.

2.2.5 Testing Procedure

A single concrete mix was used for all specimens in an attempt to eliminate concrete strength as a variable. Comparable concrete strengths were obtained except for the one pour for channel connectors. This pour was made during a period of very hot weather and the change in curing conditions resulted in a higher strength. Within each group of specimens the concrete strength varied only slightly from the twenty-eight day strengths. The specimens were tested at random to minimize the effect of uncontrolled variables such as variation in concrete strength. Since a separate test setup was required for the stress reversal tests, a random order of testing was followed within each type of test.

The test setup for loading in one direction is shown in Fig. 3. The specimen was bolted to a vertical column bracket using eight 1 inch diameter ASTM A325 high strength bolts tightened to a minimum torque of 700 ft.-lbs. During the initial tests a dial gage mounted on the column was used to check for slip, but none was detected during any of the tests. The load was applied to the concrete slab by a hydraulic jack of 55 kip or 110 kip capacity. Most of the tests were conducted at a frequency of 500 cycles per minute.

The testing procedure for each specimen consisted of an initial static test with slip readings being taken on the dial gage. The data from each static test was compared to available data on the load-slip characteristics of the connector being tested. Throughout the testing period a record of dynamic maximum slip was kept by reading the same

dial gage as a feeler gage. This served to detect any changes in loading conditions or specimen performance.

The failure criterion for the tests was established from careful observation of the first specimens to be tested. Complete failure of the entire shear connection was not a suitable criterion because all connectors did not fail together. Of principal interest was the initial connector failure. Experience gained in the pilot tests determined the settings of the safety devices for stopping the test after failure of the connectors near the load bearing surface. Generally, a ten percent drop in maximum load was used for tests in which the specimen was loaded in one direction. A five percent drop was used in the stress reversal tests.

During the pilot tests electrical resistance strain gages were mounted on the inside surface of the flange of the steel section opposite the shear connectors to measure local bending stress due to the shear connector force. These gages were used to check on the rate of failure of connectors. It was found that failure of connectors progressed rapidly requiring less than 25,000 cycles from apparent first cracking to complete failure of connectors.

The special loading arrangement used for alternating stress is illustrated in Fig. 4. One jack exerted a constant load against one end of the slab by means of a load maintainer and a column of nitrogen. The other jack was connected to a pulsator unit and applied load at a rate of 250 cycles per minute of such magnitude that the slab moved through its zero-load position and was displaced in the opposite direction. The

movement of the slab prior to failure was always quite small so that the dynamic load correction was negligible.

A single dial gage was used to measure the relative slab movement with respect to the 2 inch thick steel base plate to which the specimen was bolted. The alternating load test was also preceded by a static test and dial readings were taken at frequent intervals during dynamic loading by releasing the plunger of the dial and observing the maximum position of the slab in one direction. Initial failure of these specimens could be observed visually by separation of the slab from the beam at the pulsator jack end and vibration of the slab. Failure of connectors progressed less rapidly than in tests with loading on one end of the slab. Usually less than 100,000 cycles were required after initial cracking for failure of the shear connection. The number of cycles required for failure of the connectors was recorded. Failure of the connector or connectors adjacent to the constant load jack was not achieved in any of these tests because the constant load forced premature stopping of the pulsator.

3. DESIGN OF EXPERIMENT

3.1 TEST VARIABLES

3.1.1 General

Loading conditions on the shear connectors in composite bridge beams were studied to determine the extremes which might be encountered in various bridges. Connectors are subjected to various ranges of shear stress as live loads cross a bridge span. The magnitude of the range of shear stress can be controlled by the spacing and number of connectors provided. Each connector in a bridge span is also subjected to a minimum stress which depends on its location in the span, the magnitude of the dead load stress on the composite section, and secondary effects such as creep and shrinkage. The magnitude of this minimum stress can be approximated when appropriate assumptions are made.

It has been shown that the dead load stresses on the composite cross-section are time dependent. Methods of analysis for steel and concrete stresses have been developed[21] to evaluate time dependent changes, but the effect of these changes on the shear connector stress has not been ascertained because of the inelastic deformations of the elastic-plastic medium in which the shear connectors are embedded. The minimum stress level is also affected by shrinkage of the concrete slab as well as volumetric changes in the concrete due to variations in relative humidity and other environmental factors. Differential

temperature changes between slab and supporting beams may also influence the minimum connector stress.

Experience has shown that the extremes of these factors have not resulted in shear connector failure. It may be that the effect of the minimum stress condition as a factor in the deterioration of concrete wearing surfaces on bridges deserves greater attention than it has received, but this problem is beyond the scope of this investigation. These observations and experiences have suggested that the minimum stress level may not have a significant effect on the fatigue strength of the connectors.

3.1.2 Primary Variables

The primary variables selected for evaluation were the type of shear connector, the minimum stress, the maximum stress, and the range of stress.

The number of levels of minimum stress which could be considered was limited to three because of necessary limitations on the size of the experiment. The data plotted in Fig. 1 indicated that the S-N curve would be relatively flat and for this reason the number of levels of maximum stress or stress range was limited to five.

Table 5 shows the experiment design used for the main experiment on each type of connector. The experiment contains four 2 x 2 factorial experiments as outlined by dotted lines in the table. Each level of minimum stress was combined with three levels of maximum stress and stress range such that two complete 2 by 2 factorial experiments were included to obtain data on the effect of minimum stress on the other two

factors. Three replications of the main experiment were carried out for 3/4 inch diameter studs. Only one test was made for each of the other types of connectors. The experiments were proportioned so that the effect of the major test variables could be ascertained and their relative significance assessed.

3.1.3 Secondary Variables

In addition to the main factorial experiments for each type of connector, additional specimens were tested to evaluate other variables. Four specimens with 3/4 inch diameter studs were tested to study the effect of flange thickness on connector fatigue strength. These tests are summarized and discussed in Section 4.8.1. Four specimens with 3/4 inch diameter studs were also investigated to study the effect of occasional overloads on the fatigue strength of connections. These tests are summarized in Section 4.8.2.

There were a number of uncontrolled variables in the test program. Their effect was minimized by either holding testing conditions constant throughout the testing program or scheduling the tests in a random sequence. Three categories of uncontrolled variables were considered. These were properties of materials, testing system, and testing procedure.

The material properties were kept as constant as possible throughout the testing program. All 8 W 40 steel sections were fabricated from the same heat of steel. A single shipment of 3/4 inch diameter studs was used for all specimens except the six supplementary test specimens, which were made with a separate shipment of studs. All specimens with

7/8 inch diameter studs were made from a single shipment of studs. The channel specimens were made with channel material from a single heat of steel and the welding was as nearly uniform as possible. However, the four supplementary channel test specimens were made with a different channel material and welding procedure.

The variability of the concrete strength was kept to a minimum by casting each group of ten specimens from the same concrete mix. However, the concrete strength did vary for four different groups of specimens because of variations in aggregate, cement, and curing temperature. Since specimens were cast and tested over a period of eight months, some variation in concrete properties was unavoidable. The age of specimens at the time of testing varied considerably because of the period of time required for completion of the test program.

A random order of testing was used to avoid the confounding of concrete age and strengths with the other test variables. Previous fatigue tests of beams containing channel connections had indicated that concrete strength variation might have an effect on test results in proportion to $\sqrt{f_c'}$. However, data from previous tests of stud shear connections indicated that variations in concrete strength within the range of 3000 psi to 6000 psi would not measurably affect the results.

The factorial experiments were split into two portions compatible with each test setup. A random order of testing was followed within each portion.

The conditions of the testing system were kept constant by close supervision of details in fabrication and positioning of test specimens

as well as continuous monitoring of the testing to detect any possible variation in equipment operation. The reinforcing steel for all specimens was welded into cages using a jig so that the position of the reinforcing steel relative to the shear connections was approximately the same for all specimens having the same type of connector. The minimum concrete cover between shear connector and reinforcing steel of 1-1/2 inches was maintained with all connectors.

The bearing surface for the jack was inspected to be sure that it was flat and perpendicular to the axis of the specimen and the top flange of the steel beam. The better of the two possible bearing surfaces was chosen whenever a choice was possible. In stress reversal tests and in tests involving channel connectors where a particular bearing surface had to be used, the surfaces were prepared by grinding or capping with a quick setting gypsum plaster if grinding was impractical. These preparations were taken to ensure that the force applied by the jack was parallel to the flange at all times. Testing equipment was checked frequently to assure that the force in the slab did not change direction during the test.

Experience gained in previous investigations indicated that testing speeds of 250 and 500 cpm would not affect the test results so long as the dynamic correction did not become large due to excessive slip. The slower testing speed was used for reversal tests and tests with high loads in one direction.

3.2 INVESTIGATION OF 3/4 INCH DIAMETER STUDS

3.2.1 General

This investigation was directed toward stud shear connectors because these are extensively used in bridge construction. The magnitude of slip for this and other types of connectors has been used as the index of shear connector behavior. In the past the allowable loads for connectors has been established by an evaluation of the load-slip curves to obtain a limiting value of slip and a corresponding allowable load.

It has been shown that the magnitude of slip does not affect the ultimate strength of members,[18] and it has also been shown that the loss of interaction which results from small slips at working load under repeated loading is not significant.[7,8] The fatigue strength of beams can therefore be determined on the basis of shear connector failure and a limitation on the maximum slip is not necessary.

If slip is permitted, a very severe condition of fatigue loading exists due to the complicated system of stresses acting on a shear connector. The geometry of the base of a welded stud connector results in sharp notch conditions at the base of the stud. A sharp notch exists between the fillet and the flange of the beam. The base of a stud after fracture is shown in Fig. 5. A similar sharp notch exists between the shank of the stud and the weld bead. Either of these notch locations can provide a point for the initiation of a fatigue crack.

Because of the unfavorable geometry and stress conditions associated with repeated loading on shear connectors, the fatigue strength of connectors requires extensive experimental investigation.

3.2.2 Pilot Tests

Since the test specimen had not been used before this investigation, it was necessary to evaluate a few pilot tests and compare the data with existing information. Two specimens with 3/4 inch stud connectors were tested and carefully observed. The slip of the slab with respect to the steel flange was measured and local strains in the flange near connectors were also measured and compared with data from earlier beam tests.

The results of these pilot tests agreed with beam tests and other pushout test results. In addition, it was desirable to evaluate the selected stress levels of the main experiment to ascertain what cycle lives could be expected. During these tests it was found necessary to weld end plates on the steel section to prevent twisting of the specimens.

3.2.3 Principal Experiment

In previous research the shear connector has been evaluated on the basis of the maximum shear stress on the connector. The experiment design therefore necessarily included maximum stress as a variable, and information from previous tests was useful in designing the experiment. The useful capacity of connectors has been established as the load at which the load-slip curve becomes non-linear or the point at which the connector material begins to yield.[4] This was used as a guide to the maximum permissible load on a connector. For concrete with a strength of 3500 psi the useful capacity of 3/4 inch diameter stud connectors is approximately 25 ksi. One of the pilot specimens was tested at a maximum shear stress of 30 ksi to evaluate the effect of high shear stresses on the fatigue strength.

The principal experiment was designed so that the fatigue strength could be established for a range of cycle lives. The pilot studies indicated that cycle lives varying between 50,000 and several million cycles could be expected if the levels of minimum stress shown in Table 5 were -6, +2, and +10 ksi while the levels of maximum stress were +10, +14, +18, +22, and +26 ksi. The levels of stress range given as S_A' through S_E' in Table 5 are 8, 12, 16, 20, and 24 ksi.

This suitably covers the range of possible values of maximum stress from stress reversal to useful capacity except for the region near complete stress reversal. It was believed that this region could be safely omitted unless test results revealed otherwise. The highest value of minimum stress was selected somewhat arbitrarily to comply with the experiment design. This value was approximately 120 percent of the maximum design load permitted on connectors under the present specifications.

3.2.4 Supplementary Tests

As the tests for the main experiment were being completed, the data was examined to arrive at a suitable design stress. Using a cycle life of two million cycles as a hypothetical design criterion, a stress range of 10 ksi appeared to be satisfactory for all levels of minimum stress.

It was decided to supplement the main experiment by additional tests with a stress range of 10 ksi. Inspection of the data also revealed that only the +2 ksi and the +10 ksi minimum stress levels needed to be considered in making these supplementary tests. Six specimens were

cast; three were loaded from +2 ksi to +12 ksi and the other three from +10 ksi to +20 ksi.

To investigate the effect of overloads, four specimens were tested in which an overload of 5 ksi was imposed at intervals of either 10,000 cycles or 50,000 cycles. Two specimens were tested at a minimum stress of +2 ksi and two at a minimum stress of +10 ksi, and all four had a range of stress of 10 ksi.

The minimum thickness of flange material permitted for bridges is 3/8 inch, and it seemed desirable to test some connectors welded on a flange of similar thickness. An 8 W 31 steel section having a nominal flange thickness of 7/16 inch was selected as being the nearest available cross-section to the minimum requirement. Good design practice may preclude the use of a flange of this thickness in combination with 3/4 inch diameter connectors, but for the purposes of this investigation it was a suitable lower bound for flange thickness. To further evaluate the effect of flange thickness other than that used for the main experiment an 8 W 67 section was selected for tests with a thicker flange. This section has a flange thickness of 0.933 inch. The flange thickness of the 8 W 40 section used for the main experiment was 0.558 inches. Two specimens of each flange thickness were tested using a minimum stress of +2 ksi and a stress range of 12 ksi on the connectors.

3.3 INVESTIGATION OF 7/8 INCH DIAMETER STUDS

3.3.1 General

Current practice has indicated that larger diameter stud shear connectors are becoming more popular. Since the previous studies had

indicated a substantial difference between the fatigue strength of 1/2 and 3/4 inch diameter connectors,[8] it was desirable to ascertain the behavior of 7/8 inch studs.

It would be convenient to use the same unit stress in the design of connectors of different diameters. This is not ruled out by the size effect difference between 1/2 inch and 3/4 inch diameter connectors because the smaller connector is not used in bridge design. The feasibility of this design technique can only be investigated experimentally, and this was undertaken by repeating the main experiment as developed for 3/4 inch diameter studs.

3.3.2 Main Experiment

The unit shear stresses used in the experiment with 3/4 inch connectors were also used for this experiment. Only one specimen was tested for each loading condition (one test block) for a total of nine specimens. It was recognized in planning the experiment that if a size effect was revealed in the tests, additional studies might be necessary.

3.4 INVESTIGATION OF CHANNEL CONNECTORS

3.4.1 General

Static tests of full size channel shear connectors usually produce a tensile cracking failure of the concrete slab. A fracture of the connector is rare. In fatigue loading, the fatigue strength of the welds is critical, and it was doubted that slab failure in fatigue was possible.

Fatigue results on full size channel connectors were not available. The experiment was designed assuming that fatigue failure of the welds would be the governing factor. Values of maximum and minimum load per inch of connector were chosen using the fatigue strength of transversely loaded fillet welds as a guide. Pilot tests were necessary to verify the experiment design before proceeding with the main experiment.

3.4.2 Pilot Tests

Three pilot tests were conducted before proceeding with the main experiment design. The first two were identical to specimens used for the main experiment and were tested to confirm the selected values of maximum and minimum stress. Both specimens were loaded with the flanges of the channel facing toward the jack. Static tests had indicated this to be the weaker direction for loading of a channel connector. The direction of loading was reversed for the main experiment because most channel connectors are installed with the flanges pointing toward the supports.

It was observed during these two pilot tests that the load was not evenly distributed to the two channel connectors. The connector adjacent to the loaded edge failed completely before failure of the second connector was initiated. This led to the fabrication and testing of a third pilot test specimen in which a single connector was installed at the center of the steel section. This specimen was tested with the load per inch of connector the same as in one of the previous tests. The cycle life of this specimen was very nearly the same as that of the

corresponding specimen with two connectors. The main experiment was therefore conducted with specimens having two channel connectors.

The mode of failure for two pilot specimens was a fracture through the throat of the welds. One connector failed by fracture through the web. However, it seemed apparent that this mode of failure would not be encountered in the main experiment where the direction of loading was reversed.

3.4.3 Principal Experiment

The experiment design was the same as that shown in Table 5 with minimum stress levels at -0.5, +0.5, and +1.5 kips per inch of channel and maximum stress levels at +3.0, +3.5, +4.0, +4.5, and +5.0 kips per inch of connector. The corresponding range of shear was 2.0, 2.5, 3.0, 3.5, and 4.0 kips per inch.

The highest value of maximum stress was slightly less than the useful capacity of the channel connector in 3000 psi concrete. The results of the pilot tests had indicated that this was a reasonable limit. In terms of the unit shear stress on the throat of the fillet welds, the maximum stress varied from 11.3 ksi to 18.9 ksi and the range of shear stress varied from 9.4 to 17.0 ksi.

3.4.4 Supplementary Tests

All specimens for the main experiment with channel connectors were cast from a single batch of concrete. The concrete developed a compressive strength of about 6000 psi, and additional tests with lower concrete strength were desirable.

Static tests of channel connectors had clearly indicated that the behavior of connectors varied with $\sqrt{f'_c}$. A similar relationship between concrete strength and the fatigue strength of channel connectors had been developed through tests of model beams containing small channel connectors.[5] The cross-section geometry of these connectors was not similar to that of standard channel connectors, and it was thought that this difference might possibly have an effect on the influence of concrete strength.

Four additional channel specimens were fabricated and tested with minimum stress levels at +0.5 and +1.5 kips per inch of connector. The stress range for these connectors was 2.0 and 2.6 kips per inch. These values of stress range resulted in a unit stress on the nominal throat of the weld of about 10 ksi. The average concrete strength for these specimens was 4680 psi.

4. TEST RESULTS AND ANALYSIS

4.1 GENERAL

The data for all fatigue tests of shear connectors are summarized in Tables 6 through 9. The primary data for 3/4 inch diameter studs are given in Table 6 while data for exploratory tests involving the effect of flange thickness and occasional overloads are given in Table 9. Table 7 summarizes the data for 7/8 inch diameter stud connectors and Table 8 gives the data for the 4 inch 5.4 lb. channel connectors. The tables include specimen designation, the loads to which the pushout specimen was subjected, maximum and minimum shear stress, cycle life, and the age and strength of the concrete at the time of testing.

For stud connectors the shear stress was calculated as the average stress on the nominal diameter of the stud. The shear stress reported for channel connectors was determined as the average shear stress on the effective throat area of the fillet welds. In previous work the load on channel connectors has been reported in terms of load per inch of channel, which can be obtained by dividing the load on the specimen by the total length of channel (12 inches).

The cycle life varied from 30,600 up to 10,275,900 cycles for stud shear connectors and from 291,000 up to 9,556,300 cycles for the 4 inch 5.4 lb. channel shear connectors. The concrete cylinder strength varied from 3120 to 4730 psi for the stud connectors, and from 3610 to

6360 psi for the channel connectors. The three groups of specimens included in each of Tables 6 and 8 were treated as a single sample for purposes of analysis. All specimens exhibited a finite life. The endurance limit was not determined within 10 million cycles of loading for the types of connectors tested.

4.2 FAILURE OF CONNECTORS

All specimens were cycled until fatigue failure progressed to the extent that the shear connection could no longer support the maximum load. In every specimen this entailed complete fracture of the channel or pair of studs adjacent to the loading jack. The remaining channel or pair of studs was partially failed in fatigue and in most instances the stud connectors were sheared off and the channel connectors were pulled from the slab. During stress reversal tests the channel or pair of studs near the accumulator jack did not fail completely because the accumulator system did not permit sufficient movement of the slab for complete failure.

Two different forms of failure were observed for stud shear connectors. The most common form of failure was a crack which was initiated at the reinforcement of the stud weld and penetrated into the beam flange. This resulted in a crater being removed from the flange which was approximately the diameter of the weld bead and about 1/8 inch in depth. The other form of failure was fracture of the stud through the weld at the level of the flange surface leaving the weld bead attached to the flange of the beam. Both types of failure are shown in

Fig. 6. The first form of failure in Fig. 6a has a "hickory" appearance on the surface and is very similar in appearance to the fractured surfaces of plug welds which fail in shear.[22] The form of failure in Fig. 6b has a very smooth crystalline appearance and is similar to the initiation area of a tensile or bending fatigue failure.

Most of the fatigue failures were similar to the fracture shown in Fig. 6a. It is apparent that shear is primarily responsible for fracture of the stud connectors. The shear type of failure (Fig. 6a) was also observed most frequently in beam tests. The form of fracture was not a significant variable in any of the tests.

It was also observed in the tests with stud shear connectors that the load was more evenly distributed to both pairs of connectors for the high minimum stress level and the lower ranges of stress. In these tests all four connectors were fractured about the same extent by the fatigue loading.

One specimen with channel connectors failed through the web while all others failed through the welds. The web failure was observed in pilot specimen P3. The welds of this specimen were slightly larger than desired and load was applied at the "open end" of the channel. The reversed direction of loading with respect to the channel position and the oversize welds in this pilot test may have contributed to conditions which result in web failure. Figure 7 shows the typical failure of channel connectors through the throat of the weld.

It was obvious from the failures of the channel that the connector nearest the applied load was more highly stressed than the other

connector. The fatigue failure was usually confined to one connector and the connector located farthest from the load point failed statically. Figure 7 shows a pilot specimen on the right and a specimen from the main experiment on the left.

Because the load was not evenly distributed to channel connectors, specimen P6 was fabricated with one channel connector. The cycle life of this specimen was not significantly different from that of the specimens with two connectors. This comparison indicated that the use of two connectors would yield slightly conservative results because greater loads would be carried by the connector adjacent to the load. Since no full size beam tests were available for comparison, the two channel specimen was used.

4.3 THEORETICAL ANALYSIS OF CONNECTOR FAILURE

Both stud and channel shear connectors have been referred to in the literature as flexible connectors. The load-slip characteristics of the two types of connectors are compared in Fig. 8. The initial static curves for b 5 C and d 5 F represent specimens which had the highest loads in the main experiments for 3/4 inch diameter studs and 4 inch 5.4 lb. channel connectors respectively. Within the region up to 70 percent of the useful capacity the connectors exhibit similar performance. Previous studies have shown that either of these curves may change in linearity and slope due to changes in concrete strength, percentage of reinforcing steel or bond characteristics between beam and slab.[6,18,20]

The load-slip curves tend to become reasonably linear within the limits of the cycle loading after a small amount of permanent set. This is illustrated in Fig. 9 for specimen P1 with 3/4 inch diameter studs. It was apparent during the fatigue loading that the response was reasonably linear for all specimens.

The connectors tended to act as dowels in an elastic medium. A theoretical solution has been developed from the theory of beams on an elastic foundation(23) for each type of connector. These solutions are useful in evaluating qualitatively the mode of failure of each type of connector. The assumed forces acting on a connector are indicated in Fig. 10. The most important component is the shear force, V. The moment, M, results from the fixity of the connector at the beam flange. The tensile force, T, is dependent on the slip deformation and is small under normal loading. This component may reach considerable magnitude as the ultimate strength of the connector is reached. The tensile force is developed by the mechanical anchorage provided by the geometry of the connector. This anchorage is essential if the ultimate strength of a connector is to be used in design.

A solution for the deflected shape and bending stresses in a stud shear connector is given in Appendix B based on an assumed set of boundary conditions. The four boundary conditions used for the solution consisted of (1) and (2) the angle of rotation equal to zero at both ends, (3) the shear equal to zero at the embedded end of the connector, and (4) the deflection at the flange obtained from an average load-slip curve. The first two boundary conditions were based on the assumption that the flange and the head of the stud are adequate to prevent rotations.

The third boundary condition follows logically from the geometry of the connector. The last boundary condition was derived from the condition that the foundation and connector move by the same amount. The foundation constant used in the analysis was taken from the results of triaxial tests of concrete.[24]

The results of the analysis for a 3/4 inch diameter stud of 4 inch length indicates that when the maximum shear stress is 10 ksi the maximum fiber stress in bending at the base of the stud would be about 58 ksi or nearly 80 percent of the ultimate strength of the stud material. This analysis suggests that the point of initiation of the crack may be affected by the bending stresses. However, several factors such as local yielding of the steel and concrete and rotation of the beam flange affect the distribution of forces so that the primary force is the nominal shear acting on the stud connectors.

Channel connectors have been treated by a similar approach except that the geometry of the connector dictates different boundary conditions than those used in the analysis of stud connectors. Analysis of test results on 4 inch 5.4 lb. channel connectors showed that the stiff portion of the connector i.e., the flange which is welded to the beam, carried from 77.1 percent to 90.7 percent of the total load,[6] the remainder was carried by the flexible portion consisting of the web and embedded flange. By assuming that the channel flange attached to the beam flange takes 80 percent of the shear force, the bending moment transmitted to the main member is much less than that developed by stud connectors. The effect of bending on the stress at the throat of the

fillet weld can be neglected, and the effect of the shear force alone on the welds may be considered as producing the fatigue failure.

The concrete in the vicinity of connectors is subjected to rather high bearing stress, particularly with stud connectors. However, the bearing capacity of the concrete in this region may be nearly five times the compressive strength[24,25] and any point of high bearing stress will result in a redistribution of bearing stress due to the elastic properties of the concrete. Fatigue failure of concrete is not likely in a heavily reinforced concrete slab.

All of the channel connectors except one failed in the weldments. Pilot test P4 produced failure through the channel web because of oversize welds at the heel of the channel. However, specimens having connectors loaded in the opposite direction and a similar weld size did not fail through the web. This indicated that the direction of loading may influence the mode of failure.

The theoretical analysis reveals that the failure modes obtained by the connectors are those which would be expected from the relative magnitude of bending and shear stresses which are produced. The relatively high bending stresses in the stud connectors are important in initiation of the crack. The bending stress, on the other hand, is negligible in channel connectors because of the small portion of the total force on the connector which is carried by the web and embedded flange. In both instances the failure of the connector is the result of the shear force.

4.4 MATHEMATICAL MODELS USED FOR ANALYSIS

The S-N curve for the representation of fatigue test data is commonly expressed in semi-log or log-log form. These forms involve the plotting of either maximum stress or stress range as ordinate and cycle life as abscissa using arithmetic or logarithmic scales. Arithmetic scales for both variables are seldom used.

The S-N curves may be plotted with either the maximum stress or the stress range as ordinate. Generally the data results in a curve with a negative slope from 50,000 to about 2,000,000 cycles and a horizontal line for long cycle life which gives rise to the concept of endurance limit. Various mathematical models have been used to express the relationship between applied stress and the cycle life up to the endurance limit.

Basquin[26] proposed that the finite fatigue life could be represented by the equation

$$S_{max.} = \alpha N^{-\beta} \tag{4.1}$$

where

α, β = experimental constants

$S_{max.}$ = maximum stress in cycle

N = number of cycles to failure

This curve is a straight line when plotted on a logarithmic scale and is applicable from 50,000 cycles out to its intersection with the endurance limit. Wilson and others have used this model extensively.[22]

A slight variation of this model is to express the fatigue life in semi-logarithmic form, giving

$$S_{max.} = \alpha + \beta \log N \qquad (4.2)$$

Weibull[27] first proposed an exponential model that was similar to Basquin's but was based on the stress range rather than maximum stress. It had the form

$$\frac{S_r - S_e}{S_{ult.}} = \alpha N^{-\beta} \qquad (4.3)$$

where

S_r = stress range; $S_r = S_{max.} - S_{min.}$

S_e = endurance limit

$S_{ult.}$ = tensile strength

A modification of this model is of the form

$$S_r = \alpha N^{-\beta} \qquad (4.4)$$

This is particularly applicable when no endurance limit is apparent and where the fatigue strength is not dependent on the tensile strength.

Stress range can also be expressed in the semi-logarithmic form as was done previously for maximum stress. The resulting equation is

$$S_r = \alpha + \beta \log N \qquad (4.5)$$

This model has been used in Reference 7 and by Toprac[8] to represent the fatigue strength of stud shear connectors.

An additional model which involves only two variables is of the form

$$S_r = \alpha \, (e^N)^{-\beta} \tag{4.6}$$

where e is the base of the Napierian logarithm.

Fisher and Viest[19] have used the mathematical model

$$S_r = \alpha + \beta \log N + \gamma \, S_{min.} \tag{4.7}$$

to express the fatigue strength. This semi-logarithmic model could account for variations due to minimum stress as well as the major variable stress range.

A slight variation of this model in exponential form is

$$S_r = \alpha \left[\frac{N}{(S_{min.})^{\gamma}} \right]^{-\beta} \tag{4.8}$$

It is apparent that all models represent a decrease in fatigue strength with increased cycle life. Only Eq. 4.3 becomes assymtotic to the endurance limit (if it exists). All other equations will eventually intersect the endurance limit.

Two additional models were evaluated with channel connectors to ascertain the effect of concrete strength. These models were similar to Eqs. 4.7 and 4.8 with the variable of the minimum stress replaced by the square root of the concrete strength.

These models were

$$S_r = \alpha + \beta \log N + \delta \sqrt{f_c'} \tag{4.9}$$

and

$$S_r = \alpha \left[\frac{N}{(\sqrt{f_c'})^{\delta}} \right]^{\beta} \tag{4.10}$$

where α, β, δ, N, and S_r are as previously defined and f_c' is the concrete strength.

Where necessary these models were linearized by taking the logarithm of both sides to produce equations that were more convenient for analysis and plotting.

4.5 STUD SHEAR CONNECTORS

4.5.1 Fatigue Strength of Studs

The data from the pilot tests and main experiment with 3/4 inch diameter stud connectors are plotted in Fig. 11 with the maximum stress as ordinate on an arithmetic scale and the number of cycles to failure as the abscissa on a log scale. It is obvious that for each level of minimum stress, the data points form a distinct band across the graph. The maximum stress does not serve to correlate the test data since a separate S-N curve results for each level of minimum stress. Because of this observed behavior the mathematical models given in Eqs. 4.1 and 4.2 were not used except in preliminary analysis where the data for a single minimum stress level was studied.

The scatter of the data in each band of Fig. 11 is normal for fatigue data obtained from welded components. It is worthwhile to note that the scatter of the data for the upper band representing the highest minimum and maximum stress levels is not any greater than that exhibited by the other two bands of data. This is true in spite of the fact that the initial load-slip curves for these high loads are not linear as indicated in Fig. 9. Four of the data points represent maximum connector loads in excess of the useful capacity of the connector. The fatigue life of the connectors was not adversely affected by the non-linear characteristics of the load-slip relationship.

In Fig. 12 the same data is plotted with stress range (maximum stress minus minimum stress) as the ordinate on an arithmetic scale versus the number of cycles to failure on the abscissa as before. It is apparent that the data plots in two bands. The upper band contains the stress reversal data whereas the data for the other two levels of minimum stress form a single band. Within the lower band there is little or no difference between data points, which can be attributed to the difference in minimum stress level.

If only the zero to maximum stress range is considered for the stress reversal specimens, the test data falls at the lower limit of the other two levels of minimum stress. However, it was decided to analyze only the data for minimum stress levels other than stress reversal realizing that the stress cycle relationship would be conservative for the stress reversal loading condition.

4.5.2 Effect of Connector Diameter

Comparison of the data for 7/8 inch diameter studs given in Table 7 with that for 3/4 inch diameter studs given in Table 6 reveals that the concrete strength, age of the concrete, and stress levels were similar for both experiments. Figure 13 compares the data for the two sizes of connectors. It is apparent that there is no significant difference between the results for the two sizes of connectors.

Because of the similarity of the data for 3/4 and 7/8 inch diameter connectors, it is pointless to attempt a separate analysis of the data for the larger size connector. Hence all of the stud connector data was treated as one group for analysis. The resulting S-N curves are applicable for stud shear connectors of 7/8 inch diameter or less. It is known that these curves will be somewhat conservative for connectors smaller than 3/4 inch diameter.[7]

There is no evidence that the higher concrete bearing stress which exists with 7/8 inch studs was a significant factor in the cycle life for these connectors. The three 7/8 inch diameter data points for the highest minimum stress level fall near the lower limit of the data. Data on larger connectors is not available. However, the use of connectors larger than 7/8 inch diameter does not seem to be practical at this time.

4.5.3 Analysis of Test Data

Regression curves were computed for each level of minimum stress using the mathematical model given by Eq. 4.5. An analysis of variance showed that the slopes of all three curves were not significantly different even at the 10 percent level. Therefore, the variation in stress range

affected the cycle life at each level of minimum stress to the same degree. The distances between the three curves are significant. They indicate that the stress range and the minimum stress were responsible for variations in the experiment.

From inspection of the plotted data it has already been observed that the effect of minimum stress is largely confined to the condition of stress reversal. The distance between the curves obtained for the +2 ksi and +10 ksi minimum stress levels was not statistically significant, but the distance of either curve from the data for the -6 ksi minimum stress level was significant even at the 1 percent level.

The six supplementary test specimens listed in Table 6 were not included in the preliminary analysis of data because they were tested at a later date and were not part of the factorial experiment. These tests were made to obtain results with a cycle life in the vicinity of two million cycles. The data from these tests was used in obtaining all of the regression curves which follow.

The data for connectors of both sizes having minimum levels of +2 and +10 ksi were analyzed using Eq. 4.5. Thirty-two test results were included in the analysis. The results are summarized in Table 10 as regression curve A. Table 10 includes the linear form of the mathematical model, the experimental constants for both forms of the model, the standard error of estimate, and the coefficient of correlation for each regression curve.

In addition to developing the constants for all test data, separate curves were fitted to the two levels of minimum stress. The

first sample consisted of fifteen points having a minimum stress level of +2 ksi while the second group consisted of seventeen points having a minimum stress level of +10 ksi. Regression curves B and C represent the curves for the +2 ksi and +10 ksi minimum stress levels respectively. The difference between these two curves was significant only at the 5 percent level.

Regression curves A, B, and C are compared in Fig. 14. Curve A is shown as the solid line, curve B as the dotted line, and curve C as the dashed line. It is apparent from Fig. 14 that the extreme values affect the fit of curve A, particularly at the longer cycle life. The mathematical model described by Eq. 4.5 is applicable only to the intermediate range of cycle life between 300,000 and 5,000,000 cycles. Nevertheless, the coefficient of correlation obtained for the overall analysis indicated regression curve A accounted for about 92.8% of the variation in the experiment.

It has been assumed in the preceding analysis that the probability density function which applies to the data is

$$f(x) = \frac{e^{-\frac{(x - \mu)^2}{2\sigma^2}}}{\sigma\sqrt{2\pi}} \qquad (4.11)$$

where μ is the mean, σ is the standard deviation, and x is the log N. It is difficult to check this assumption because of the small number of points at each level of stress range. However, it can be shown that 71.9% of the data points lie within ± one standard error of estimate

from the mean curve of Eq. 4.5, 93.8% lie within ± two standard error of estimate, and 100% lie within ± three standard error of estimate. The theoretical percentages for the above probability density function are 68%, 95%, and 99% respectively. This comparison indicates that the assumption of the log normal distribution is reasonably satisfactory once the true regression curve is found.

The mathematical model given by Eq. 4.4 is superior to the semi-log model in that it better represents the test results over the entire range of data. When the thirty-two data points were evaluated by considering the two levels of minimum stress separately it was found that there was no difference between regression curves for the +2 ksi and +10 ksi levels of minimum stress even at the 10 percent level. The curve for the overall analysis is given as regression curve D and is plotted in Fig. 15. The limits of dispersion are shown for twice the standard error of estimate in Fig. 15. Regression curve D accounted for 93.3 percent of the variation in the experiment which was slightly better than regression curve A. Comparison of values for the standard error of estimate for both curves also reveals that a smaller value was obtained for regression curve D. It is obvious from Fig. 15 that this curve is satisfactory for representing the data over the range from 30,000 to 10,000,000 cycles. It was found that 71.9% of the data fell within ± one standard error of estimate, 96.9% fell within ± two, and 100% fell within ± three.

Other mathematical models involving only stress range and number of cycles to failure were also used to analyze the data. The model given

by Eq. 4.6 was found to result in the smallest difference between curves for the two levels of minimum stress. This analysis is given as regression curve E in Table 10. This model was not able to fit all the test data and accounted for only 81.42 percent of the variation in the experiment.

Mathematical models which included the effect of minimum stress as well as stress range were also used in the analysis of data. The model given by Eq. 4.7 was used to treat the thirty-two test points previously used with other models as well as the twelve stress reversal test results. Regression curves F and G summarize the results of these studies. A comparison of the standard errors of estimate and the coefficients of correlation for these two analyses again indicate that the effect of the minimum stress is important only for the stress reversal data. When the stress reversal data was included in the analysis this model accounted for 80.9 percent of the variation in the experiment. Equations 4.5 and 4.7 (A and F) both accounted for 92.8 percent of the variance when the stress reversal data was omitted.

The mathematical model given by Eq. 4.8 was also used and the regression coefficients obtained with thirty-two and forty-four data points are given in Table 10 as regression curves H and I respectively. This model was superior to the previous one in accounting for the variance when the stress reversal data was included and was applicable to the full range of the test data. Equations 4.4 and 4.8 provided the best fit to the test data from 30,000 to 10,000,000 cycles. The S-N

relationships are summarized by the following equations:

$$S_r = 154.415\ N^{-0.1903} \tag{4.12}$$

$$S_r = 155.487 \left[\frac{N}{(S_{min.})^{-0.4131}} \right]^{-0.1807} \tag{4.13}$$

Equation 4.12 is plotted as the solid line in Fig. 15. Equation 4.13 provided the best fit if the stress reversal data is considered. However, stress range becomes a function of both cycle life and minimum stress. As is apparent from Fig. 16, the inclusion of minimum stress did not improve the fit for the +2 and +10 ksi minimum stress levels.

4.5.4 Effect of Concrete Strength

The concrete strength of the specimens listed in Table 6 varied from 3120 psi to 4730 psi and the age at testing varied from 21 days to 92 days. The variation in concrete strength was in part due to the variation in age, but the specimens listed in Table 6 also represent five different pours of concrete.

In order to determine if the age and strength of the concrete had any effect on cycle life, each group of three specimens with 3/4 inch diameter studs having the same minimum stress and stress range were arranged in the order of increasing cycle life. In each group the residual increase or decrease in concrete strength or age was analyzed. The randomness of the resulting array of residuals for the entire

experiment was then examined with strength and age treated separately. It was found that at the 5 percent level of significance the value of N was not dependent on the value of concrete strength or the age of the concrete.

For the two highest levels of stress range it appeared that concrete age but not concrete strength had some influence on cycle life. Although no statistical evaluation of this could be made with the limited number of specimens available, it seemed that there was a slight tendency for a longer cycle life to be associated with concrete specimens of greater age.

It was concluded that variations in concrete properties had little or no effect on the cycle life of specimens within the range of properties encountered in the experiment. Therefore, these uncontrolled variables could be neglected in the analysis. The concrete strengths were typical of those encountered in bridge deck construction. The test results reported by Thurlimann(13) indicate that specimens tested at 41 days with a concrete strength of 5920 psi had fatigue strengths comparable to specimens tested in this experiment. The fatigue strength of the stud connectors seems to be relatively independent of concrete strength and age when concrete strength exceeds 3000 psi and the age at the time of testing is not less than 21 days.

4.6 CHANNEL SHEAR CONNECTORS

4.6.1 Fatigue Strength of Channels

The data obtained from all tests with 4 inch 5.4 lb. channel connectors is summarized in Table 8. The three groups of specimens were

cast at different times and the concrete strength varied considerably between groups. During the pilot tests the channel connectors were positioned so that the channels faced the applied load. The supplementary test specimens were fabricated at Fritz Laboratory and the throat dimensions of the welds were found to be undersize after testing.

Previous investigations[6,18] had indicated that both the static and fatigue strengths vary with $\sqrt{f'_c}$. The concrete strengths for the three groups of specimens were varied intentionally to further evaluate this effect. Since the effect of concrete strength on the cycle life of full size stud connectors was negligible, it was desirable to evaluate the influence of this variable on the cycle life of full size channel connectors.

The orientation of shear connectors with respect to the forces in the concrete slab affects the behavior of the connector. As a result of previous research,[6] shear connectors are usually oriented with the embedded flange facing the support of simple span beams.[28] During the pilot tests the direction of the connectors was reversed in an effort to obtain minimum cycle life results for the experiment design. In all other tests, the orientation corresponded to that used in bridge construction.

4.6.2 Analysis of Data

The test results for the pilot and main experiment are summarized in Fig. 17 where the connector load in kips per inch is plotted as ordinate on an arithmetic scale versus the number of cycles to failure as abscissa on a log scale. The pilot tests are represented in the plot

by the open and closed dots with crosses. The test data indicates that concrete strength and connector orientation did not adversely affect the cycle life.

It is apparent from Fig. 17 that the stress range rather than the maximum stress is the important variable. The stress reversal data again exhibited longer fatigue life than the other test data.

Since it is common to express connector loads by the load per unit width of channel connector, this was also done in Fig. 17. The curve shown in Fig. 17 was obtained using the mathematical model given by Eq. 4.5. The coefficients for this curve are summarized in Table 11 (J). Also included in Table 11 are the linear form of the empirical equation, the experimental constants for both forms of the mathematical model, the standard error of estimate, and the coefficient of correlation for each regression curve.

The test data was also analyzed using the exponential model given by Eq. 4.4 (K). This curve and the data are plotted in Fig. 18 where both stress range and cycle life are plotted on a log scale. The analysis indicated that there was no difference between the levels of minimum stress for either equation, even at the 10 percent level of significance. The exponential model (Eq. 4.4) appeared to be slightly superior to the semi-log model in that it resulted in a lower value for the standard error of estimate and a higher value of the coefficient of correlation.

In order to extend the analysis to include all available data, the applied loads were expressed in terms of the stress on the throat of the fillet welds. When the four supplementary tests were considered

along with the other nine points, the analyses of the test data was repeated with both Eqs. 4.4 and 4.5. Equation 4.5 (L) is compared with the test data in Fig. 19 with the limits of dispersion taken as twice the standard error of estimate. Equation 4.4 (M) is compared with the test data in Fig. 20. It is apparent that the supplementary tests fall near the lower limit of dispersion. Again, minimum stress was not found to be significant.

All test data was also evaluated using the mathematical models given by Eqs. 4.7 and 4.8. The results of this study are summarized in Table 11 (see N and O). This analysis showed that minimum stress was not able to account for a significant amount of the variation in cycle life.

The exponential models represented by Eqs. 4.4 and 4.8 resulted in the best representation of the data for channel connectors. The inclusion of the minimum stress did not produce a significant improvement in the correlation.

4.6.3 Effect of Concrete Strength

Although the comparison of pilot test data with test data from the main experiment indicated that the effect of concrete strength on the cycle life was small, this variable was studied further.

The mathematical models given by Eqs. 4.9 and 4.10 were used to determine if cycle life was a function of the concrete strength. In these models the $\sqrt{f_c'}$ replaced the minimum stress as one of the terms in the empirical equations. The results of this study are summarized in

Table 11 (P and Q). The analysis confirmed that concrete strength did not significantly influence the cycle life.

The four supplementary test specimens appear to have shorter cycle lives for reasons other than concrete strength. The fractured surfaces were measured to find if weld size was a factor. The throat of the weld on the supplementary specimens was only 74 to 78 percent as large as the throat area for the other specimens.

Table 12 summarizes the measured fracture area for the connector which failed first in each specimen. The third column of this table gives the ratio of nominal throat area of each specimen divided by the average fracture area. The table shows that there is reasonable agreement between the nominal throat area and the fracture surface. The supplemental test specimens were fabricated separately and it was apparent that the weld quality was not as high as that of the other specimens. The variations in weld size encountered in this experiment are probably no greater than the normal variations with hand welding. The possible variations in weld size and quality are greatly reduced in the automatic stud welding process. This is the principal reason why there was greater scatter in the test results for channel connectors.

4.7 CORRELATION WITH PREVIOUS INVESTIGATIONS

4.7.1 Model Beam Tests with Channel Connectors

Eighty-five model beams were tested in fatigue at the University of Illinois from 1944 to 1948.(5) The beams were 3 I 6.5 steel sections with concrete slabs 1-3/4 inches thick by 1'-6" wide. The shear

connectors were channels 1 inch high by 1-1/2 inches long with flange widths of 3/8 inch. Eight of these specimens (F1A through F8A) had channel connectors similar in cross-section to American Standard channels, and the fatigue failures were in the weld similar to those of the present investigation. The mortar strength in these specimens varied from 2220 psi to 4510 psi.

The data from the model beams is compared with the data from this investigation in Fig. 21. An analysis of the test data indicated that the beam tests were not significantly different at the 5 percent level. The regression curve plotted in Fig. 21 corresponds to curve R in Table 11.

The degree of correlation between the results of the two investigations is good when one considers the difference in dimensions of the specimens and connectors involved. It is apparent that the properties of the concrete had little or no influence on the cycle life. Concrete slabs were used in this investigation and the slabs in the University of Illinois tests were mortar.

4.7.2 Results of Transversely Loaded Fillet Welds

Since the primary failure of the channel shear connectors occurred in the fillet welds it is of interest to compare the results of these tests with results obtained in testing coupons with transverse fillet welds.

Figure 22 shows a type of specimen used for fatigue tests of transversely loaded fillet welds. The welds on channel shear connectors

are also stressed in bending, but it has been shown earlier that the bending stress is probably negligible.

The test data for all channel connector tests are plotted in Fig. 23 and compared with the results of tests on transverse fillet welds.[29] Also shown is Eq. 4.5 (S). An analysis of variance showed that there was no difference between curves for the two groups of data even at the 10% level of significance.

The stress reversal data for transverse fillet welds also resulted in slightly longer cycle life as was the case with channel connectors. It is apparent that the fatigue strength of channel shear connectors does not differ significantly from the fatigue strength of transverse fillet welds.

4.7.3 Correlation with Full Scale Beam Tests

The only full scale beam tests were conducted at the University of Texas. The test specimens had 3/4 inch diameter welded stud connectors.[8] In Fig. 24 the regression curves and the limits of dispension given as twice the standard error of estimate are shown for the beam tests and pushout tests. The mean curves are nearly the same for short cycle lives. The beam tests exhibited greater fatigue strength at longer cycle lives.

It was noted previously that the pushout tests yielded conservative results because of the lack of redistribution of load between connectors and the lack of dependable friction forces which can develop in a beam. These observations are confirmed in Fig. 24. The mean curve

of the pushout tests nearly coincides with the lower limit of dispersion for the beam tests for 500,000 to 1,000,000 cycles.

4.8 FATIGUE STRENGTH OF SHEAR CONNECTORS

In the preceding sections the various factors which were thought to have some effect on the cycle life of shear connectors of each of the three types under investigation were considered. The factorial design of the main experiments made it possible to ascertain the relative significance of minimum stress, maximum stress, and stress range. To a lesser extent the effect of uncontrolled variables such as concrete strength and age, orientation of connectors, and size effect were also evaluated. In all comparisons it became obvious that the stress range was by far the most important variable to be considered. Furthermore, it was always conservative to neglect the stress reversal data and utilize the regression curves which resulted from the use of the simple mathematical models of Eqs. 4.4 and 4.5.

Comparing the regression coefficients which result from comparable models in Tables 10 and 11 reveals that the values of coefficients are nearly equal. This indicates that the fatigue strength of both types of shear connectors is nearly the same.

The data from all tests involving studs, channels, and transversely loaded fillet welds are summarized in Figs. 25 and 26. The test data was analyzed using Eqs. 4.4 and 4.5. The following empirical equations were obtained

$$S_r = 49.2876 - 6.2605 \log N \tag{4.13}$$

and

$$S_r = 148.155\ N^{-0.1857} \tag{4.14}$$

The standard errors of estimate were 0.2404 and 0.2384 respectively, while the coefficients of correlation were 0.934 and 0.936. An analysis of variance indicated that there was no difference between the individual regression curves at the 5 percent level.

The test data and Eq. 4.13 are compared in Fig. 25 and the corresponding plot for Eq. 4.14 is given in Fig. 26. It is apparent that the agreement is good for all connectors. The critical parameters for the connectors are the throat areas of the fillet welds and the nominal shear area of the stud. Either Eq. 4.13 or 4.14 is a suitable representation of the S-N curve for flexible connectors.

If stiffer connectors are considered, it is not so much a problem of designing the connectors as it is one of determining the loads to which the connectors are subjected. Redistribution of connector forces is less likely as the flexibility of connectors is decreased.

4.8.1 Effect of Flange Thickness on the Fatigue Strength of Stud Shear Connectors

Table 9 summarizes the data from the tests of studs with various flange thicknesses. The four tests with 8 W 67 and 8 W 31 steel sections were made with a stress range of 12 ksi. This corresponded to the load history of connectors in the main experiment which were attached to 8 W 40 beams.

Static pushout tests involving 1/2, 5/8, and 3/4 inch diameter studs have revealed that a transition in mode of failure takes place for ratios of stud diameter to flange thickness of about 2.7[30] For 3/4 inch diameter studs this transition flange thickness is 0.278 inches. The characteristic failure for flanges thinner than the above value is a pulling out of the connector from the flange while the characteristic failure for thicker flanges is a shear failure of the stud. In the vicinity of the transition flange thickness a combination of the two failure modes may occur. The flange thickness of an 8 W 31 beam is considerably greater than the transition thickness found from static tests.

The flange thickness of an 8 W 31 section corresponds to the minimum thickness of material on which studs would be welded in bridges. Since transition thickness in fatigue may differ from that found in static tests, it is of interest to determine if a reduction in cycle life occurs for the flange thickness of the 8 W 31 section. If the data obtained with the 8 W 31 specimens is compared with the mean curve in Figs. 16 or 17, it will be found that these points fall within the limits of dispersion obtained for the main experiment.

The data for the 8 W 67 pushout tests was not conclusive, for inspection revealed that the connectors were not properly welded. Figure 27 shows the fracture surface of the four connectors on one of the 8 W 67 beams. Apparently the welds on the 8 W 67 sections were made with improper weld settings. Table 13 summarizes the data obtained from the investigation concerning the effect of flange thickness on cycle life.

The eight specimens in the table were tested at the same levels of minimum stress and stress range with the stud-diameter to flange-thickness ratio varying from 0.80 to 1.73. It appears that the higher ratio may have a minor effect on cycle life.

In studying the fractured surface of all of the stud connectors it was found that many welds were not of as good quality as those shown in Fig. 6. However, none were as poor as those shown in Fig. 27. The appearance of the welds on each specimen was compared with their cycle life. Only three specimens (c 3 A, a 4 B, and b 5 C) exhibited poor weld quality which produced a noticeable effect on the cycle life. In some cases welds which appeared to be poor yielded longer cycle life than studs exhibiting a better quality weld.

4.9 EFFECT OF OVERLOADING ON THE FATIGUE STRENGTH OF STUD CONNECTORS

The data obtained from four exploratory tests in which overloads were applied are summarized in Table 9. The notion that overloads have a detrimental effect on cycle life originates from the effect on the concrete around connectors. Once it is realized that concrete properties are of little importance, the possible effect of overloads no longer looms as a serious problem.

The effect of a large number of overloads of the magnitude applied in this test could be evaluated by Miner's Hypothesis[31] or other cumulative damage theories.

As would be expected from the regression curves, an overload stress range of 15 ksi applied for a maximum of about 80 times did not

appreciably reduce the cycle life since the mean cycle life for the overload stress range is about 300,000 cycles. All four overload test results fell within the 95 percent confidence limits of the test data. Specimen No. 4 fell near the lower limit of the data, and all four test points were below the mean curve. Therefore, some cumulative damage may have resulted from the applied overloads. Any attempt to use Miner's Hypothesis or any similar theory fails in this case because the number of overload cycles is extremely small compared to the cycle life at the overload stress range.

The tests indicate that occasional overloads on composite bridges do not endanger the integrity of the shear connection. It seems probable that shear connectors are no more susceptible to damage from overloads than any other welded detail. In fact, the shear connectors in an actual bridge may be even less of a problem than other welded details because of the possibilities of redistribution of forces among connectors and the presence of friction.

5. DESIGN CRITERIA BASED ON TEST RESULTS

5.1 DEVELOPMENT OF A DESIGN CRITERIA

The importance of stress range on the cycle life of shear connectors and welds subjected to repeated loading has been shown by the results of this investigation and the contributions of other investigators.[32,33] Current specifications usually limit the maximum stress as a means of proportioning members and connections subjected to repeated loads. In bridge design, the use of the maximum stress for design is often inconvenient because the relationship between dead load and total stress is not known. In large structures where the dead load stress is high, the range of stress is so small that the design need not be treated as a fatigue problem.

The range of stress on shear connectors does not depend on the size of the structure. The method of construction may significantly affect the magnitude of the range of stress or connectors. Since shored construction is rarely used in bridges, the loading on shear connectors nearly always involves a relatively large range of stress and a relatively low dead load stress. It is of interest to consider the stresses in shear connectors as loads move across a bridge structure. If shored construction is considered, the maximum shear envelope given in Fig. 28 can be easily determined for dead load, live load, and impact. Using

this envelope the maximum shear stresses that connectors would be subjected to can be computed from elastic theory. For the more common case of unshored construction, the dead load supported by the composite section consists only of the wearing surface, curb and railing, and both the maximum shear envelope and the shear connector stresses are largely due to the effect of live load plus impact.

The maximum shear diagram results in a variable spacing of connectors between support and midspan. At midspan the spacing of connectors is based on a maximum shear which is approximately half of the maximum shear at the support.

As a vehicle passes a connector the direction of the shear force on the connector is reversed. At midspan the range of shear is nearly double the maximum shear stress. This reversal occurs to varying degrees at each connector location as shown in Fig. 29. An examination of the fatigue test data in Chapter 4 showed that the maximum stress was different for each level of minimum stress. Obviously the level of minimum stress varies continuously across any bridge as is apparent from Fig. 29. In Chapter 4 it was shown that the range of shear stress is the major controlling variable. Hence, shear connectors should be proportioned on the basis of the range of shear to which they are subjected.

The analysis of data in Chapter 4 revealed that the minimum stress had virtually no effect on the fatigue life of connectors, and for practical purposes the cycle life depends solely on the stress range. Therefore, it is logical that the spacing of connectors should be

proportioned based on stress range by using a diagram similar to Fig. 29. It is necessary only to consider the shear due to live load and impact when constructing the shear range diagram. The dead load connector stress need not be considered in this phase of the design. The spacing of connectors which results from this approach is entirely different from that which results from designing on the basis of the maximum shear diagram.

In addition to having adequate fatigue strength under working loads it is also desirable for the shear connection to be sufficiently strong so that the ultimate strength of the member or structure can be developed. This criterion is followed in the current design procedure through the concept of the useful capacity of shear connectors. This approach is quite conservative in that it intends that the shear connectors remain elastic until the ultimate strength of the member is developed.

Recent research[18] has shown that the ultimate strength of a member can be developed so long as the compressive force in the concrete slab does not exceed the combined ultimate strength of connectors in the shear span. Providing connectors to develop the ultimate strength indirectly provides a means of preventing the maximum stress on connectors from becoming excessive at working loads.

The design concept based on the dual criteria of proportioning shear connectors by the range of shear and providing sufficient shear connectors to develop the flexural ultimate strength is discussed in limited detail in Ref. 34.

5.1.1 Design Provisions for Adequate Fatigue Strength

The actual shear range as shown in Fig. 29 is the result of computing the extreme values of positive shear and negative shear which can develop at each point as a vehicle moves across the span. The upper curve is merely the maximum shear curve shown in Fig. 28 computed for only live load plus impact. However, the curve must be computed for the entire span rather than for half span as shown in Fig. 28. The negative shear curve of Fig. 29 is seldom computed, and the procedure for computing this curve may not be clear to many design engineers.

The magnitude of the negative shear curve is generally less than the magnitude of the positive shear curve at an equal distance from the opposite support of a simple span beam. The two ordinates become more nearly equal for longer spans.

It is possible to avoid computation of the negative shear diagram with little sacrifice in accuracy by considering the positive shear diagrams resulting from traffic moving in both directions on the same member and inverting one of the diagrams. The diagram of Fig. 29 then consists of the positive shear diagram above and below the zero shear axis.

It is of interest to compare the actual range of shear with the range of shear shich results from the two maximum shear curves for vehicles moving in opposite directions. In Fig. 30 such a comparison is made for spans of 50, 70, and 90 feet. It is obvious that the actual range of shear does not differ significantly from the approximation based on the maximum shear diagrams. Hence, for simple span beams the

designer need only compute the maximum shear ordinates due to live load and impact, similar to the procedures now used in proportioning shear connectors. Also, it will be noted in Fig. 30 that the range of shear between support and midspan does not vary greatly from the maximum shear $(V_r)_{max.}$ for these spans. A comparison of the midspan range of shear in Fig. 30 shows that the range of shear becomes more nearly uniform throughout the span as the span length increases.

For a span of 50 feet the actual range of shear at midspan is 80 percent of the maximum range of shear at the support, and the spacing of connectors from support to midspan should increase by 20 percent. The variation in connector spacing from support to midspan for a 90 foot span becomes only 10 percent, and if longer spans are considered the spacing becomes nearly uniform. It is apparent that if the spacing of connectors required at the support were used throughout the span, it would increase the cost very little.

The spacing of shear connectors is obtained by using elastic theory. Formula 1.1 can be written in the following form for the range of shear, V_r. The geometric properties of the composite

$$S_r = \frac{V_r Q}{I} \tag{5.1}$$

section are Q and I which are applicable at each cross-section. S_r is the computed range of horizontal shear stress per inch of member. The variation in the magnitude of S_r throughout the span is apt to be influenced as much by changes in Q/I as by variations in V_r for typical bridges.

The spacing of connectors is then readily obtained by the formula

$$P = \frac{\Sigma\, Z_r}{S_r} \tag{5.2}$$

where S_r is obtained from Eq. 5.1 and $\Sigma\, Z_r$ represents the allowable load for the group of connectors at a cross-section.

The spacing of connectors for fatigue results in a nearly uniform spacing. This is in contrast to the variable spacing which results from current procedures based on the maximum shear. Since dead load shear is usually zero at midspan, the use of the maximum shear as a method of design results in the proportioning of these connectors based on about half of the actual range of shear to which they are subjected by actual loading.

The design procedure for shear connectors in continuous bridges should also consider the range of shear to which all connectors are subjected. In particular, the negative moment region of these members must be designed differently. The concrete slab is often neglected in the negative moment region of continuous beams, and this assumption results in the omission of shear connectors for this region. Such a design does not provide adequate fatigue strength unless there are no forces in the slab which can produce a range of shear on connectors. Obviously this latter condition can only be guaranteed by a discontinuous slab.

5.1.2 Provisions for Ultimate Strength

As was noted, the number of shear connectors should be sufficient to develop the flexural ultimate strength of the member. In order to

ensure that this requirement is met, a limitation on the maximum force acting on shear connectors must be imposed at ultimate moment.

Recent research[18] has shown that the flexural strength of beams can be developed if sufficient connectors are provided to resist the maximum horizontal force which can exist in a slab. This maximum slab force can be readily determined from the fully plastic state of stress which exists at each section of maximum positive bending moment as shown in Fig. 31. Two cases are shown, both of which are encountered in longer spans.

Reference 18 has shown that the maximum horizontal force which must be developed by shear connectors is given by

$$H_1 = A_s F_y \tag{5.3}$$

$$H_2 = 0.85 f_c' bt \tag{5.4}$$

where A_s and F_y are the area and yield point of the steel cross-section and f_c', b, and t are the concrete strength and cross-section dimensions of the concrete slab. The force H_1 comes from Case I while H_2 is derived from Case II (Fig. 31). Obviously, the smaller of the forces H_1 and H_2 must be resisted in any member between the cross-section at which the ultimate moment occurs and any cross-section where the applied moment is zero.

The forces which develop in the negative bending moment region of continuous beams must also be resisted by shear connectors. The largest force which can be developed in this region is the tensile strength of the concrete slab. The concrete slab will crack on nearly

the first cycle of working load which passes across the bridge because of the combined effect of stress due to volumetric changes, bending of the slab to the curvature of the steel beam, and the direct tensile force resulting from anchorage of the slab steel. Reference 34 has suggested that the maximum force which can exist in the cracked slab is the force H_3 which can be developed in the reinforcing steel.

$$H_3 = A_s^r F_y^r \tag{5.5}$$

where A_s^r and F_y^r are the area and yield point of the reinforcing steel in the slab.

In general, then, the number of shear connectors, N_i, which must be provided between any of the cross-sections of maximum and zero moments is given by

$$N_i = \frac{H_i}{\emptyset\, Q_u} \tag{5.6}$$

where H_i is the appropriate horizontal force at the point of maximum moment, Q_u the ultimate connector strength, and $\emptyset$ a load reduction factor. The factor $\emptyset$ is inserted in this equation to ensure that the shear connection is designed with a greater factor of safety than other elements of the member. A value of $\emptyset$ equal to 0.85 has been suggested.[34]

5.1.3 Design Loads for Connectors

Each part of the design criteria requires a set of load values for connectors. Values of the allowable range of load per connector, Z_r, are required for adequate fatigue strength. Also, values for the ultimate strength of connectors, Q_u, are required.

The allowable values for Z_r given in Ref. 34 and listed in Table 14 were developed from the regression curves for each type of connector. It was shown (Fig. 24) that the mean curve from pushout tests nearly coincides with the lower limit of dispersion for beam test results. The values of Z_r for stud connectors were then obtained directly from the mean curve for pushout tests. In the latest revision of the AASHO Specifications[35] allowable stresses based on cycle lives of 100,000, 500,000, and 2,000,000 cycles are used for bridges with different traffic conditions. Therefore, three corresponding stress levels were given for connectors.

Since the pushout test data for full size channel connectors was considerably less than that available for studs and no test results were available on full size beams, the values for channel connectors were obtained from the lower limit of dispersion based on twice the standard error of estimate. Also, model beam test results fell near the lower limit of dispersion, and the quality control on the size and condition of the hand welds used to attach channel connectors is not apt to be as good as that for automatically welded studs.

Since this study has shown that the fatigue strength of flexible connectors is the same, it would be possible to replace Table 14 with a single formula which provides the allowable range of shear that connectors may be subjected to in terms of their required cycle life. Such a relationship is

$$F_r = A_w \quad 148.2\ N^{-0.1857} - \epsilon \qquad (5.7)$$

where F_r is the allowable load in kips per connector, A_w is the area of the weld, N is the design cycle life, and ϵ a correction term to account for the lower fatigue life of channel connectors in beams. For stud connectors $\epsilon = 0$ and for channel shear connectors ϵ may be taken as 2.4 ksi. This value of ϵ is the average difference between the mean curve and the lower limit of dispersion considering the three specified finite lives of 100,000, 500,000, and 2,000,000 cycles. This formula was developed on the basis of the shear stresses on the nominal throat area of the channel fillet welds and the nominal stud cross-sectional area. The allowable range of shear stress can be readily obtained for any desired cycle life.

Formula 5.7 is applicable to stud and channel connectors and might be suitable for other connectors as well. Certain limitations must be placed on the use of Eq. 5.7 if the geometry of a connector were to result in a mode of failure other than through the weld. It has been shown that increasing the size of welds on channel connectors does result in failure through the web.(5) Therefore the use of Eq. 5.7 must be restricted to standard channels with a weld size equal to the minimum channel flange thickness.

The ultimate strength of stud and channel connectors is given by the following expressions:(18)

$$\text{Stud connectors} \quad Q_u = 930\, d^2 \sqrt{f'_c} \tag{5.8}$$

$$\text{Channel connectors} \quad Q_u = 550\,(h + 0.5t)\, w \sqrt{f'_c} \tag{5.9}$$

where Q_u is the ultimate strength of a connector, d is the diameter of a stud, h and t are average flange thickness and web thickness of a

channel, w is the length of a channel connector, and f'_c is the concrete strength. The values for use in design for various sizes of connectors are given in Table 15.

5.1.4 Influence of Effective Slab Width

The values of S_r from Eq. 5.1, H_2 from Eq. 5.4, and H_3 from Eq. 5.5 are all computed on the basis of an effective slab width, b. The value of b is established by Section 1.9.3 of the AASHO Specifications(3) and is not greater than the following for beams having a flange on both sides of the web:

1. One-fourth of the span length of the span,
2. The distance center to center of beams, or
3. Twelve times the least thickness of the slab.

The first requirement is seldom if ever encountered in composite bridge beams. The second is logical and obviously necessary. The third requirement is restrictive and often governs the design. For beams having a flange on only one side similar limitations are given.

A limitation of the slab width is necessary to provide for shear lag. The actual limitations are based in part on engineering judgment and in part on analytical solutions. The limitations on effective width of slab have been developed to safeguard the proportioning of a structure for flexure. The value of b provides a simple design tool which makes allowance for shear lag without considering all of the variables which are involved in the shear lag problem. Generally, the approach is conservative and probably underestimates the part played by the concrete slab in resisting bending.

The shear connector design is affected by the value of b, and the result may not be conservative. Several tests of composite beams have indicated that a greater effective slab width than that specified contributed to the forces on the shear connectors. In cases where the actual slab width exceeds b, the former should be used in designing shear connectors. The existence of a monolithic concrete wearing surface which is neglected in calculating b may result in the value of Q/I being underestimated by as much as 20 percent.

The effective width of the slab for ultimate strength has not been defined because it has not been used directly as a design method. The same effective width should apply for both elastic and ultimate strength design in lieu of evidence to the contrary. Actually the problem of computing b is of less practical importance for ultimate strength because the number of shear connectors provided will develop a compressive force in the concrete slab which has been found to be sufficient to develop a certain flexural ultimate strength.[18] This slab compression force is then a fixed value regardless of the actual slab width.

The effective slab width to be used in the negative moment region is also open to question. It is computed in the same manner as for the positive moment region. For the design of shear connectors it would be desirable to use the center-to-center beam spacing in both the positive and negative moment regions.

5.1.5 Spacing of Connectors

The spacing of shear connectors is governed by the range of shear at each cross-section for fatigue. The ultimate strength provision requires no special arrangement. Previous investigations have shown that extreme precision is not required in spacing connectors to develop the flexural strength of a member.[18] Uniform spacing has been found to be satisfactory for a dead load moment diagram and therefore a uniform spacing would be satisfactory for bridges since the curve of absolute maximum moment for simple span members is essentially parabolic. Any arrangement of connectors which tends to group connectors nearer to the point of zero moment than would occur for uniform spacing would also be satisfactory. The spacing of connectors for fatigue loading is therefore satisfactory for flexural strength. If the number of connectors for a region of a beam is determined from ultimate strength requirements and a uniform spacing of connectors used, this spacing would be satisfactory for the fatigue loading condition as well, since range of shear is nearly uniform.

If the ultimate strength condition governs the design, an important consideration is the ability of the member to support the ultimate load as it moves across the bridge. It is possible to check any cross-section of the span for this condition. When the condition is checked, the force in the concrete slab is not limited to the ultimate strength of the shear connectors except at the point of ultimate moment. Forces in the longitudinal reinforcing steel of the concrete slab may transfer horizontal slab forces to shear connectors on the opposite side of the load provided that such forces do not exceed the value of $A_s^r F_y^r$.

Since a steel section has considerable moment capacity without the concrete slab and because of the wide permissible latitude in shear connector spacing, the truck load which would produce ultimate moment can move onto the bridge without producing a failure of the member by overloading the shear connectors. For the general case the spacing of connectors need not be checked for the passage of the ultimate load. A forthcoming report of a recent investigation will deal with this problem more completely.

5.2 APPLICATION OF THE DESIGN PROCEDURE TO SIMPLE BEAMS

In the design of simple span beams the number and spacing of the shear connectors will probably be controlled by the fatigue requirements so long as the construction is not shored. This represent nearly all of the bridge construction of this type and a comparison of the economic advantages afforded by the use of this approach to design is of interest. The number of shear connectors provided in each bridge will be dependent on the cycle life for which the bridge is to be designed, whereas previously the shear connectors were designed for the same allowable loads regardless of the expected cycle life.

The number of 3/4 inch diameter studs required per beam for a typical bridge shown on Bureau of Public Roads Standard Bridge Plans[28] is compared in Table 16 for the present AASHO design using a factor of safety of 3.0 and for the proposed design for the three different cycle life requirements of the revised specifications. The number of connectors for the proposed design procedure was determined by using only

the maximum shear at the support for obtaining the connector spacing. The proposed procedure always requires fewer connectors than the current design procedure, and the reduction in the number of connectors per beam ranges from about 14.5 to 54.5 percent for the ranges of span and cycle lives indicated.

If the more exact procedure of considering the variation in the range of shear throughout the span is used in computing the number of shear connectors, the requirements can be reduced by 4 to 10%. A slight additional reduction amounting to about 2 percent is possible by considering the change in Q/I which results from the addition of the cover plate near midspan. For short spans (60 feet or less) the use of the more exact procedure for spacing connectors may be justified.

The last column in Table 16 gives the number of connectors required to develop the flexural strength. The ultimate strength requirement is shown to be less than the fatigue requirement for any cycle life, and the number of connectors becomes constant for ultimate moment for all spans longer than about 50 feet. The fatigue requirements for 50 and 60 foot spans will be less than the ultimate strength requirements. The reason for this being that the properties of the concrete slab determine the shear connector requirements and the number of connectors required remains constant as long as the slab thickness or beam spacing remains constant.

5.3 APPLICATION TO CONTINUOUS BEAMS

Continuous composite beams may be designed under the current specifications.[3] Shear connectors are not required in the negative moment region unless the reinforcing steel is considered part of the cross-section. It is common practice to design the beam composite in the positive moment region, and non-composite in the negative moment region. Shear connectors are placed only in the positive moment region. It has been shown by tests of model continuous beams[4] that a member constructed in this manner develops full composite action in the positive moment region and partial composite action in the negative moment region. The measured stresses in the beam without shear connectors in the negative moment region compared favorably with computed stresses based on assuming a fully composite section for positive moment and a non-composite section for negative moment using conventional elastic theory. As far as the overall behavior of the beam in flexure is concerned, it has been generally accepted that the use of shear connectors in the negative moment region is of little value.

The shear connection in the negative moment region has not been examined in the past. The proposed method of designing shear connectors requires that the spacing of connectors be based on the range of shear on the connector because the fatigue failure of connectors is governed by this variable. Therefore, the concrete slab can not be neglected in the negative moment region unless there are no forces acting on it. If there is no expansion joint in the slab at the interior support, the slab bends with the curvature of the beam and tensile forces are

introduced in the reinforcing steel as it is tied at each end by embedment into the positive moment region. It was shown in Ref. 4 that the force developed in the reinforcing steel was sufficient to develop some degree of interaction. Hence, the slab force in the negative moment region must be anchored by shear connectors within this region or it will be transferred to connectors near the points of contraflexure.

For the negative moment region (between dead load points of contraflexure) the design should be based on the maximum horizontal force in the slab produced by live load plus impact. Computations indicate that the slab can be expected to crack in the negative moment regions during the initial loading of the beam. Experiments have verified that the slab cracks initially within the working load range.[36] Hence, the maximum tensile force in the slab can be approximated from the reinforcing steel in the cross-section.

Typical moment and shear diagrams for live load plus impact are shown in Fig. 32 for a three-span continuous beam with spans of 80, 100, and 80 feet.

The horizontal shear per inch of beam is obtained by using Eq. 5.1 with appropriate section properties. The shear connector spacing is then determined from Eq. 5.2 as in the positive moment region.

The shear connector forces in the negative moment region may be produced by both negative and positive moment. In some designs involving odd span ratios it may be necessary to check the magnitude of the shear connector forces for positive bending moment without neglecting the concrete. From studying the influence lines for various span ratios in

two-span, three-span, and four-span members, it appears that this situation will not occur in span ratios of practical importance. However, for span ratios of 10:6:6 in three-span members and span ratios similar to 10:7:7:10 in four-span members, the positive bending moment or interior supports reaches sizeable proportions and this condition may govern the design.

The ultimate strength provisions are equally applicable to simple span and continuous beams. Figure 33 shows the forces which must be developed by connectors for the condition of ultimate moment. Five separate regions of half of the symmetrical three-span beam must be considered, for flexural strength requirements may control in one of them.

In spite of the two requirements which must be considered in designing connectors for the negative moment region, the number of connectors required is usually small. However, the spacing of connectors in most of this region should probably not exceed the maximum spacing of 24 inches now given in the AASHO Specifications. The connectors perform the useful function of limiting the magnitude of movement between slab and beam, and also aid in producing a more desirable distribution of cracks. The flexural conformance of slab and beam should be developed in the negative moment region, and this is best accomplished when the connectors provide a nearly continuous shear connection. It is therefore preferable to arrange the connectors in a closer spacing and reduce the number of stud connectors in a group or the length of channel connectors.

One problem created by this design approach is that the fatigue strength of the tension flange of the steel beam in the negative moment region is reduced by the welding of connectors on the flange. Allowable stresses for tension flanges which have connectors are given in the AASHO Specifications.[35] The reinforcing steel acts with the steel beam and to some extent reduces the top flange stress. However, it may be desirable to reposition shear connectors over the negative support when the moment is highest. The reversed curvature of the slab over the interior support tends to produce sizeable friction forces which cause the slip in this region to be rather small even if shear connectors are not supplied. Also, beam and slab separation is not a problem. It therefore seems feasible to shift some connectors from the region near the support to regions of lower moment. The main function of the shear connectors is to transfer the force in the reinforcing steel into the steel beam between the support and the point of inflection.

The strain in the region without connectors can not be distributed and therefore accumulates in one or more large cracks, the size of the cracks being a function of the unconnected length. Limitations on the permissible length are difficult to establish without the benefit of further research. This permissible unconnected length is also a function of the percentage of longitudinal reinforcing steel in the slab. The possibility of developing fatigue failure of the end shear connectors increases as the unconnected length is increased. Only an arbitrary rule can be provided at this time. For example shear connectors may be omitted in a region equal to the maximum permissible connector spacing on both sides of the interior support. Studies now

in progress should result in a better evaluation of this problem, and it seems probable that the permissible unbonded length can be increased.

5.4 CONSIDERATION OF VOLUMETRIC CHANGES IN THE CONCRETE

The effect of volumetric changes in the concrete must also be considered in the treatment of steel and concrete composite beams. Section 1.2.1 of the AASHO Specifications implies that such effects must be considered in the analysis of composite beams. Since the volumetric changes do not effect the ultimate strength of a member, the ultimate strength requirement for beams is not altered. For bridge beams, one must evaluate their effect on the fatigue strength.

In the fatigue test results which have been reported for composite beams, there was no evidence that the shrinkage and creep of the concrete adversely affected the performance. The correlation of results from beam tests where the effect of changes in the concrete volume was greatest with pushout test results was good. The volumetric change is obviously not a very significant variable. It has been observed that shrinkage of the concrete usually destroys the bond between the concrete and steel, but this is not necessarily serious since the resulting change in the minimum connector stress is not significant for fatigue or ultimate strength.

The maximum horizontal shear force exerted on shear connectors due to live load is in most instances in the opposite direction to the force produced by shrinkage. The effect of creep of the concrete is to reduce the magnitude of the minimum stress on the connector. These

observations and the test results obtained in this investigation indicate that the level of minimum stress on connectors can vary substantially without affecting the fatigue life of the connectors. Hence, the volumetric changes are not critical with regard to connector cycle life.

It has been shown that reasonable correlation can be obtained between measured strains and deflections compared with calculated values for various types of composite beams.[37,38,39,40,41] None of these investigations is concerned with the shear connection but the importance of stresses due to volumetric change for the top and bottom surfaces of the member is shown. The magnitude of stress is small compared to the magnitude of dead load and live load stress. This indicates that the magnitude of the shear connector stress is also small for volumetric changes compared to dead load and live load stress. It is not known how the connector forces resulting from volumetric change of the concrete are distributed. It is doubtful that these changes cause variations of minimum stress which exceed the live load stress ranges.

Initial analytical studies considered that the slab and beam act independently of each other. The concrete slab was assumed to shorten from shrinkage or other volumetric change, and a tensile force was applied at the centroid of the slab to stretch the slab to the same length as the steel beam. A force required to cancel the hypothetical tensile force was then applied to the composite section. Refinements have been introduced such as considering the forces being applied only to the slab and beam sections separately since no external forces are applied and no forces act on the composite section.[38] Another refinement is the

consideration that the forces act at the interface between slab and beam rather than at the centroids of the sections.

It has been pointed out that some of the simplifying assumptions that have been used in the analysis of stresses due to volumetric changes are not valid. The separate section method does not require that plane sections remain plane, but it does require that there be zero slip between the slab and beam. It is tacitly assumed that a somewhat arbitrary shrinkage coefficient can be used in calculating the shrinkage force. More recent studies have shown that the shrinkage force is really a function of the percentage of slab steel.[40] Also, the curing conditions for the top and bottom of the slab are different and the assumptions of a uniform shrinkage of the slab are hardly justified. The methods of analysis and assumptions have resulted in the shrinkage force being concentrated near the end of the beam. This is unlikely in steel-concrete beams because creep of the concrete would tend to redistribute horizontal forces and reduce the force on highly stressed connectors.

It is generally accepted that the shrinkage forces in concrete-concrete composite beams are concentrated near the end of the beam within a length approximately equal to the depth of the beam.[42] This notion is substantiated by the stress distributions which result in zero slip at the interface. This in turn is consistent with constant moment throughout the span.

If slip is small in a steel-concrete composite beam the shear connector stress is extremely small and the horizontal shear transfer

is almost entirely by bond. Although the bond strength may be sufficient to resist volumetric and live load forces as it was in the case of one beam tested by King et al.,[7] generally the slip will not be zero.

It appears that the distribution of the shrinkage forces which accompany slip in a steel-concrete composite beam has not been studied except on a very limited scale.[38] Measurements on a single beam indicated that the force is not concentrated at the ends of the beam but distributed along the length of the span. It was also shown by measurements made by Branson and Ozell[38] that a significant difference in the shrinkage force exists in concrete-concrete beams near the end and at midspan. This was attributed to variations in shrinkage and creep coefficients along the span when there is essentially zero slip.

The forces due to shrinkage in an actual bridge are nearly constant for the various spans because the slab dimensions remain the same. The composite method results in a force of about 330 kips while the separate section method results in a force of about 15 kips. The difference in the magnitude of these forces indicates a need for considerable work in this area.

If the force of 330 kips does exist, it would be impossible to prevent slip between beam and slab when flexible connectors are used. Once slip takes place, the load will be redistributed nearly equally to the connectors. The stiffness of the shear connection will then determine the magnitude of the force. If the force computed by the separate action method were more nearly correct, this force could be developed in a length equal to the depth of the beam from the end. In either case,

it is unlikely that the minimum stress level would ever exceed the highest level of minimum stress used in the main experiments.

The beam tests that have been made[7,8] include the effect of the shrinkage forces in the test results and these test results exhibit larger cycle life than the pushout results, indicating that the shrinkage forces are inconsequential and not a significant factor in the fatigue strength of connectors.

One final important aspect of shrinkage forces is that the effect of shrinkage of the slab produces tensile holddown forces in the end connectors of a composite beam. This may explain why end connectors fail first in beam tests even though theoretical analysis[43] and actual measurements[44] during tests indicate that shear connectors located approximately twice the depth of the member from the end are stressed higher.

5.5 COMPARISON OF PROPOSED DESIGN PROCEDURE WITH EXISTING SPECIFICATIONS

In the previous sections of this chapter, the proposed design procedure for shear connectors has been compared with the AASHO Specifications. The proposed procedure could replace existing provisions for the design of shear connectors in the AASHO Specifications without requiring any drastic changes in other sections. The established methods for proportioning the composite cross-section are in harmony with the proposed procedure for designing the shear connectors. The designer would not be required to make additional or more elaborate calculations of stresses than are now required.

There is not now in existence in an adopted specification a design procedure for shear connectors similar to the one proposed in this report. In general, other specifications use an allowable stress design for shear connectors based on the maximum stress determined by elastic theory. The allowable stress values specified are sufficiently low so that the loss of interaction is slight and fatigue failure of connectors is precluded by the small magnitude of the slip and the small range of shear on the connectors. Neither ultimate strength design nor a direct consideration of the fatigue strength of connectors forms a part of current design procedures.

The German Specifications DIN 4239 and DIN 1078 [45,46] include what are perhaps the most elaborate provisions for the design of composite steel-concrete beams. The latter specification pertains to the design of bridges while the former covers the design of other structures. The German Specifications differ from specifications for composite structures used in this country in that they are based entirely on elastic concepts whereas a consideration of the conditions existing at ultimate load have directly or indirectly influenced the specifications in the United States.

Strict adherence to elastic considerations leads to extensive calculations in which stresses must be computed for concrete shrinkage and creep, temperature change, shoring manipulation, and differential temperatures between slab surfaces. Jacking supports and prestressing of the concrete slab in continuous beams to control stresses in the negative moment region are used rather extensively.

Shear connector forces are also treated elastically and the design of connectors is based on limiting the concrete bearing pressure and steel stresses within the parts of the connector. With the view that the shrinkage, creep, and temperature changes or the use of shoring does not affect the ultimate strength of the member, the approach has been greatly simplified in U. S. Specifications. Experience to date indicates that this has been largely successful. However, it may be that some of the difficulties encountered with the deterioration of the concrete on bridge decks are influenced by insufficient attention to the computation of actual stresses.

There are some aspects of the German Specifications which are of interest in relation to the proposed shear connector design procedure. The effective width of tee beams is not restricted by a function of the slab thickness and depends only on the spacing of beams and a function of span length. A lower value of the modular ratio than that specified in the AASHO Specifications is used in proportioning the cross-section. These two factors result in the shear connectors being required to resist larger horizontal shear forces.

Experience in this country does not seem to warrant the placing of so high a value on the horizontal shear forces. First of all, the concrete slab cracks even in the positive moment regions due to shrinkage, and for small loads it may not be stressed in compression as assumed in design. Secondly, the more flexible shear connection which is used in this country does not produce slab stresses as high as those which may be produced by the stiffer shear connection which is in common use in

Germany. The development of shrinkage forces requires a particularly heavy shear connection near the ends of simple span beams according to the German Specifications.[45] The spacing of shear connectors is also restricted to three times the slab thickness. When all provisions of the specification are met there is little or no loss of interaction in a beam.

A much greater interest and concern for the design of continuous beams is reflected in the German Specifications. Their efforts have been directed toward creation of a fully composite beam whereas the practice in this country has been toward beams composite only in the positive moment region. Our approach is concerned with controlling the distribution of cracking rather than attempting to prevent cracking. Elastic theory must be more strictly adhered to if one attempts to prevent cracking.

The proposed design prcedure for shear connectors should provide a better distribution of cracking in continuous beams. It seems to be entirely within the realm of possibility to design continuous composite bridges successfully without resorting to methods of preventing the cracks from forming. This is certainly the more economical approach for continuous bridges.

The British are developing a new code of practice for bridges.[47] In draft form this includes a design procedure for shear connectors similar to that suggested in this report. Other provisions will follow closely the code of practice in current use for buildings.[48] This latter specification contains elastic design requirements related to

the development of the shrinkage force by heavy shear connectors near the support and restrictions on shear connector loads based on limiting the magnitude of the concrete stresses near shear connectors.

The proposed British Code of Practice for bridges currently requires that continuous beams have shear connectors in the negative moment region.[47] The design of shear connectors will undoubtedly be based on the fatigue and ultimate strengths of connectors.

The similarities and contrasts between the various specifications suggest some fruitful areas for future research which are discussed in Chapter 6.

6. FUTURE STUDIES OF COMPOSITE BRIDGES

6.1 GENERAL CONSIDERATIONS

It is now apparent that the cycle life of shear connectors can be formulated as a function of the range of stress on the weld area. The use of range of stress necessarily entailed the abandonment of the current approach to fatigue design based on the level of maximum stress. The test data and analysis show that maximum stress can not be used conveniently and was not the governing factor to be considered.

The use of range of shear as the governing design criterion resulted in the establishment of an allowable shear stress range which could be used between the limits of complete stress reversal and the upper limit of practicality with regard to the minimum stress for loading in one direction. This approach becomes conservative for the stress reversal condition, but this loading condition is seldom the controlling situation in problems of interest to the structural engineer. The procedure developed for the design of shear connectors is rational yet simple, and it could be considered for the treatment of the fatigue design of other details.

The design of composite beams by the proposed method has revealed the need for research in the following areas:

1. Fatigue test data on other types of shear connectors

in current use are needed to verify the applicability of the proposed allowable weld stresses. For example, test results on full scale beams with channel connectors should be made to compare with the results of this investigation.

2. A study of the influence of concrete properties and slab reinforcement on the effective width of the concrete slab to be used in shear connector design should be made.

3. The problem of shrinkage stresses should be investigated to determine the magnitude of such stresses and the distribution of connector forces as well as the resulting stresses in the reinforcing steel and concrete slab.

6.2 NEGATIVE MOMENT REGIONS OF CONTINUOUS BEAMS

There is a need for further evaluation of the behavior of the shear connection in negative moment regions. The recommendations which have been made in this report are not in harmony with design practice in most of the fifty states. As mentioned in Chapter 5, the use of shear connectors in the negative moment region presents some problems which should be resolved. The validity of the recommendations must be assessed from the theoretical as well as the practical point of view.

Specific attention should be given to the following problems with regard to the negative moment regions:

1. Cracking of the slab should be controlled by the proper percentage of longitudinal reinforcing steel and shear connection design. Design rules pertaining to these two factors should be developed in a rational manner. In particular the maximum spacing of connectors in this region requires further study.

2. The optimum cross-section geometry should be studied. The amount of slab reinforcing steel which should be considered effective for the design of shear connectors requires study.

3. The feasibility and economic advantages of prestressing the concrete slab should be investigated.

6.3 BEHAVIOR OF SHEAR CONNECTORS

The results of this investigation appear to be conservative compared to beam test results[8] and results obtained in other tests involving pushout specimens.[49,50] Factors which tend to improve the fatigue behavior apparently existed in the other investigations. Friction forces which restrain slab separation and movement are probably the source of the improved fatigue behavior. However, the dependability of these forces in actual bridge decks is questionable. Before the allowable shear connector loads given in this report can be safely exceeded, this problem should be resolved and the following items should be investigated:

1. The distribution of load among connectors of varying degrees of rigidity should be studied. This bears on the limitations which might be advisable for the number and size of connectors at a cross-section as well as the use of connectors which are more rigid than those that have been tested.

2. The cumulative damage aspect of loading on connectors should be considered. Tests involving typical load spectrums should be made. Observations indicate that rest periods during which the overstressed concrete areas recover may improve fatigue life.

3. The fatigue behavior of shear connectors in light-weight concrete should be determined. This has been done for stud connectors,[49] but the greater susceptibility of lightweight concrete to tensile cracking makes it desirable to test channel connectors also.

6.4 COMPOSITE BEAM DESIGNS

Many of the possible areas of research which have been mentioned as having bearing on the proportioning and behavior of shear connectors may in fact be of great importance in the behavior of the member. One has the feeling particularily when comparing the German and AASHO Specifications, that the design of composite beams has become grossly simplified in the United States. Any problems which exist with composite

construction in this country may have their origin in the gap which exists between these two approaches to design.

The design of composite bridges by use of the familiar tee beam concept is now the only approach to design provided for in our specifications. It has been suggested that composite bridges be designed by orthotropic plate theory.[51,52] This concept may deserve more attention than it has received. The problems of lateral load distribution and the effect of transverse bending of the slab are revealed in a new light by this treatment of the problem.

Regardless of whether bridges are designed by the tee beam concept or orthotropic plate theory, the distribution of moments in a continuous structure having incomplete interaction is of some importance in design. It has been shown by Sattler[53] that the magnitude of the extreme moments in continuous beams is affected considerably by the slip between slab and beam. This problem has been largely ignored because fully composite structures have not been designed to any great extent. Undoubtedly the designer of major fully continuous structures is in need of guidance in the analysis and design of such structures. However, there does not appear to be a sufficiently comprehensive publication available on the subject.

7. SUMMARY AND CONCLUSIONS

A study has been made of the behavior of stud and channel shear connectors subjected to constant cycle fatigue loading. The main purpose of the investigation was to ascertain the governing factors affecting the fatigue life of these two widely used types of connectors for steel-concrete composite bridge beams. The investigation also included the development of a design procedure for shear connectors based on the experimental data.

Factorial experiments were designed for the evaluation of 3/4 and 7/8 inch diameter studs and 4 inch 5.4 lb. channel connectors. The experiments were similar for each type of connector and involved the testing of specimens at three levels of minimum stress and three levels of maximum stress or stress range at each level of minimum stress. A special pushout specimen was developed for the experimental program, and a total of 68 specimens were tested. The cycle life of specimens varied from 30,000 to 10,000,000 cycles and both types of connectors exhibited finite lives within this range.

The design procedure developed from this investigation requires that shear connectors be proportioned for the range of shear stress to which connectors located at points along the span are subjected when moving loads across the bridge. A further requirement is that the shear connection be made adequate to develop the flexural strength of the composite section. Separate allowable connector loads for each of the two design requirements are given.

The current design procedure based on allowable maximum shear connector stresses can be replaced by the proposed method in Section 9 of the AASHO Specifications for design without making changes in other design provisions. This would result in a substantial saving in the number of connectors required as well as simplification of the design, detailing, inspection, and fabrication of members. The procedure may be used for all types of mechanical shear connectors, and it is applicable to simple span members as well as continuous beams.

The specific conclusions from the investigation are summarized as follows:

1. The governing factor which affects the fatigue life of shear connectors was found to be the stress range. The magnitude of either the minimum or the maximum stress was not found to be significant except in the case of stress reversal.

2. A mathematical model expressing the stress range as an exponential function of the cycle life was found to provide the best fit for the test data. An analysis of variance indicated that minimum stress was a significant variable only for the stress reversal condition. The maximum stress did not serve to correlate the data.

3. The age or strength of the concrete was not significant with regard to the cycle life of any of the connectors investigated.

4. For a given stress range it was found in all cases that stress reversal specimens exhibited a longer cycle life than specimens subjected to loading in one direction. Therefore, the allowable stress range obtained by neglecting stress reversal is conservative for the stress reversal condition.

5. The fatigue strength of the 3/4 and 7/8 inch diameter studs was not significantly different and a single relationship was found to be applicable to both sizes of studs.

6. The fatigue strength of channel connectors was found to be a function of the stress range on the throat of the fillet welds. No significant difference was found between the test results with channel connectors and the results of tests of tensile specimens with transversely loaded fillet welds.

7. The fatigue strength of all three types of connectors was found to be a function of the stress range on the nominal area of the weld. The same S-N relationship was applicable for the data from all three types of connectors. The mean curve for all data was found to coincide with the lower limit of dispersion obtained in tests of full size beams with 3/4 inch studs. Hence, the mean curve could be used in a design. Since full size beam tests were not available with channel connectors, the lower limit of dispersion was recommended for the design of channel connectors.

APPENDIX A

NOMENCLATURE

1. SYMBOLS

A_s	area of the steel beam and cover plates
A_s^r	area of the longitudinal reinforcing steel in the slab
A_w	nominal area of the weld by which a shear connector is attached to the flange of the beam
b	the effective width of the concrete slab of the composite beam
d	diameter of a stud shear connector
E_c	modulus of elasticity of concrete
E_s	modulus of elasticity of steel
F_r	allowable shear connector range of load in kips
F_y	yield stress of the steel beam
F_y^r	yield stress of the reinforcing steel
f_c'	compressive strength of the concrete
h	flange thickness of a channel shear connector
H_1, H_2, H_3, H_i	limiting values of the compressive force in the concrete slab at ultimate load
I	moment of inertia of the composite beam
n	modular ratio equal to E_s divided by E_c
N	number of cycles to failure

ΣZ_r	allowable range of load for a group of shear connectors at a cross-section
ϵ	a correction factor for obtaining an allowable range of loading for shear connectors
μ	mean value
θ_A, θ_B	slope of shear connector at points A and B in elastic foundation analysis
σ	standard deviation
$\emptyset$	reduction factor for the ultimate strength of shear connectors

2. GLOSSARY

bond	Shear strength of the joint between steel beam and concrete slab resulting from natural adhesion of the two materials.
friction	Shear strength of the joint between steel beam and concrete slab produced by vertical forces between the two elements and the coefficient of friction of the interface surface.
flexible connector	A mechanical shear connector which transfers horizontal shear forces as a result of a measurable movement of the slab with respect to the beam and exhibits a load-slip curve with considerable ductility.
loss of interaction	The decrease in the effectiveness of the concrete slab which results from the difference between the actual stress distribution and the theoretical stress distribution based on zero slip which occurs in the bending of a composite beam.

pushout specimen	A specimen consisting of a small segment of slab and beam which can be used for evaluation of a shear connector whenever the performance of the connector is not affected by the stress distribution in the slab.
ultimate strength	The ultimate moment capacity of the cross-section as it is used in reinforced concrete design, redistribution of moments is not implied by this term.
uplift	The vertical separation of slab and beam which occurs whenever a horizontal slip movement takes place.
useful capacity	The load at which the load-slip curve for a flexible shear connector begins to become non-linear due to yielding of the connector.

APPENDIX B

BEAMS ON ELASTIC FOUNDATION

In Ref. 5 a channel shear connector was analyzed theoretically by a modification of the theory for a semi-infinite beam on an elastic foundation. The most important forces acting on a connector were assumed to be the shear load, V, the bending moment, M_o, at the flange of the beam, and the foundation pressure. The shape of the connector resulted in the assumption that the flange welded to the beam was infinitely stiff ($I = \infty$) while the web stiffness was taken as the EI of a unit width of the web. The web portion of the connector was considered to be infinite in length. The foundation constants for the two portions of the connector were derived from the load-slip curves of pushout tests of full-scale connectors, and the resulting solution was considered to be valid up to yielding of the connector material.

The geometry of a stud connector requires that this type of connector be treated as a beam of finite length. A relation between the deflection of the connector, y, at the welded end with respect to the elastic medium and the shear load, V, is also available from load-slip curves from pushout tests. The four boundary conditions for the analysis are stated in Fig. 10.

The differential equation for a beam on an elastic foundation is given by

$$\frac{E\ I\ d^4_y}{d^4_x} = -\ k\ y + q \qquad (B.1)$$

where E and I are properties of the beam, k is the foundation constant, y is the beam deflection, and q is the distributed loading on the beam. A solution of the corresponding homogeneous equation is

$$y = e^{\lambda x}(C_1 \cos \lambda x + C_2 \sin \lambda x) + e^{-\lambda x}(C_3 \cos \lambda x + C_4 \sin \lambda x) \qquad (B.2)$$

$$\text{where} \quad \lambda = \sqrt[4]{\frac{k}{4\ E\ I}} \qquad (B.3)$$

A suitable value of k is available from experimental investigations in which the modulus of elasticity of concrete in triaxial compression was obtained.[24] The assumed value for k when 3/4 inch diameter studs are used was 1,500,000 psi. The values of E and I for this size of connector are E = 29,000,000 psi and I = 0.01553 in.^{4}. The resulting value of λ becomes 0.9553 in.$^{-1}$.

The solution of an infinite beam with a concentrated load, P, at some point is considered first. If $x = \infty$, $y = 0$ and the constants C_1 and C_2 must be zero in Eq. B.2. From the condition of symmetry under a concentrated load the constants C_3 and C_4 of Eq. B.2 are found to be equal and the following equation results:

$$y = C\ e^{-\lambda x}(\text{Cos}\ \lambda\ x + \sin\ \lambda\ x) \qquad (B.4)$$

From equilibrium of vertical forces,

$$P = 2 \int_{o}^{\infty} k\, y\, d\, x \tag{B.5}$$

By substituting of Eq. B.4 in Eq. B.5 and performing the integration, $C = P\,\lambda/2\,k$ is obtained and the following equation is obtained for a concentrated load on an infinite beam

$$y = \frac{P\,\lambda}{2\,k}\, e^{-\lambda x}\, (\mathrm{Cos}\,\lambda\, x + \sin\lambda\, x) \tag{B.6}$$

In a similar manner an expression for a concentrated moment, M_o, on an infinite beam is obtained as

$$y = \frac{M_o\,\lambda^2}{k}\, e^{-\lambda x}\, \sin\lambda x \tag{B.7}$$

Equations B.6 and B.7 can be differentiated to obtain expressions for slope, moment, and shear of the infinite beam. The eight equations which are thus obtained are useful in the solution of beams of finite length by superposition of forces.

The solution of a finite beam is obtained by solving first a beam of infinite or semi-infinite length having the same loading condition. From this solution values of deflection, slope, moment, and shear at points corresponding to the ends of the finite beam can be computed. In general the values thus obtained will not fit the boundary conditions of the finite beam. However, the end conditions can be adjusted to fit the boundary conditions by the addition of a vertical force at each end and a concentrated moment at each end. The solution

of the finite beam then consists of the superposition of the effect of the actual loading plus the effect of the four end condition forces.

In Fig. B1 the finite beam representing a 3/4 inch diameter stud of length 4 inches is given. The deflection at end A has been found to be 0.005 inches for an applied shear load of 7500 lbs. The boundary condition for the deflection at A results in the equation

$$y_A + \frac{P_A \lambda}{2\,k} + \frac{P_B \lambda\, e^{-\lambda x}(\cos \lambda x + \sin \lambda x)}{2\,k} + 0 + \frac{M_B \lambda^2}{k}\, e^{-\lambda x} \sin \lambda x = 0.005$$

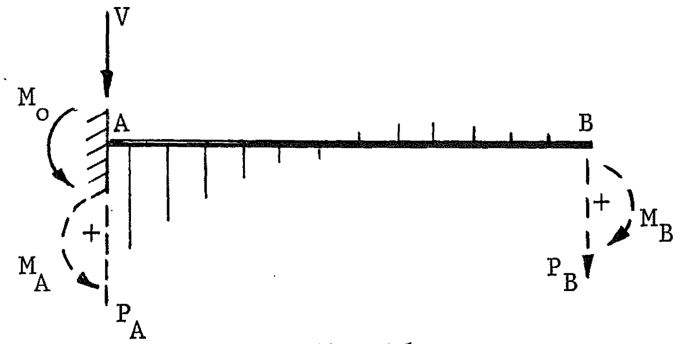


Fig. B1

The term y_A represents the deflection at A due to the shear force of V = 7500 lbs. which is easily obtained from the solution of a semi-infinite beam with a concentrated load at end A. The second and third terms are due to the concentrated end conditioning forces P_A and P_B. The deflection due to an end conditioning moment at A is zero, and the last term is due to the end conditioning moment at B. The deflections on the left side of the equation must be equal to the actual end condition deflection of 0.005 inches. Similar equations can be obtained for each of the other three end conditions. The four equations can then be solved simultaneously for the end conditioning forces P_A, P_B, M_A, and M_B.

The deflection, slope, moment, or shear at any point along the beam can now be computed by the sum of the effects of V, P_A, P_B, M_A, and M_B at that point. Since the solution is primarily of interest as a qualitative solution rather than a quantative one, the results are given in Fig. B2 where Y, θ, M, and V are plotted to scale and the maximum values are indicated as a function of the shear force V.

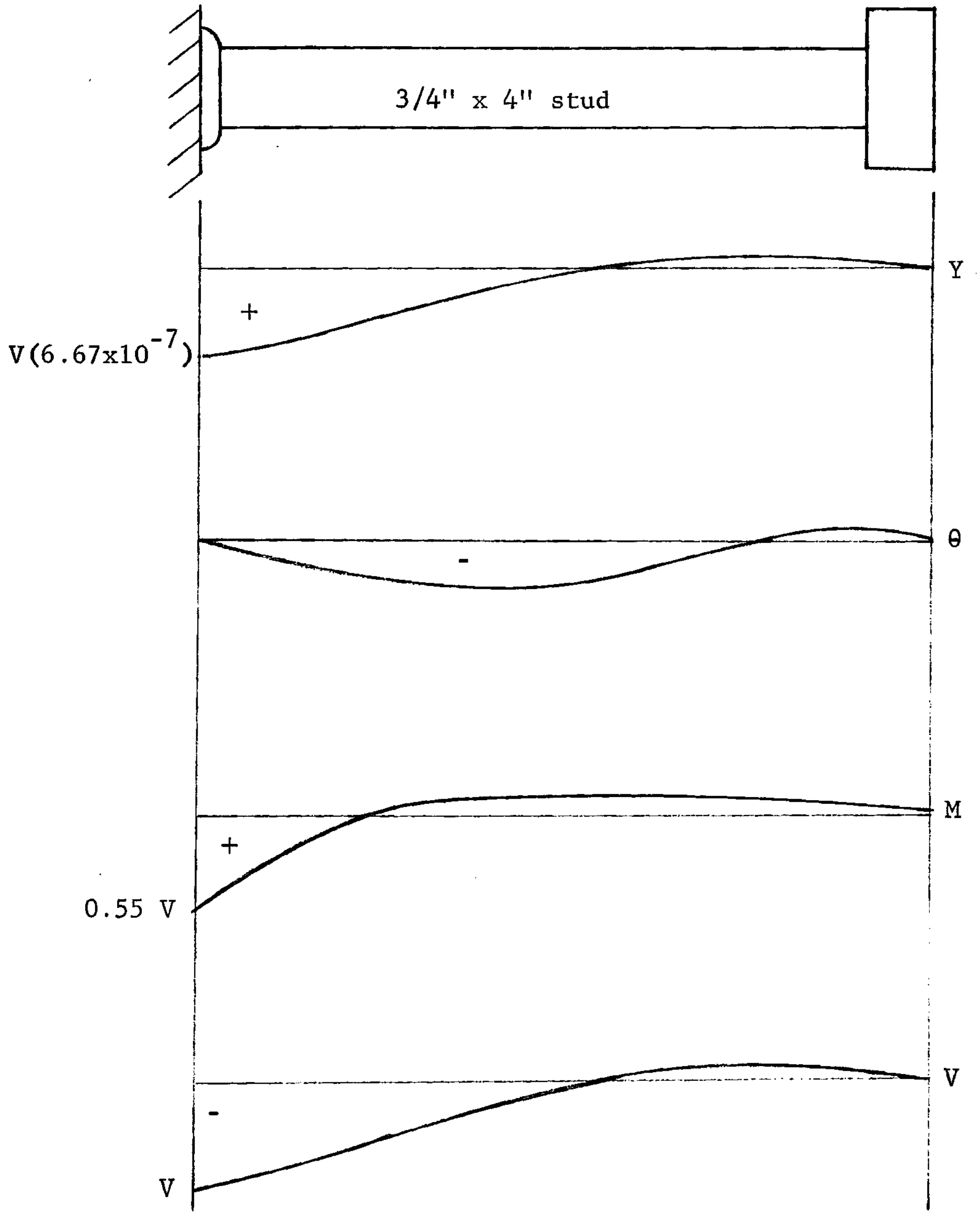


Fig. B2

TABLE 1 SUMMARY OF EARLIER TEST RESULTS

Specimen No.	Ref. No.	Type of Specimen	Min. Shear Stress (ksi)	Max. Shear Stress (ksi)	Cycles to Failure (kc.)	Remarks
4	13	1/2" Ø Bent Stud, Pushout	2.9	22.3	223.2	Stud Fracture
5	13	1/2" Ø Bent Stud, Pushout	2.2	17.8	134.2	Stud Fracture
6	13	1/2" Ø Bent Stud, Pushout	2.2	17.8	261.0	Stud Fracture
7	13	1/2" Ø Bent Stud, Pushout	1.9	15.6	1,748.0	Stud Fracture
9	13	3/4" Ø Headed Stud, Pushout	2.8	22.3	169.4	Stud Fracture
10	13	3/4" Ø Headed Stud, Pushout	1.7	15.6	474.0	Stud Fracture
Bridge	12	1/2" Ø Bent Stud, Beam	1.85	15.7	256.8	No Failure
B4	14	1/2" Ø Bent Stud, Beam	1.50	21.0	619.0	No Failure
B4	14	1/2" Ø Bent Stud, Beam	1.50	24.1	122.4	No Failure
BF-A	7	1/2" Ø Headed Stud, Beam	5.73	23.9	50.3	Initial Failure
BF-B	7	1/2" Ø Headed Stud, Beam	4.90	32.6	55.4	Initial Failure
BF-C	7	1/2" Ø Headed Stud, Beam	3.6	27.7	78.0	Initial Failure
BF-D	7	1/2" Ø Headed Stud, Beam	2.16	20.1	820.0	Initial Failure
BF-1	7	1/2" Ø Headed Stud, Beam	1.88	22.2	490.0	Initial Failure
BF-2	7	1/2" Ø Headed Stud, Beam	1.88	22.2	480.0	Initial Failure
BF-3	7	1/2" Ø Headed Stud, Beam	1.88	19.4	980.0	Initial Failure
BF-4	7	1/2" Ø Headed Stud, Beam	2.19	19.4	3,315.0	No Failure
BF-5	7	1/2" Ø Headed Stud, Beam	1.88	22.6	140.0	Initial Failure
BF-6	7	1/2" Ø Headed Stud, Beam	1.88	21.1	168.5	Initial Failure
BF-7	7	1/2" Ø Headed Stud, Beam	1.88	20.3	450.0	Initial Failure
BF-8	7	1/2" Ø Headed Stud, Beam	1.88	19.5	1,445.0	Initial Failure
1-A	8	3/4" Ø Headed Stud, Beam	18.4	1.9	70.0	Initial Failure
1-B	8	3/4" Ø Headed Stud, Beam	16.5	2.0	1,620.0	Initial Failure
1-C	8	3/4" Ø Headed Stud, Beam	20.3	2.5	205.0	Initial Failure
1-D	8	3/4" Ø Headed Stud, Beam	13.9	1.7	1,400.0	Initial Failure
2-A	8	3/4" Ø Headed Stud, Beam	11.9	1.4	1,500.0	Initial Failure
2-B	8	3/4" Ø Headed Stud, Beam	16.5	2.0	600.0	Initial Failure
2-C	8	3/4" Ø Headed Stud, Beam	20.3	2.5	90.0	Initial Failure

TABLE 2 MATERIALS TEST DATA

Specimen	Yield	Tensile Strength (psi)	% Elongation in 2 inches (psi)	% Reduction in Area	Remarks
Longitudinal specimens from web of 4 inch channel	45,100	65,080	27.3	59.9	
	38,500	64,520	31.0	59.0	
3/4 inch studs for Phase I & II	50,000	61,450	38.5	64.4	
	48,650	60,950	--	--	Failure in Base Metal
	47,800	57,560	41.0	66.1	
	48,200	61,170	41.0	61.6	
3/4 inch studs for supplementary specimens	56,750	70,270	24.5	52.9	
	57,500	71,400	24.0	51.8	
	57,500	70,470	25.5	52.3	
	54,800	69,530	25.5	51.5	
7/8 inch studs	54,800	61,190	--	--	Failure in Base Metal
	59,100	64,240	--	--	Failure in Base Metal
	58,900	68,540	--	--	Failure in Base Metal
	58,800	66,890	--	--	Failure in Base Metal

Notes:

1. Specimens were tested to 1.5 percent strain at a strain rate of 0.025 inches per minute and thereafter at a strain rate of 0.050 inches per minute.

2. The static yield point is reported for the channel web specimens while the 0.2 percent offset value is reported for all stud connectors.

3. All stud specimens were 4 inch long connectors welded to a 4" x 5" x 1/2" plate of ASTM A36 steel except the 3/4" studs for Phase I & II. The latter specimens were 4" x 4" plates cut from the flange of an extra 8 W 40 test beam.

4. The failure of 7/8" studs in the base metal was caused by excessive bending of the plate. The failure of one 3/4" specimen in this manner was due to a combination of plate bending, stud welded out of square with the plate and porosity in the weld.

TABLE 3 CONCRETE MIX USED FOR ALL SPECIMENS

Material	Weight per cubic yard
Cement	517 lbs.
Sand	1362 lbs.
Coarse Aggregate (SSD) (1 inch maximum)	1800 lbs.
Water	254 lbs.

(Slump of concrete was maintained at 3-1/2 inches)

TABLE 4 MILL TEST REPORT FOR STEEL MATERIALS

Section	Yield Point (psi)	Tensile Strength (psi)	Elongation in 8 inches (%)	Chemical Analysis C	Mn	P	S
8 W 40	41,320	63,620	30.5	0.21	0.59	0.014	0.030
8 W 67	37,370	62,200	31.8	0.20	0.64	0.011	0.022
4 in. 5.4 lb. channel	*	*	*	0.31	0.59	0.025	0.022
#4 bars	54,200	76,700	16.3	0.37	0.53	0.023	0.044
#5 bars	51,200	84,700	20.0	0.34	0.51	0.017	0.045

*See Table 2

TABLE 5 DESIGN OF MAIN EXPERIMENT

Minimum Stress \ Maximum Stress	S_A	S_B	S_C	S_D	S_E
S_a	a 1 A d 1 D e 1 G	a 2 A d 2 D e 2 G	a 3 A d 3 D e 3 G		
S_b		a 2 B d 2 E e 2 H	a 3 B d 3 E e 3 H	a 4 B d 4 E e 4 H	
S_c			a 3 C d 3 F e 3 I	a 4 C d 4 F e 4 I	a 5 C d 5 F e 5 I

Minimum Stress \ Stress Range	S_A'	S_B'	S_C'	S_D'	S_E'
S_a			a 1 A d 1 D e 1 G	a 2 A d 2 D e 2 G	a 3 A d 3 D e 3 G
S_b		a 2 B d 2 E e 2 H	a 3 B d 3 E e 3 H	a 4 B d 4 E e 4 H	
S_c	a 3 C d 3 F e 3 I	a 4 C d 4 F e 4 I	a 5 C d 5 F e 5 I		

TABLE 6 SUMMARY OF DATA FOR 3/4-INCH DIAMETER STUDS

Specimen	Test Load Max. (kips)	Test Load Min. (kips)	Shear Stress $S_{max.}$ (ksi)	Shear Stress $S_{min.}$ (ksi)	S_r (ksi)	N (kc.)	f'_c (psi)	Age (days)
P1	53.8	17.7	30	10	20	27.9	3990	21
P2	42.7	17.7	24	10	14	383.6	3920	24
			PHASE I MAIN EXPERIMENT					
a 1 A	17.7	-10.6	10	- 6	16	1,587.4	3690	75
b 1 A	17.7	-10.6	10	- 6	16	1,975.2	3730	57
c 1 A	17.7	-10.6	10	- 6	16	2,557.5	3830	65
a 2 A	24.8	-10.6	14	- 6	20	104.6	3670	56
b 2 A	24.8	-10.6	14	- 6	20	104.8	4730	85
c 2 A	24.8	-10.6	14	- 6	20	171.1	3890	74
a 3 A	31.8	-10.6	18	- 6	24	106.5	4380	86
b 3 A	31.8	-10.6	18	- 6	24	85.5	3750	71
c 3 A	31.8	-10.6	18	- 6	24	30.6	3820	53
a 2 B	24.8	3.54	14	2	12	897.3	4200	30
b 2 B	24.8	3.54	14	2	12	565.3	4390	48
c 2 B	24.8	3.54	14	2	12	551.1	4410	62
a 3 B	31.8	3.54	18	2	16	139.4	4490	42
b 3 B	31.8	3.54	18	2	16	114.7	4450	49
c 3 B	31.8	3.54	18	2	16	199.5	4290	62
a 4 B	38.9	3.54	22	2	20	41.5	4790	41
b 4 B	38.9	3.54	22	2	20	50.7	4310	48
c 4 B	38.9	3.54	22	2	20	58.7	4490	58
a 3 C	31.8	17.7	18	10	8	7,481.1	4380	34
b 3 C	31.8	17.7	18	10	8	10,275.9	4430	51
c 3 C	31.8	17.7	18	10	8	5,091.2	4340	64
a 4 C	38.9	17.7	22	10	12	798.0	4630	28
b 4 C	38.9	17.7	22	10	12	1,215.4	4370	44
c 4 C	38.9	17.7	22	10	12	1,010.4	4400	59
a 5 C	46.0	17.7	26	10	16	335.8	4420	43
b 5 C	46.0	17.7	26	10	16	99.2	4580	51
c 5 C	46.0	17.7	26	10	16	197.0	4630	57
			SUPPLEMENTARY TESTS					
a 6 B	21.2	3.54	12	2	10	962.5	3230	55
b 6 B	21.2	3.54	12	2	10	919.1	3280	92
c 6 B	21.2	3.54	12	2	10	1,144.6	3360	61
a 6 C	35.4	17.7	20	10	10	1,213.6	3150	67
b 6 C	35.4	17.7	20	10	10	1,295.3	3300	79
c 6 C	35.4	17.7	20	10	10	1,618.9	3120	83

TABLE 7 SUMMARY OF DATA FOR 7/8-INCH DIAMETER STUDS

Specimen	Test Load Max. (kips)	Test Load Min. (kips)	Shear Stress $S_{max.}$ (ksi)	Shear Stress $S_{min.}$ (ksi)	S_r (ksi)	N (kc.)	f'_c (psi)	Age (days)
e 1 G	24.0	-14.4	10	- 6	16	1,056.4	4990	95
e 2 G	33.7	-14.4	14	- 6	20	218.6	4590	94
e 3 G	43.3	-14.4	18	- 6	24	48.3	4390	102
e 2 H	33.7	4.8	14	2	12	2,133.0	4440	53
e 3 H	43.2	4.8	18	2	16	112.5	4400	53
e 4 H	52.9	4.8	22	2	20	33.0	4460	59
e 3 I	43.3	24.0	18	10	8	4,885.1	4450	63
e 4 I	52.9	24.0	22	10	12	587.7	4600	59
e 5 I	62.5	24.0	26	10	16	134.3	4520	88

TABLE 8 SUMMARY OF DATA FOR 4-INCH 5.4 LB. CHANNELS

Specimen	Test Load Max. (kips)	Test Load Min. (kips)	Shear Stress $S_{max.}$ (ksi)	Shear Stress $S_{min.}$ (ksi)	S_r (ksi)	N (kc.)	f'_c (psi)	Age (days)
P3	54	18	16.97	5.66	11.31	1,271.1	3900	28
P4	54	6	16.97	1.89	15.08	341.5	3920	35
P6≠	27	3	16.97	1.89	15.08	542.2	3610	35

≠Contained a single connector

MAIN EXPERIMENT

Specimen	Max. (kips)	Min. (kips)	$S_{max.}$ (ksi)	$S_{min.}$ (ksi)	S_r (ksi)	N (kc.)	f'_c (psi)	Age (days)
d 1 D	36	- 6	11.31	-1.89	13.20	4,175.3	5720	189
d 2 D	42	- 6	13.20	-1.89	15.09	9,556.3	6000	28
d 3 D	48	- 6	15.09	-1.89	16.98	1,013.0	6360	174
d 2 E	42	6	13.20	1.89	11.31	4,222.7	5900	64
d 3 E	48	6	15.09	1.89	13.20	904.4	5970	76
d 4 E	54	6	16.97	1.89	15.08	291.2	6140	49
d 3 F	48	18	15.09	5.66	9.43	7,035.6	6110	53
d 4 F	54	18	16.97	5.66	11.31	3,095.1	6060	35
d 5 F	60	18	18.86	5.66	13.20	476.3	6130	47

SUPPLEMENTARY TESTS

Specimen	Max. (kips)	Min. (kips)	$S_{max.}$ (ksi)	$S_{min.}$ (ksi)	S_r (ksi)	N (kc.)	f'_c (psi)	Age (days)
a 6 E≠	37.2	6	14.90	2.40	12.50	415.9	4640	27
b 6 E≠	30.5	6	12.21	2.40	9.81	2,962.4	4700	28
a 6 F≠	49.2	18	19.71	7.21	12.50	260.9	4660	32
b 6 F≠	42.5	18	17.02	7.21	9.81	721.3	4720	33

≠Weld size was 1/8-inch instead of 3/16-inch

TABLE 9 DATA FOR FLANGE THICKNESS AND OVERLOAD TESTS

Specimen	Test Load Max. (kips)	Test Load Min. (kips)	Shear Stress $S_{max.}$ (ksi)	Shear Stress $S_{min.}$ (ksi)	S_r (ksi)	N (kc.)	f_c' (psi)	Age (days)
PHASE II THICK AND THIN FLANGE TESTS								
8 W 31	24.8	3.54	14	2	12	405.5	3900	35
8 W 31	24.8	3.54	14	2	12	727.5	6330	79
8 W 67*	24.8	3.54	14	2	12	205.5	3480	89
8 W 67*	24.8	3.54	14	2	12	125.1	3450	89

*Welds were very poor

Specimen	Test Load Max. (kips)	Test Load Min. (kips)	Shear Stress $S_{max.}$ (ksi)	Shear Stress $S_{min.}$ (ksi)	S_r (ksi)	N (kc.)	f_c' (psi)	Age (days)
PHASE II OVERLOAD TESTS								
#1	21.2	3.54	12	2	10	807.8	3670	227
#2	21.2	3.54	12	2	10	897.6	3830	231
#3	35.4	17.7	20	10	10	800.3	3390	104
#4	35.4	17.7	20	10	10	650.0	3180	109

TABLE 10 RESULTS OF ANALYSIS OF DATA FOR STUD SHEAR CONNECTORS

Regression Curve (on Plane)	Linearized Model	Regression Coefficients A	B	C	Eq. of Model	Model Constants α	β	γ	Stand. Error of Est.	Coeff. of Correl.
A	Log N = A + B S_r	7.9772	-0.1717		4.5	46.4695	-5.8253		0.1901	0.9623
B	Log N = A + B S_r	7.6093	-0.1481		4.5	51.0237	-6.7054		0.1799	0.9586
C	Log N = A + B S_r	8.2148	-0.1886		4.5	43.5552	-5.3020		0.1755	0.9700
D	Log N = A + B log S_r	11.5033	-5.2558		4.4	154.4146	0.1903		0.1835	0.9657
E	Log e^N = A + B log S_r	4.8414	-2.1425		4.4	181.8270	0.4667		0.1329	0.9023
F	Log N = A+B S_r+C $S_{min.}$	7.8557	-0.1675	+0.0013	4.7	46.8888	-5.9687	0.0793	0.1931	0.9633
G	Log N = A+B S_r+C $S_{min.}$	8.1472	-0.1599	-0.0041	4.7	50.9536	-6.2541	-0.2605	0.3093	0.8996
H	Log N = A+B log S_r+C log $S_{min.}$	11.3351	-5.1636	0.0979	4.8	156.7480	0.1937	0.0979	0.1836	0.9669
I	Log N = A+B log S_r+C log $S_{min.}$	12.1293	-5.5342	-0.4132	4.8	155.4870	0.1807	-0.4131	0.2749	0.9215

TABLE 11 RESULTS OF ANALYSIS OF DATA FOR CHANNEL SHEAR CONNECTORS

Regression Curve (on Plane)	Linearlized Model	Regression Coefficients A	B	C	Eq. of Model	Model Constants α	β	γ	Stand. Error of Est.	Coeff. of Correl.
J	Log N = $A + B\ S_r$	8.9469	-0.8554		4.5	10.4598	- 1.1691		0.1975	0.9299
K	Log N = $A + B \log S_r$	9.4669	-6.5137		4.4	28.4037	0.1535		0.1919	0.9341
L	Log N = $A + B\ S_r$	8.1384	-0.1752		4.5	46.4542	- 5.7080		0.3378	0.7369
M	Log N = $A + B \log S_r$	12.5551	-6.0222		4.4	121.5654	0.1661		0.3342	0.7433
N	Log N = $A+B\ S_r+C\ S_{min.}$	8.2970	-0.1077	-0.1534	4.7	54.0820	- 6.5183	- 0.7019	0.4616	0.5491
O	Log N = $A+B \log S_r+C \log S_{min.}$	14.0126	-6.7791	-1.0970	4.8	116.6884	0.1475	- 1.0970	0.3226	0.8115
P	Log N = $A+B\ S_r + C\sqrt{f_c'}$	4.7649	-0.0863	1.0620	4.9	55.1747	-11.5793	12.2987	0.4592	0.5557
Q	Log N = $A+B \log S_r + C \log \sqrt{f_c'}$	6.9934	-2.5447	5.3069	4.10	560.0063	0.3930	5.3069	0.4576	0.5600
R	Log N = $A + B\ S_r$	8.0750	-0.1675		4.5	48.1991	- 5.9689		0.2833	0.9276
S	Log N = $A + B\ S_r$	7.6209	-0.1380		4.5	52.2402	- 7.2467		0.2809	0.8432

TABLE 12 COMPARISON OF FRACTURE AREA WITH SPECIMEN PERFORMANCE FOR CHANNEL CONNECTORS

Specimen	Total Area of Fracture Surface (in.^{2})	$\frac{\text{Area}}{\text{Average Area}}$
P3	2.31	1.02
P4	2.74	1.21
P6	2.24	0.99
d 1 D	2.13	0.94
d 2 D	1.79	0.79
d 3 D	2.48	1.10
d 2 E	2.45	1.08
d 3 E	2.59	1.15
d 4 E	2.01	0.89
d 3 F	2.06	0.91
d 4 F	2.23	0.99
d 5 F	2.07	0.92
a 6 E	1.68	1.00
a 6 F	1.67	0.99
b 6 E	1.90	1.13
b 6 F	1.49	0.89

TABLE 13 COMPARISON OF CYCLE LIFE WITH STUD DIAMETER TO FLANGE THICKNESS RATIO

Steel Section	Flange Thickness (in.)	Stud Diameter (in.)	d/t_f	S_r (ksi)	N (kc)
8 W 31	0.433	3/4	1.73	12	405.5
8 W 31	0.433	3/4	1.73	12	727.5
8 W 40	0.558	7/8	1.57	12	2133.0
8 W 40	0.558	3/4	1.34	12	897.3
8 W 40	0.558	3/4	1.34	12	565.3
8 W 40	0.558	3/4	1.34	12	551.1
8 W 67	0.933	3/4	0.80	12	205.5 *
8 W 67	0.933	3/4	0.80	12	125.1 *

*Connectors were welded with an incorrect setting

TABLE 14 VALUES OF ALLOWABLE RANGE OF LOAD

Type of Connector		Cycles*	Allowable Range of Load 100,000	500,000	2,000,000
Studs (lbs./conn.)	1/2 in.		3,340	2,560	1,900
	5/8 in.		5,450	4,180	3,100
	3/4 in.		7,750	5,940	4,400
	7/8 in.		10,500	8,100	6,000
Channels**	3 in. 4.1 lbs.		4,000	3,200	2,600
	4 in. 5.4 lbs.		4,000	3,200	2,600
	5 in. 6.7 lbs.		4,000	3,200	2,600

*See AASHO Specification section 1.8.3 for the number of cycles of maximum stress to be considered in the design.

**At least 3/16 in. fillet welds at the heel and toe of the channel.

TABLE 15 ULTIMATE STRENGTH OF CONNECTORS Q_u

Type Connector		Ultimate Strength in lbs. $f'_c = 3000$	$f'_c = 3500$	$f'_c = 4000$
Studs	1/2 in.	12,700	13,700	14,700
	5/8 in.	19,900	21,500	24,800
	3/4 in.	28,700	31,000	35,800
	7/8 in.	39,000	42,100	48,600
Channels per inch of length	3 in. 4.1 lbs.	10,800	11,700	12,500
	4 in. 5.4 lbs.	11,600	12,500	13,400
	5 in. 6.7 lbs.	12,500	13,500	14,400

TABLE 16 COMPARISON OF SHEAR CONNECTOR REQUIREMENTS FOR CURRENT AND PROPOSED DESIGN PROCEDURE

Bridge Span (ft.)	Current Design F.S. = 30	Proposed Design			Ultimate Strength Requirements
		2×10^6 cycles	5×10^5 cycles	1×10^5 cycles	
50	238	194	144	110	102
60	252	208	154	118	114
70	276	226	168	128	114
80	294	234	174	134	114
90	292	250	186	142	114

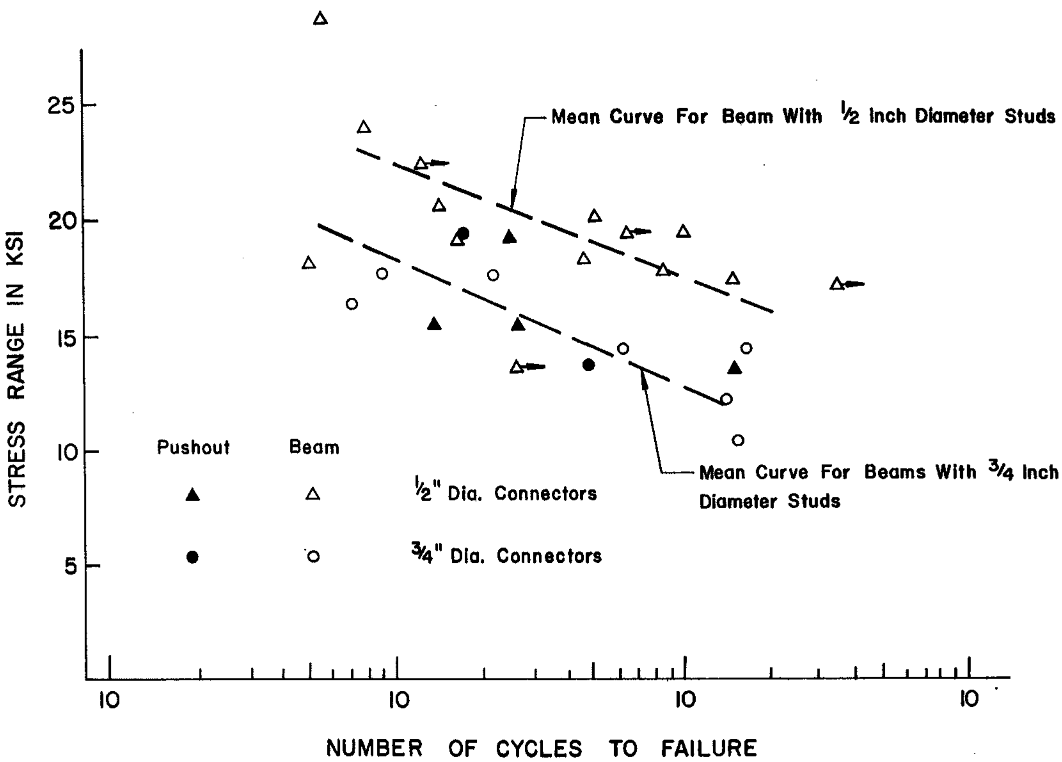


Fig. 1 DATA FROM PREVIOUS TESTS OF STUD SHEAR CONNECTORS

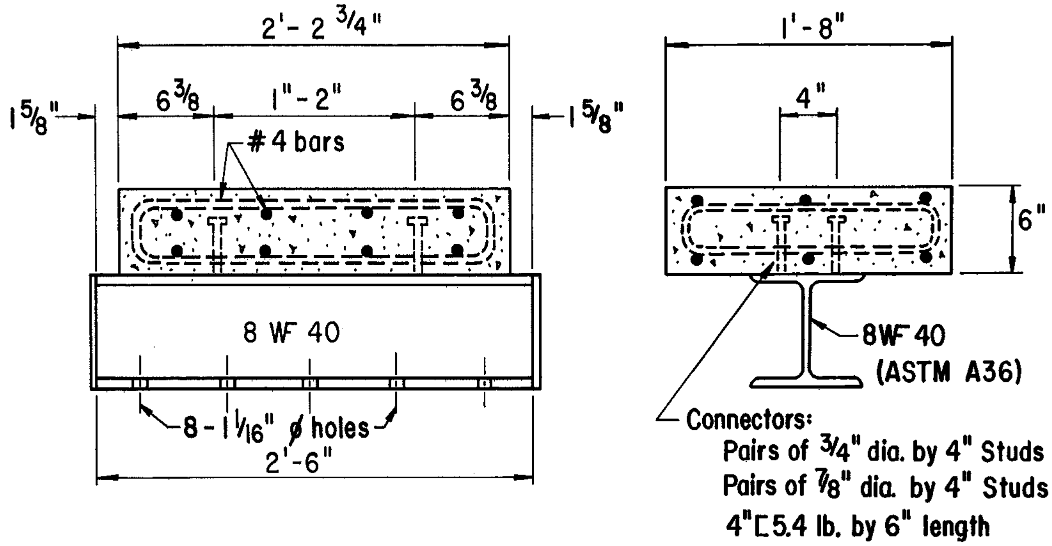


Fig. 2 DETAILS OF TEST SPECIMENS

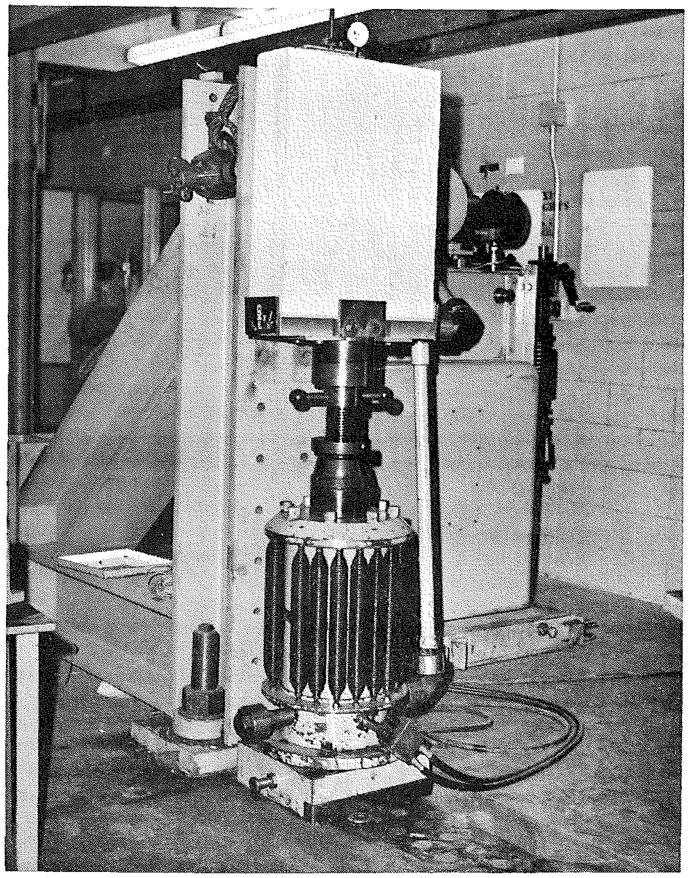


Fig. 3 TEST SETUP FOR LOADING IN ONE DIRECTION

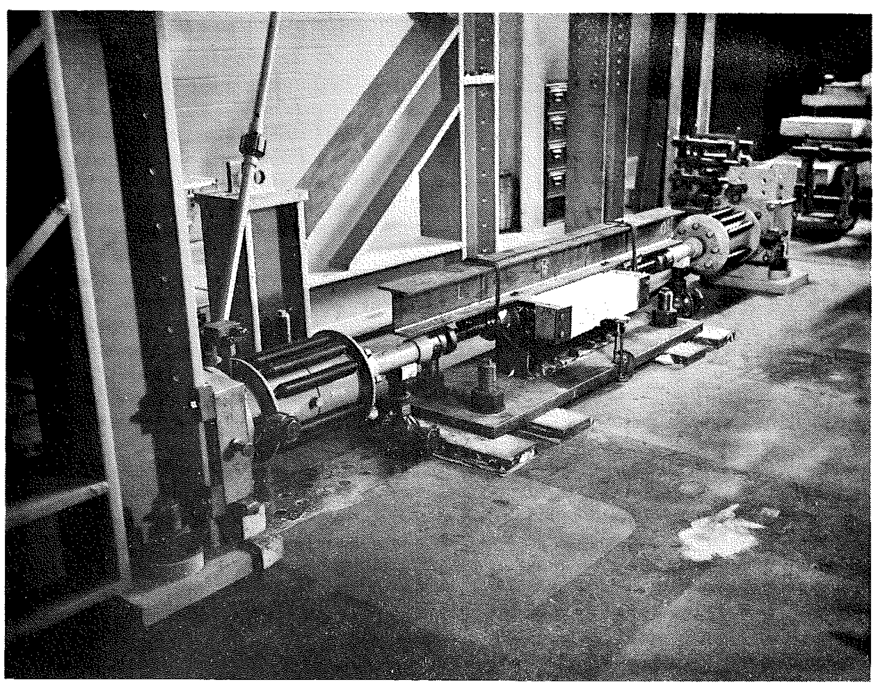


Fig. 4 TEST SETUP FOR STRESS REVERSAL TESTS

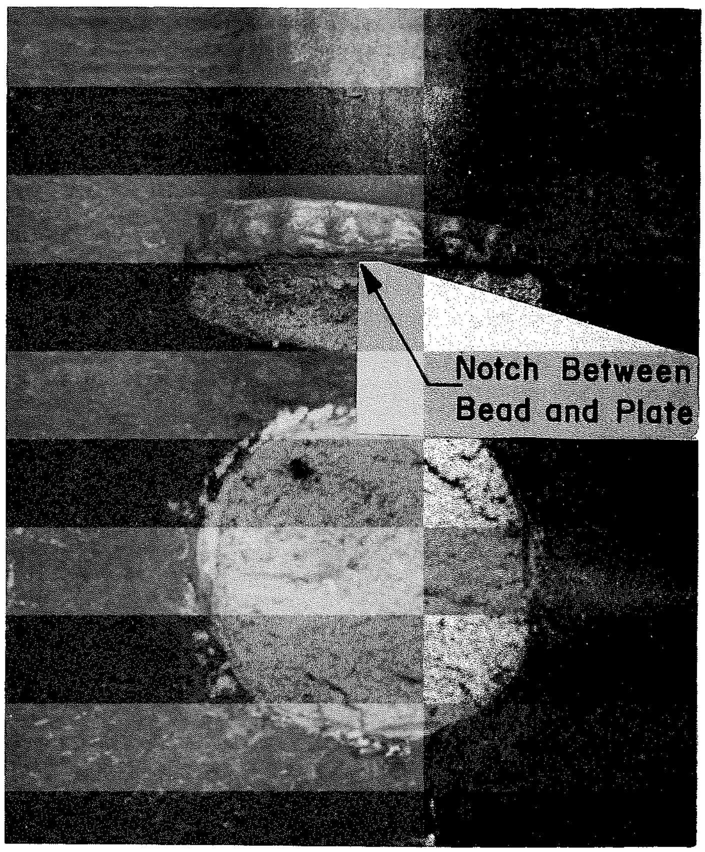
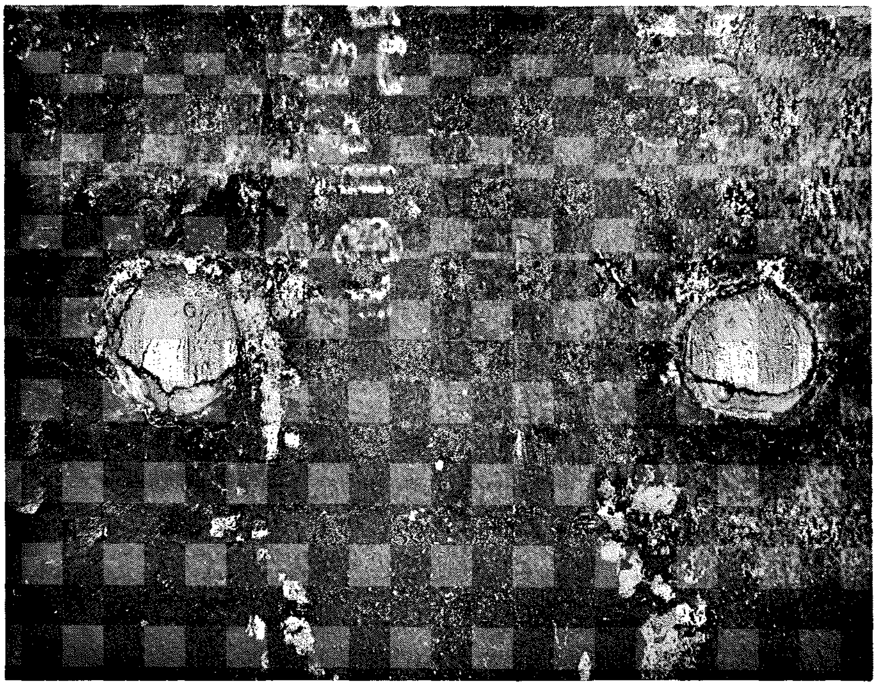
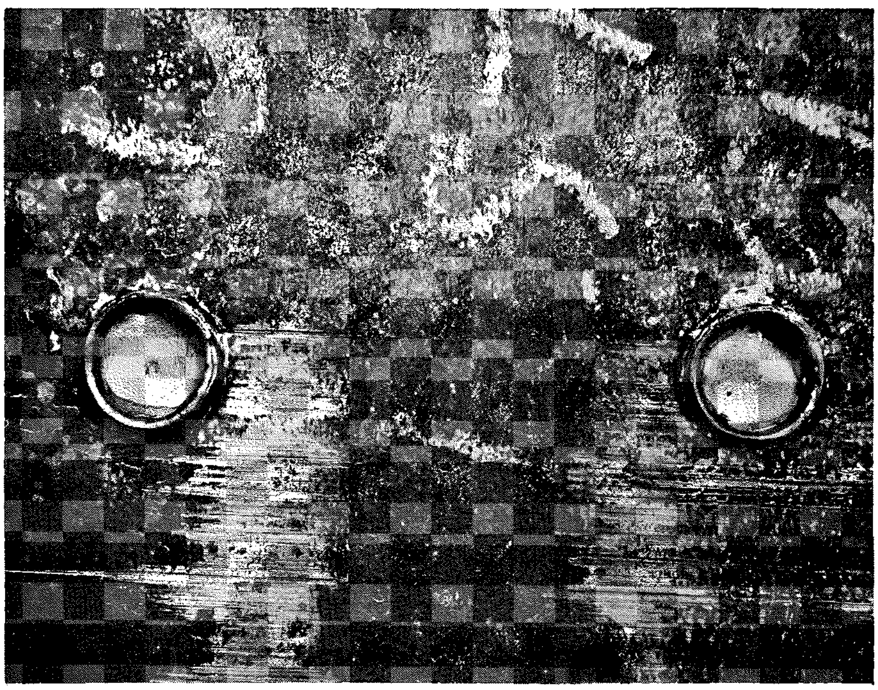


Fig. 5 NOTCH AT THE BASE OF A STUD CONNECTOR



(a)



(b)

Fig. 6 TYPICAL FAILURES OF 3/4 INCH STUD CONNECTORS

Fig. 7 TYPICAL FAILURES OF CHANNEL CONNECTORS

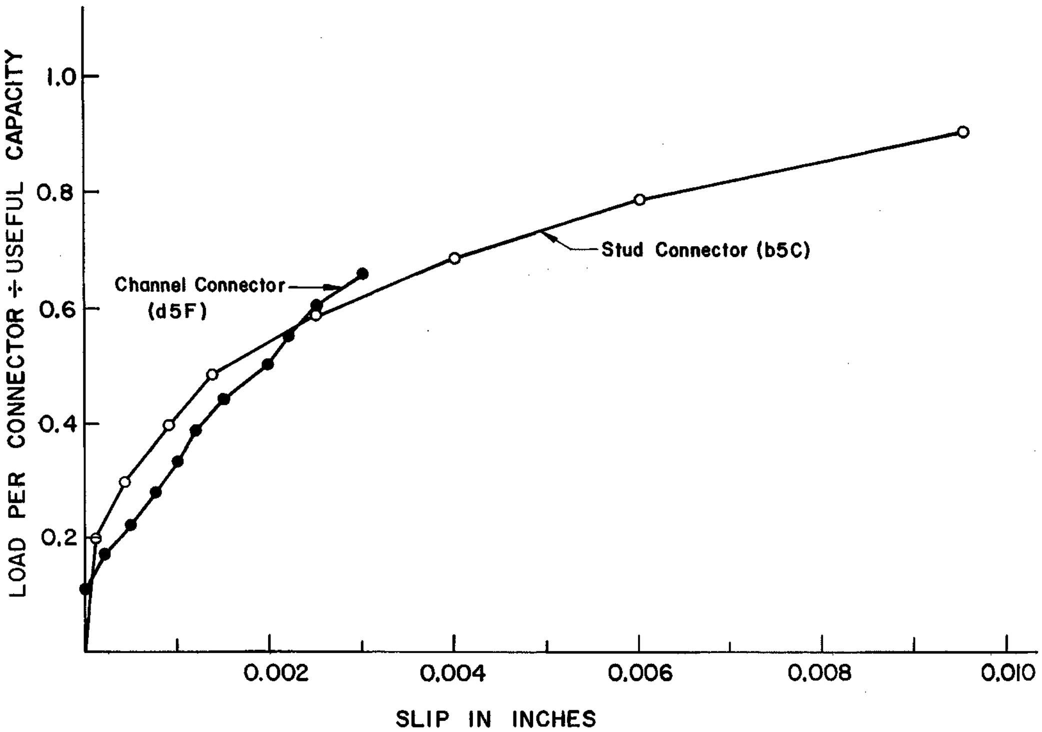


Fig. 8 COMPARISON OF LOAD-SLIP CHARACTERISTICS OF CHANNEL AND STUD CONNECTORS

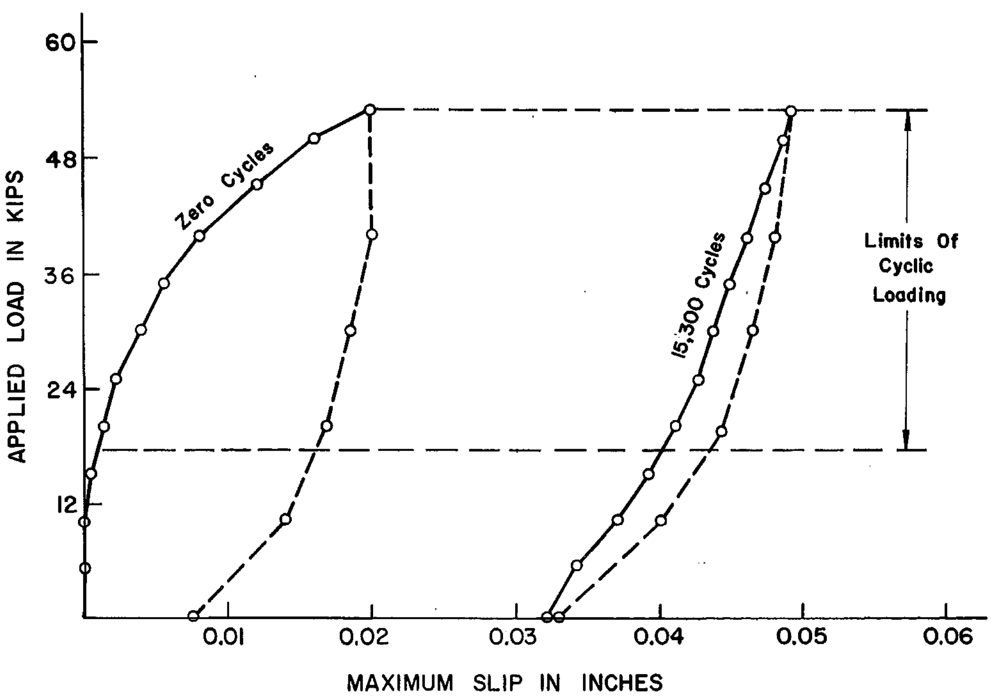


Fig. 9 LOAD-SLIP CURVES FOR SPECIMEN P1

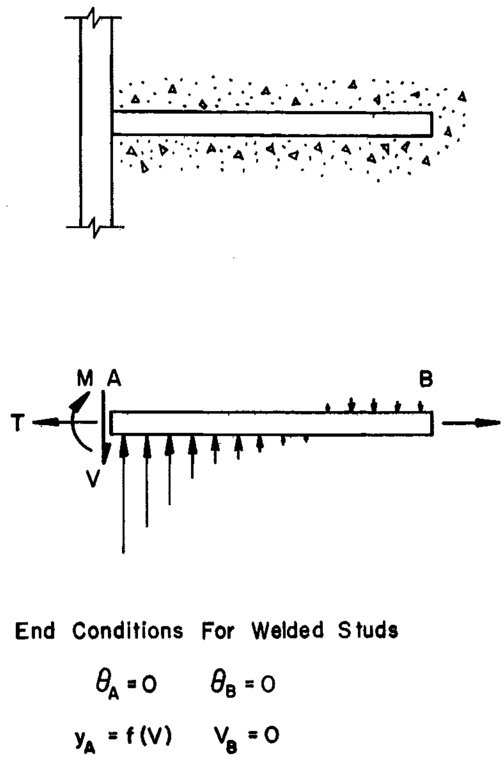


Fig. 10 FORCES ON A SHEAR CONNECTOR ACTING AS A DOWEL

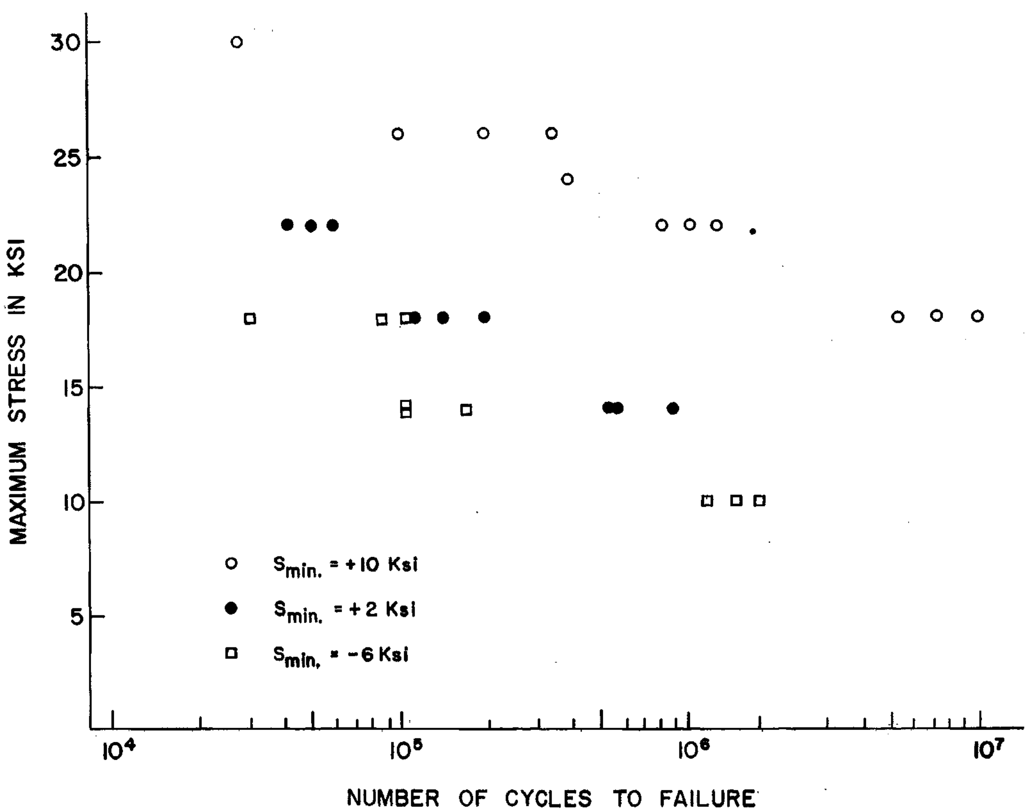


Fig. 11 PLOT OF MAXIMUM STRESS VERSUS CYCLE LIFE FOR 3/4 INCH STUD CONNECTORS

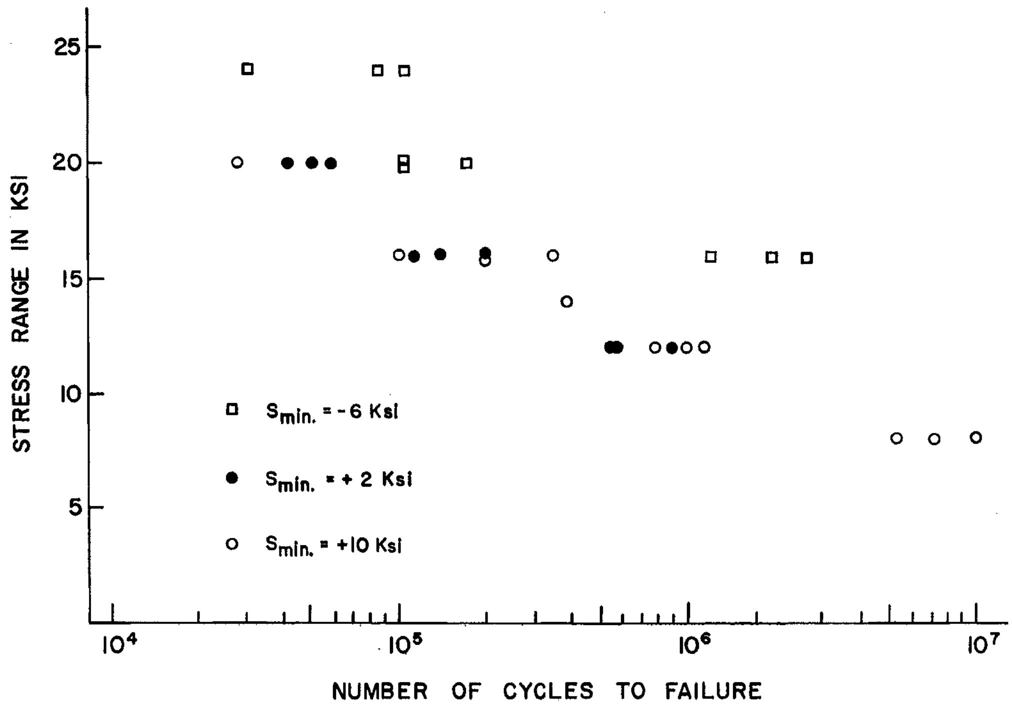


Fig. 12 PLOT OF STRESS RANGE VERSUS CYCLE LIFE FOR 3/4 INCH STUD CONNECTORS

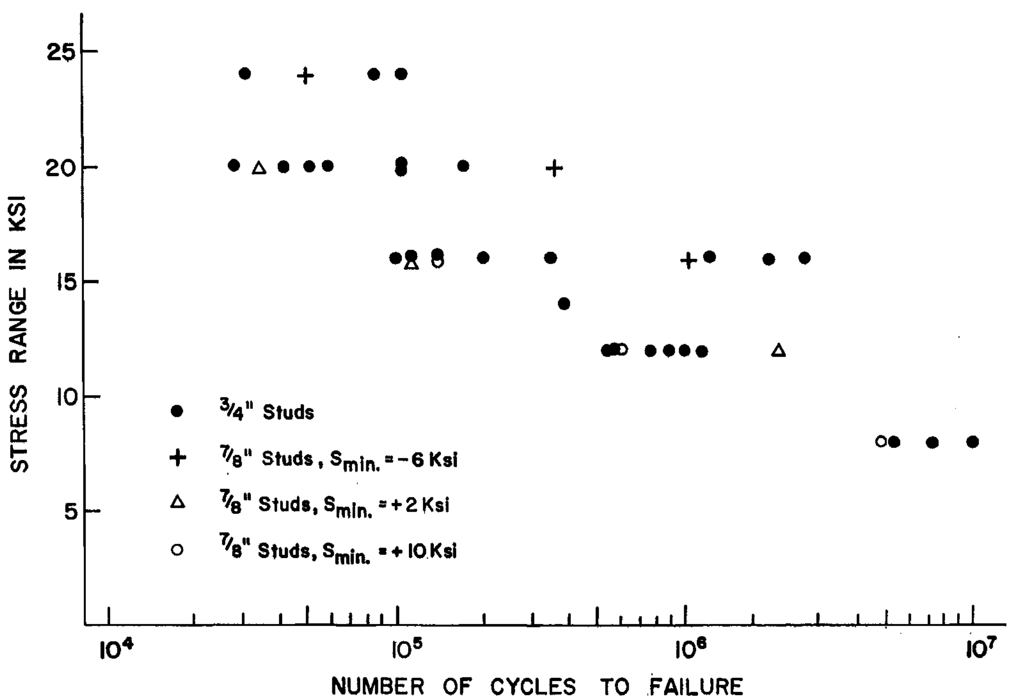


Fig. 13 COMPARISON OF TEST RESULTS FOR 3/4 AND 7/8 INCH STUD CONNECTORS

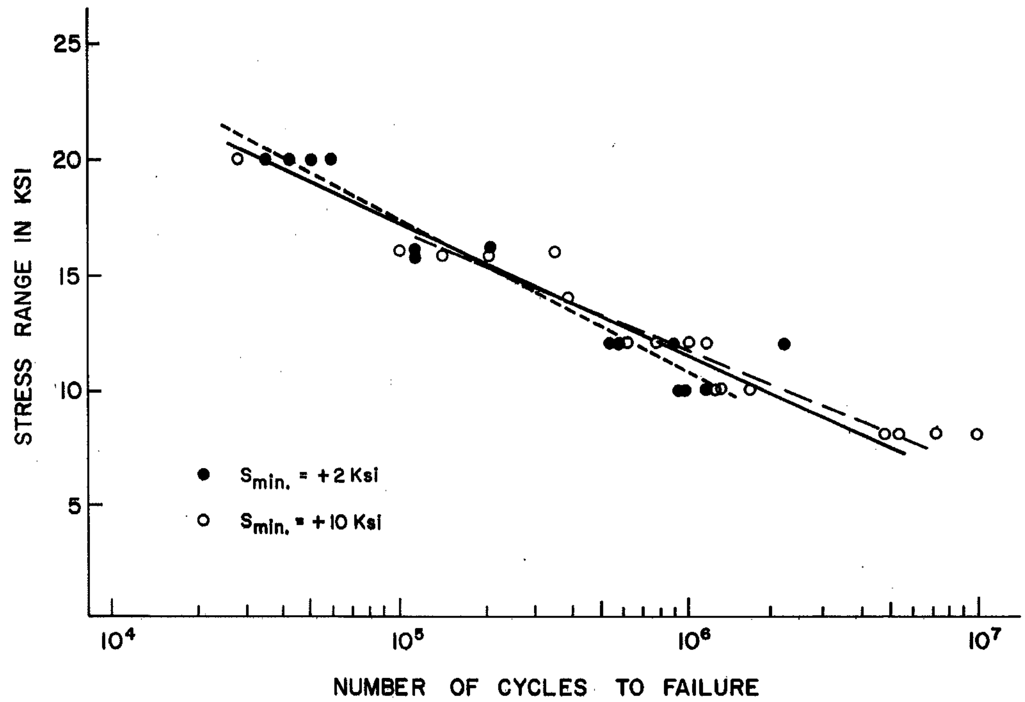


Fig. 14 COMPARISON OF REGRESSION CURVES FOR 3/4 INCH STUD CONNECTORS

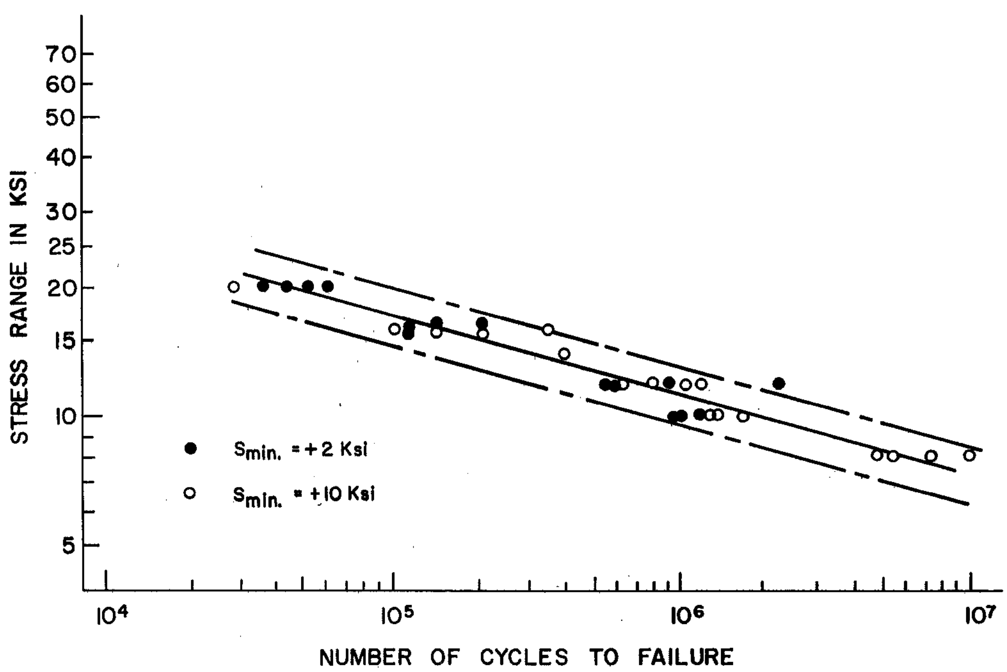


Fig. 15 LOG-LOG PLOT OF DATA ON STUD CONNECTORS

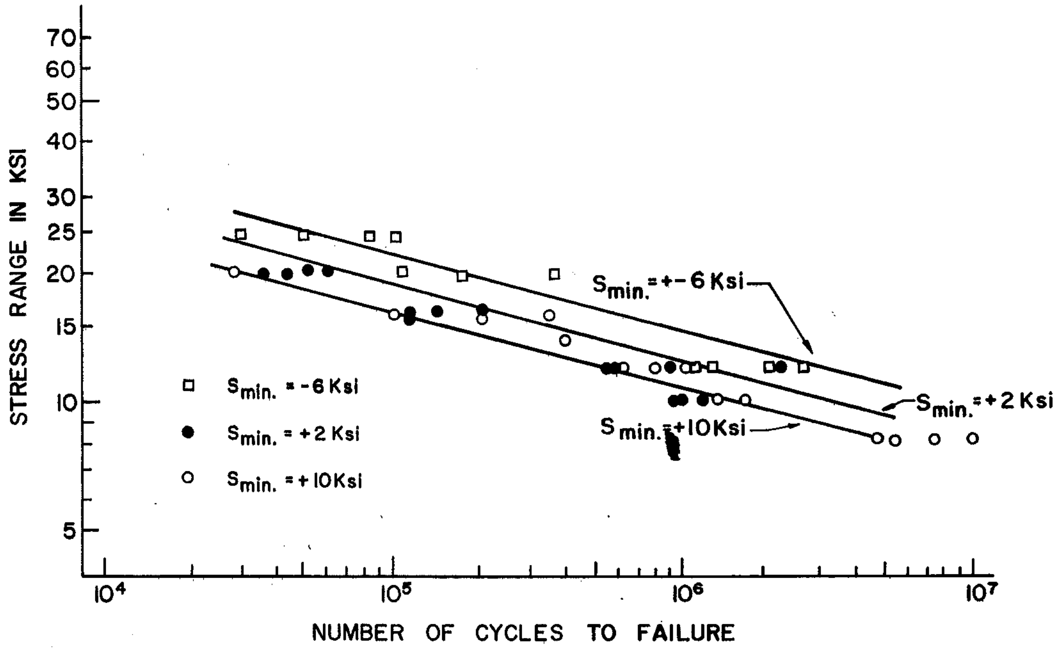


Fig. 16 COMPARISON OF STUD CONNECTOR DATA WITH CURVES OF EQUATION 4.8

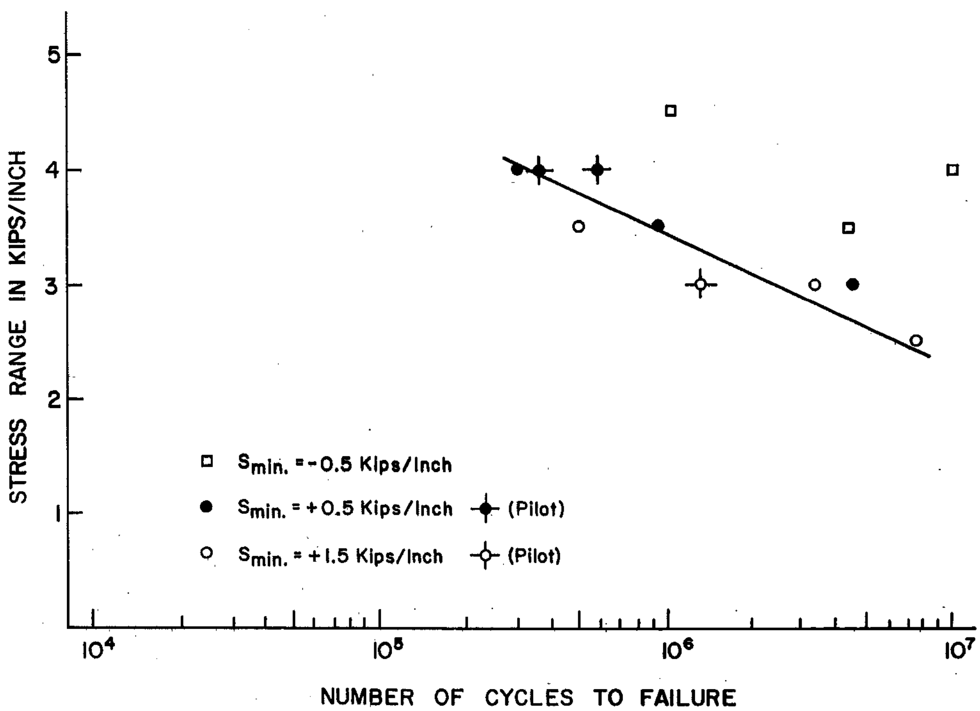


Fig. 17 SEMI-LOG PLOT OF DATA FOR MAIN EXPERIMENT AND PILOT TESTS OF CHANNEL CONNECTORS

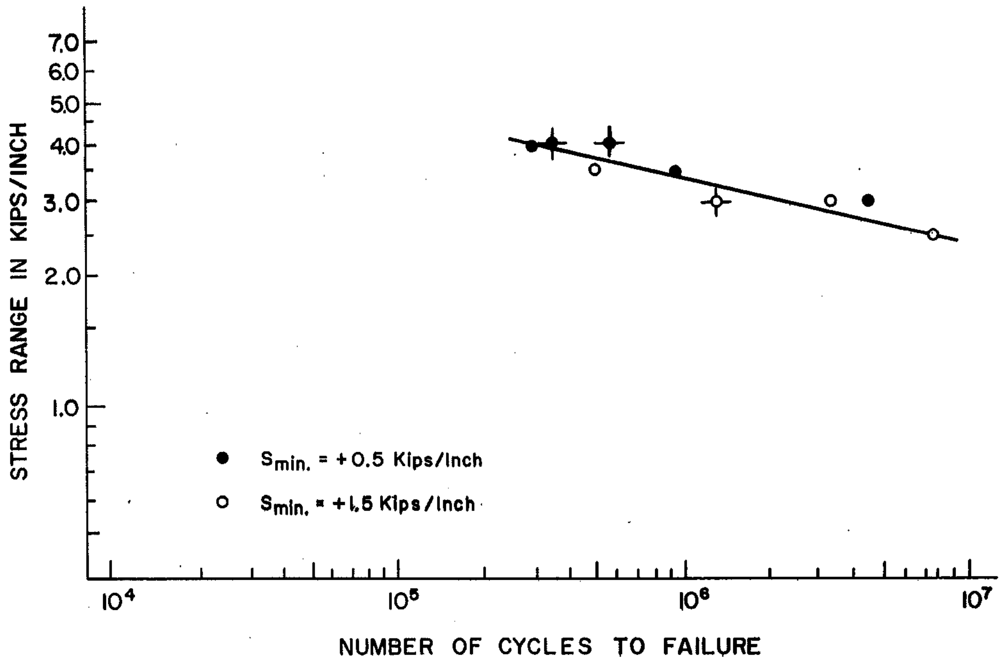


Fig. 18 LOG-LOG PLOT OF DATA FOR MAIN EXPERIMENT AND PILOT TESTS OF CHANNEL CONNECTORS

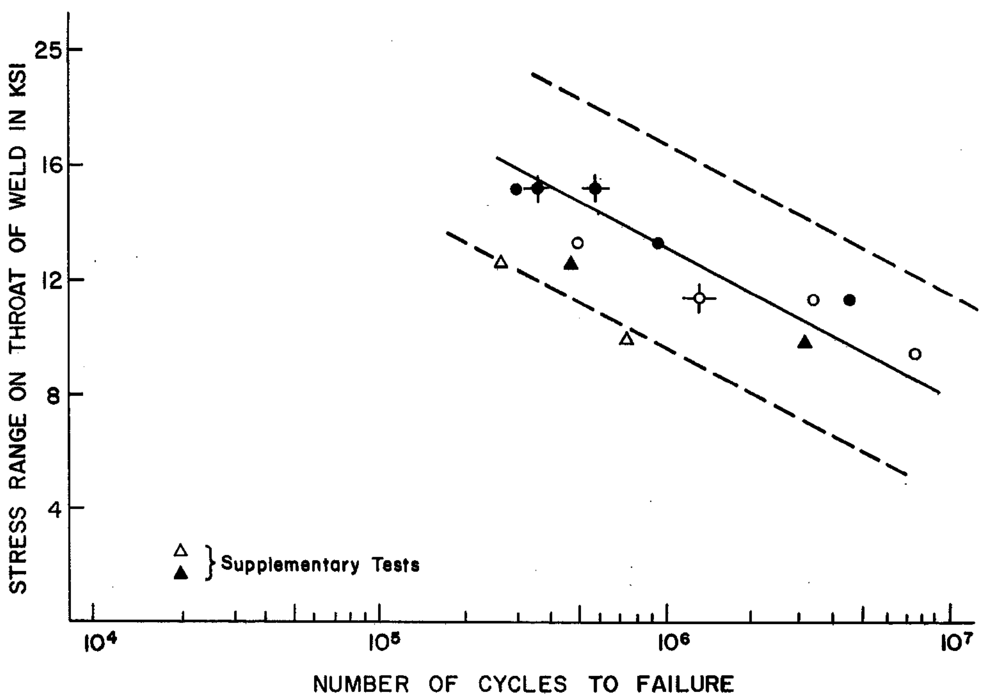


Fig. 19 SEMI-LOG PLOT OF DATA FOR CHANNEL CONNECTORS INCLUDING SUPPLEMENTARY TESTS

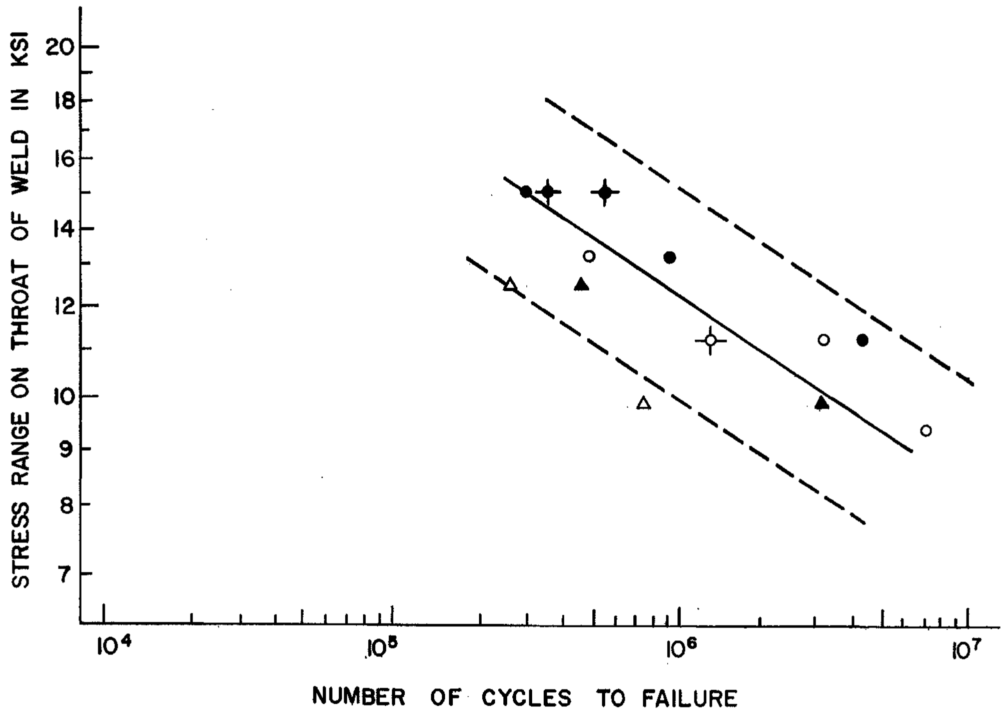


Fig. 20 LOG-LOG PLOT OF DATA FOR CHANNEL CONNECTORS INCLUDING SUPPLEMENTARY TESTS

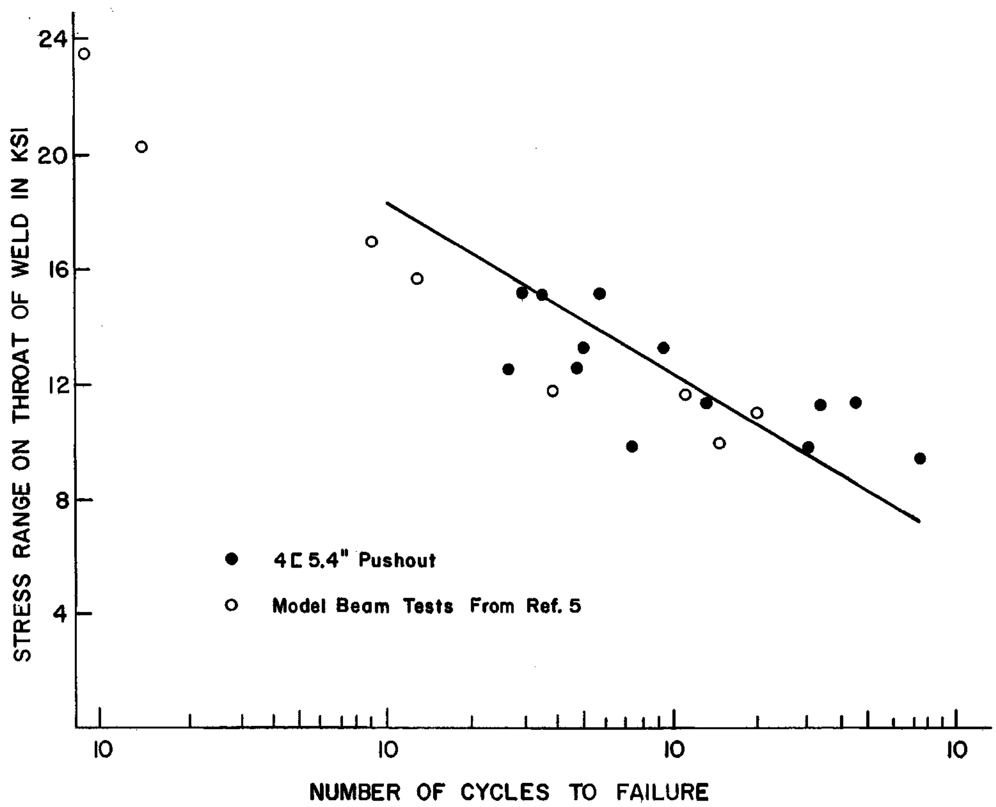


Fig. 21 COMPARISON OF DATA FROM PUSHOUT AND MODEL BEAM TESTS

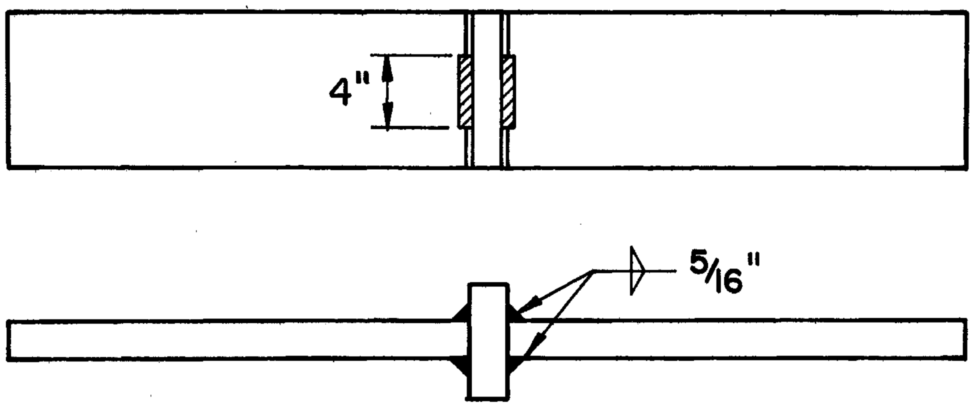


Fig. 22 TEST SPECIMEN FOR TRANSVERSE TESTS OF FILLET WELDS

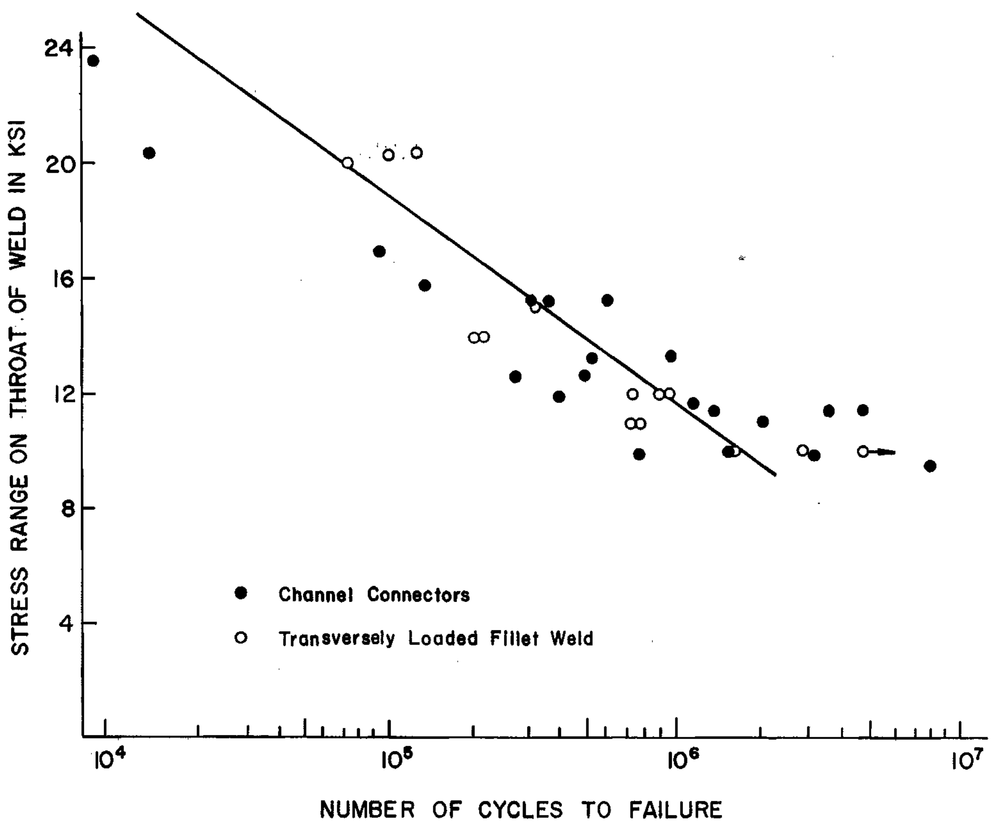


Fig. 23 COMPARISON OF DATA FROM CHANNEL CONNECTORS AND TRANSVERSELY LOADED FILLET WELDS

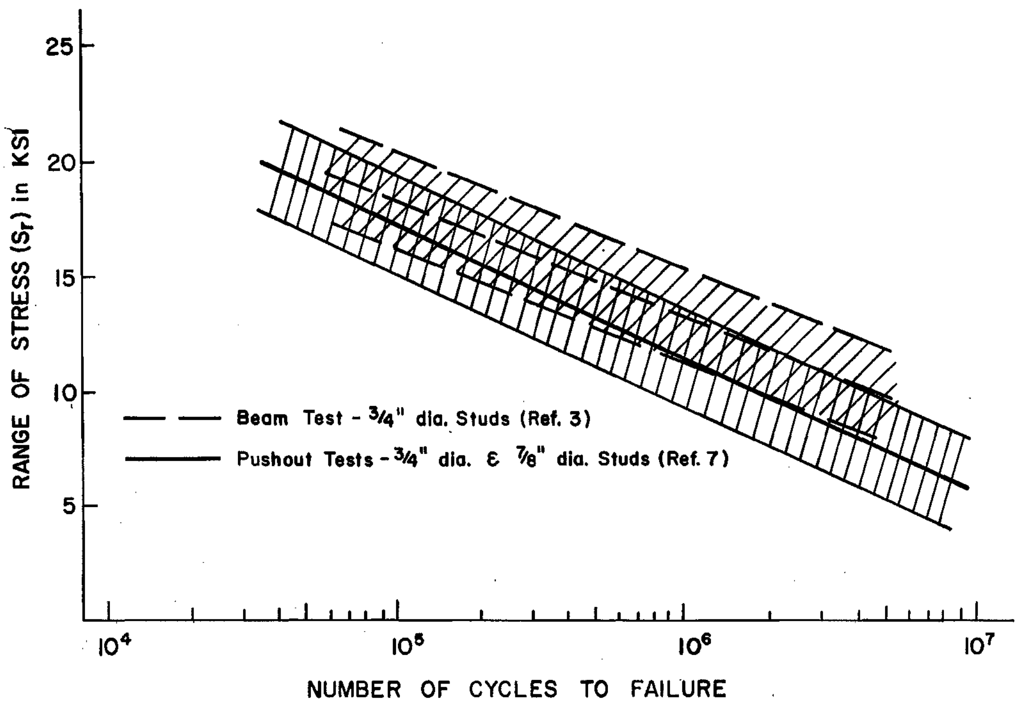


Fig. 24 COMPARISON OF DATA FROM PUSHOUT AND FULL SCALE BEAM TESTS

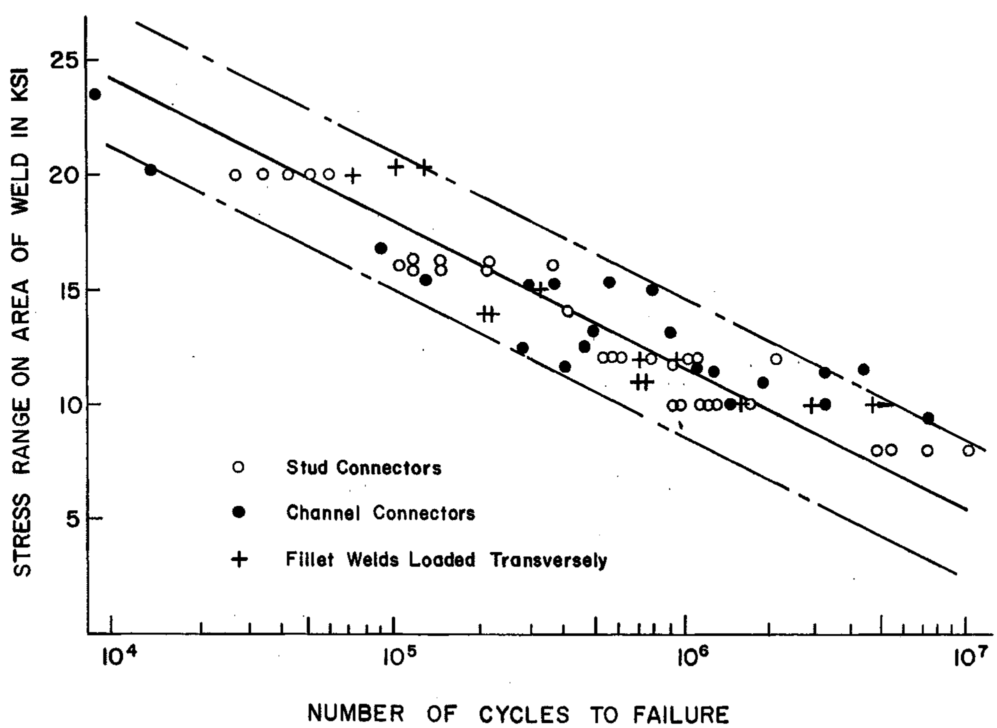


Fig. 25 SEMI-LOG PLOT OF ALL DATA

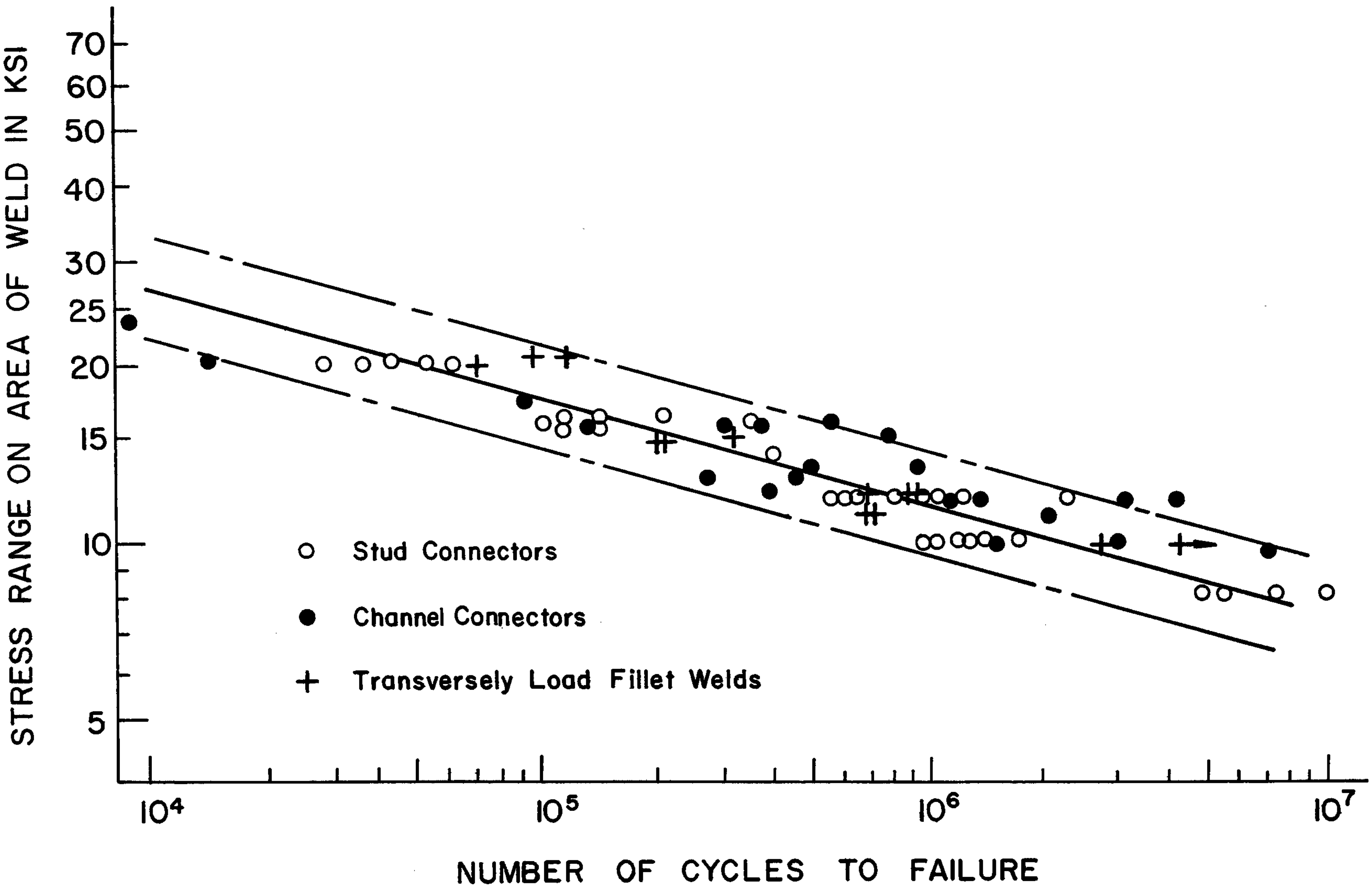
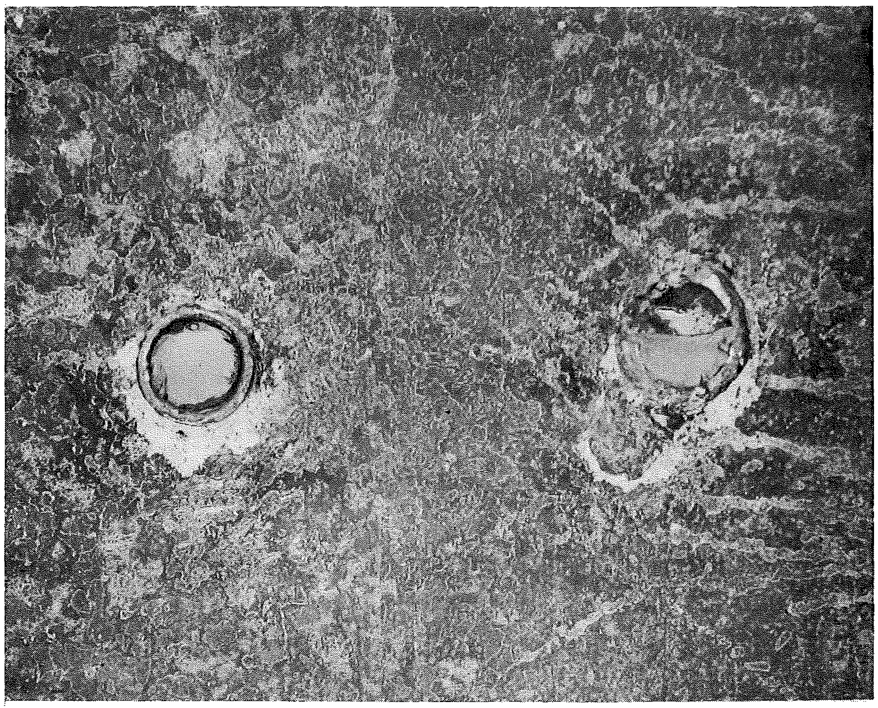


Fig. 26 LOG-LOG PLOT OF ALL DATA



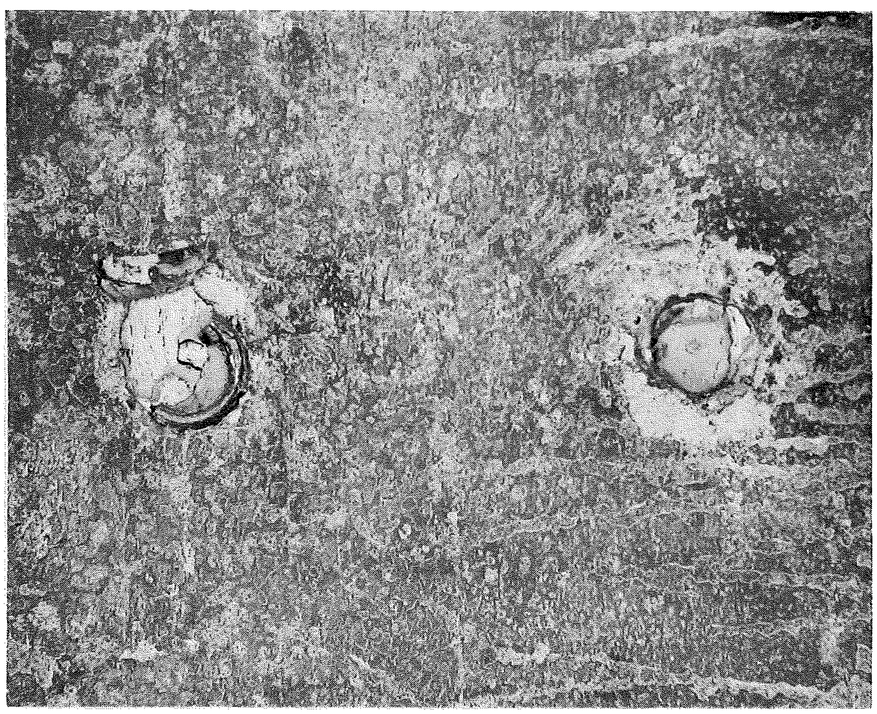


Fig. 27 FRACTURED WELDS ON 8 W 67 STEEL BEAMS

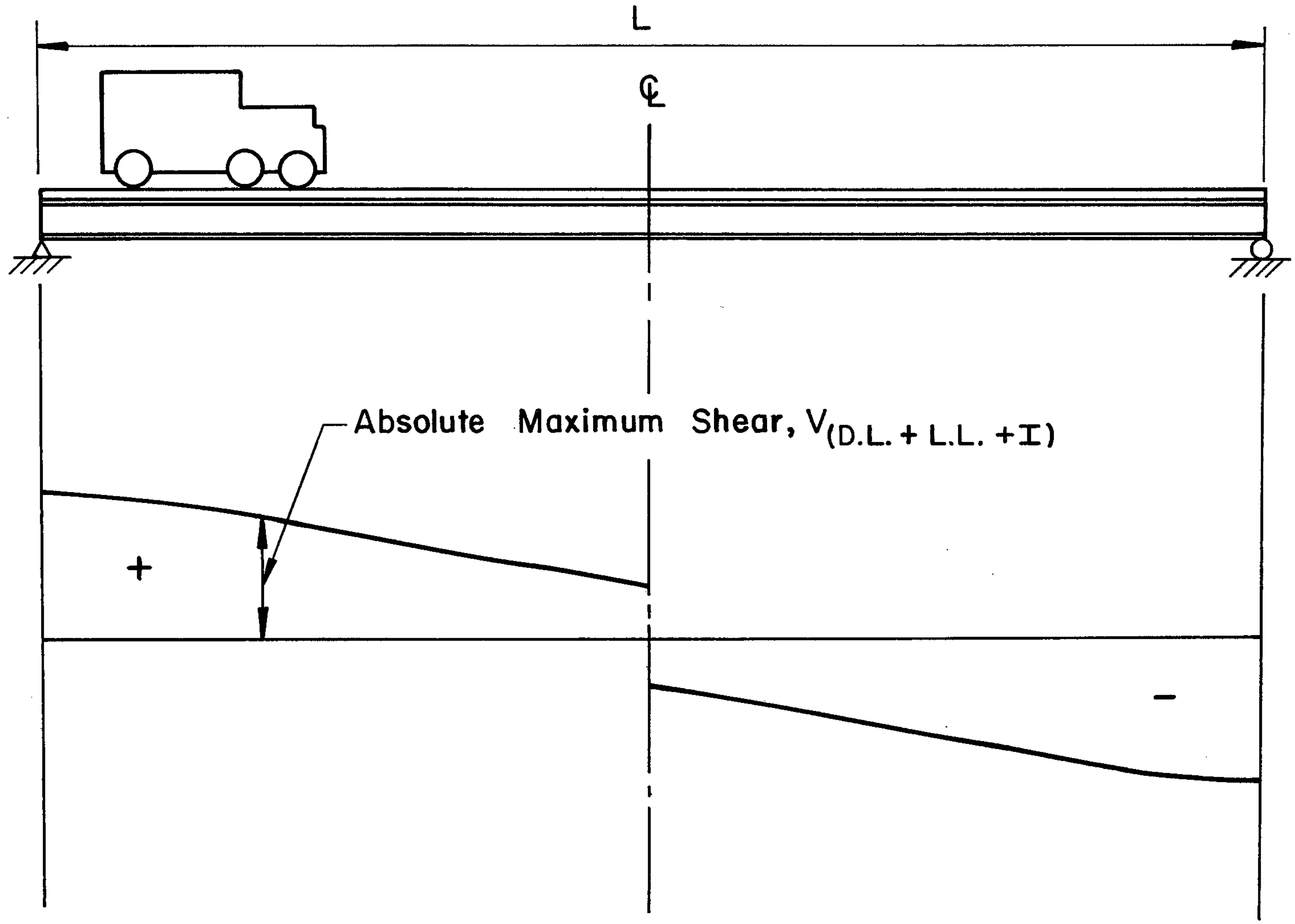


Fig. 28 MAXIMUM SHEAR DIAGRAM FOR SIMPLE SPAN BEAMS

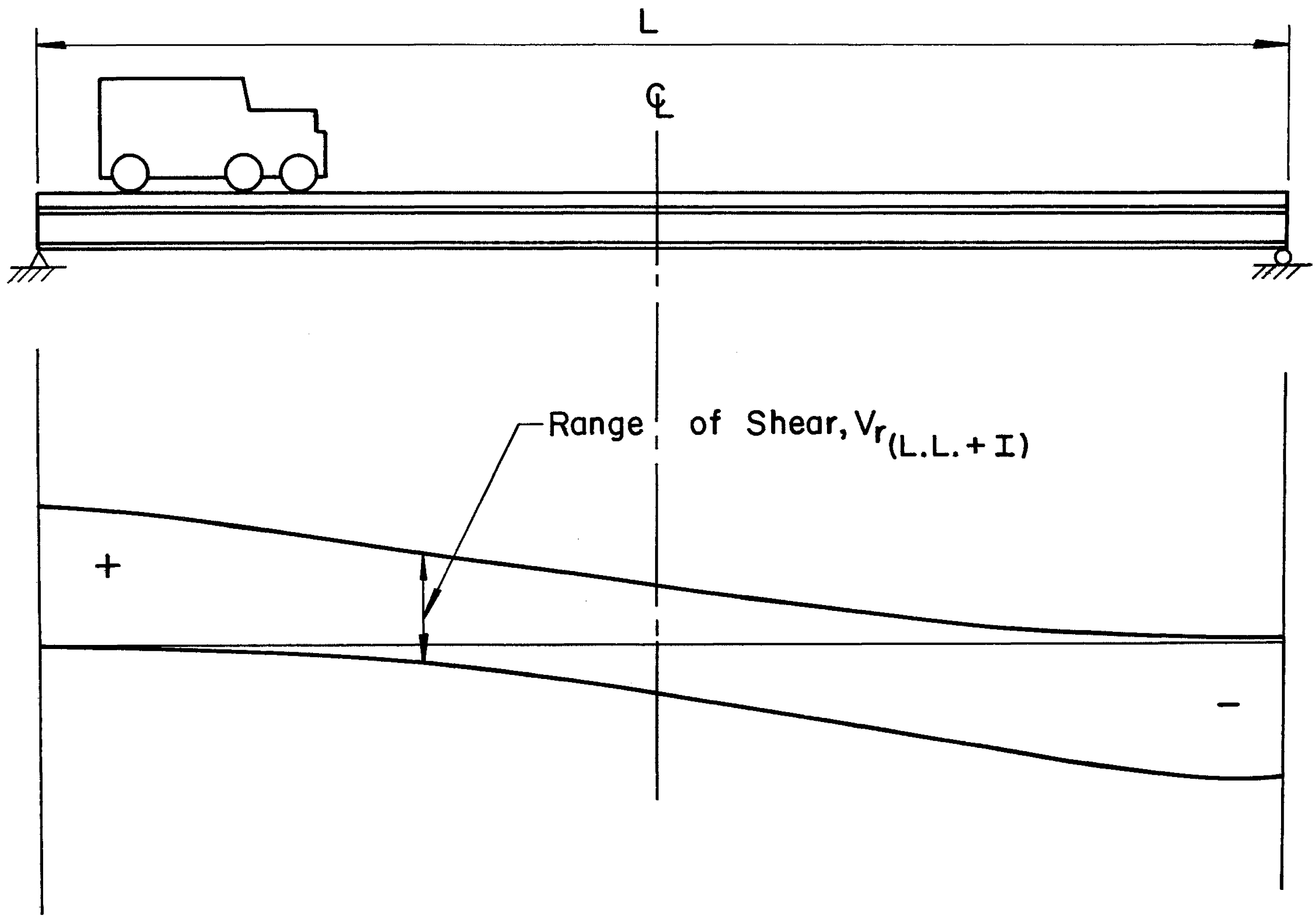


Fig. 29 RANGE OF SHEAR DIAGRAM FOR A SIMPLE SPAN BEAM

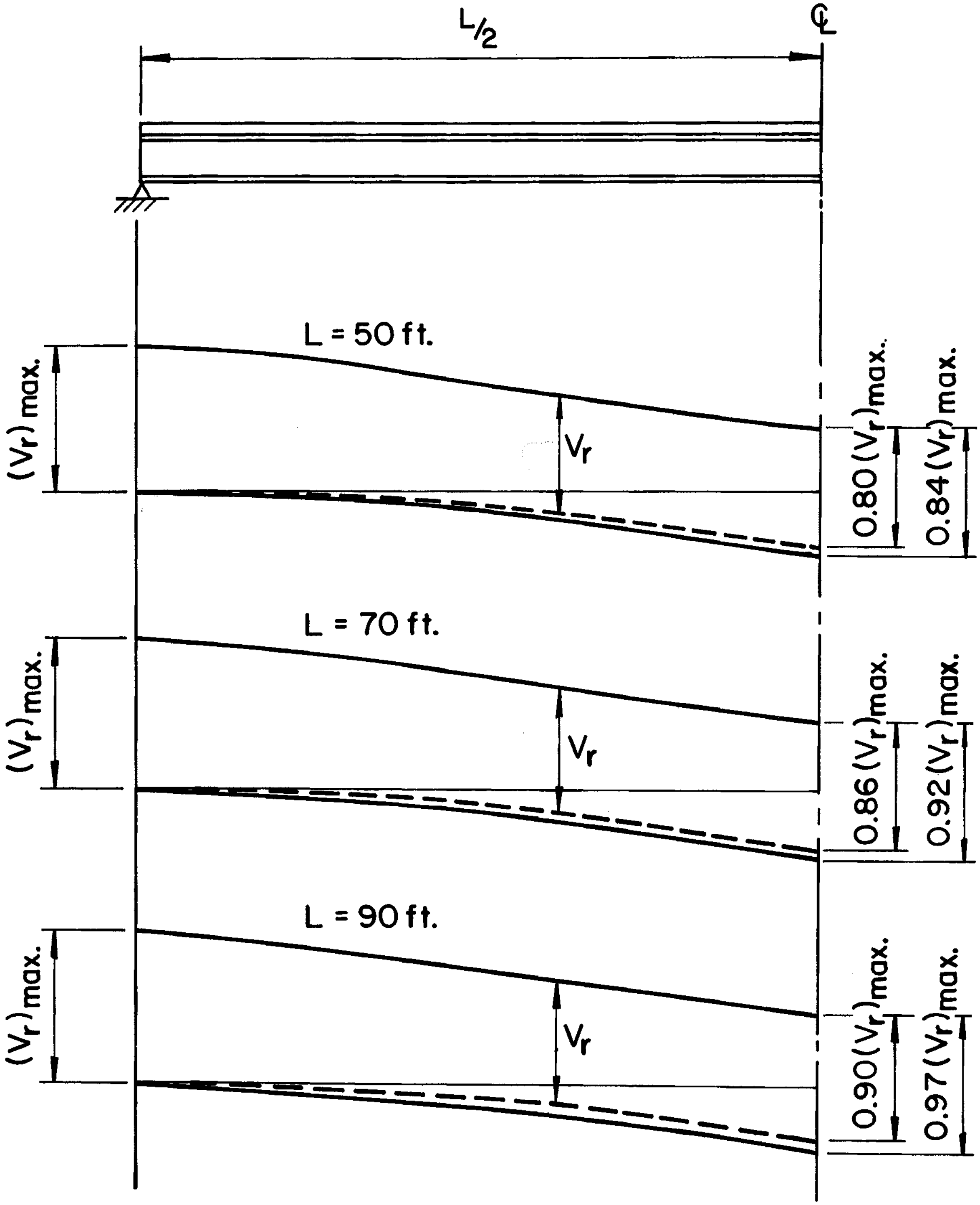


Fig. 30 VARIATIONS IN RANGE OF SHEAR DIAGRAMS

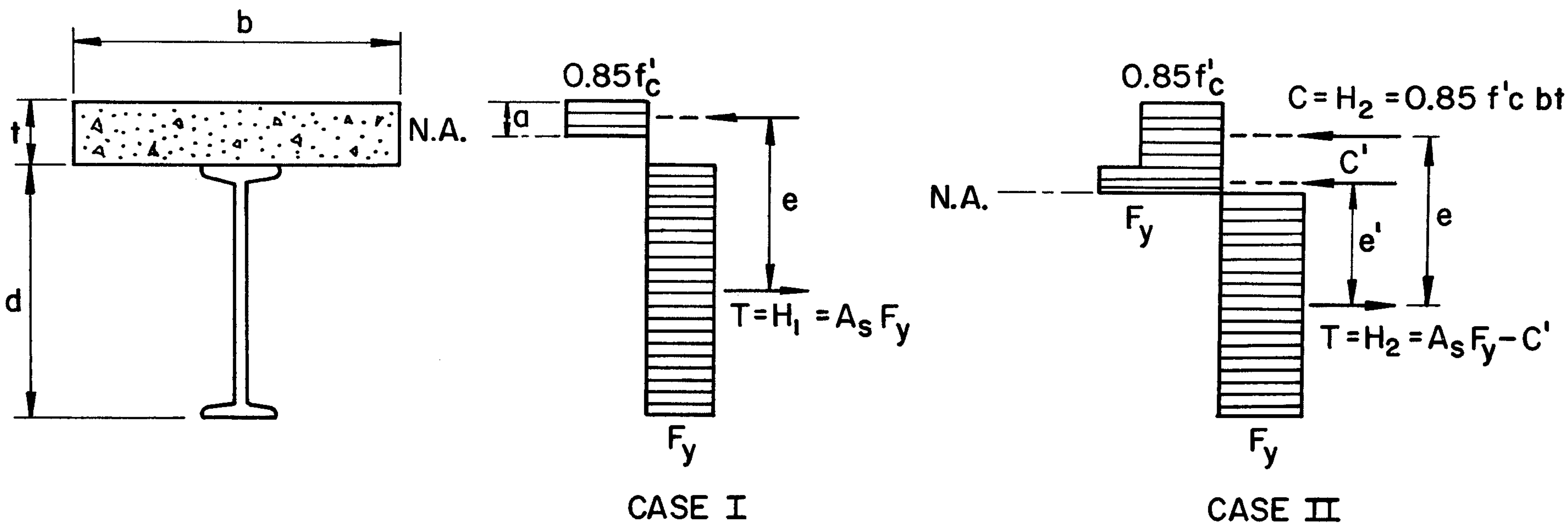


Fig. 31 EQUILIBRIUM CONDITIONS AT ULTIMATE LOAD

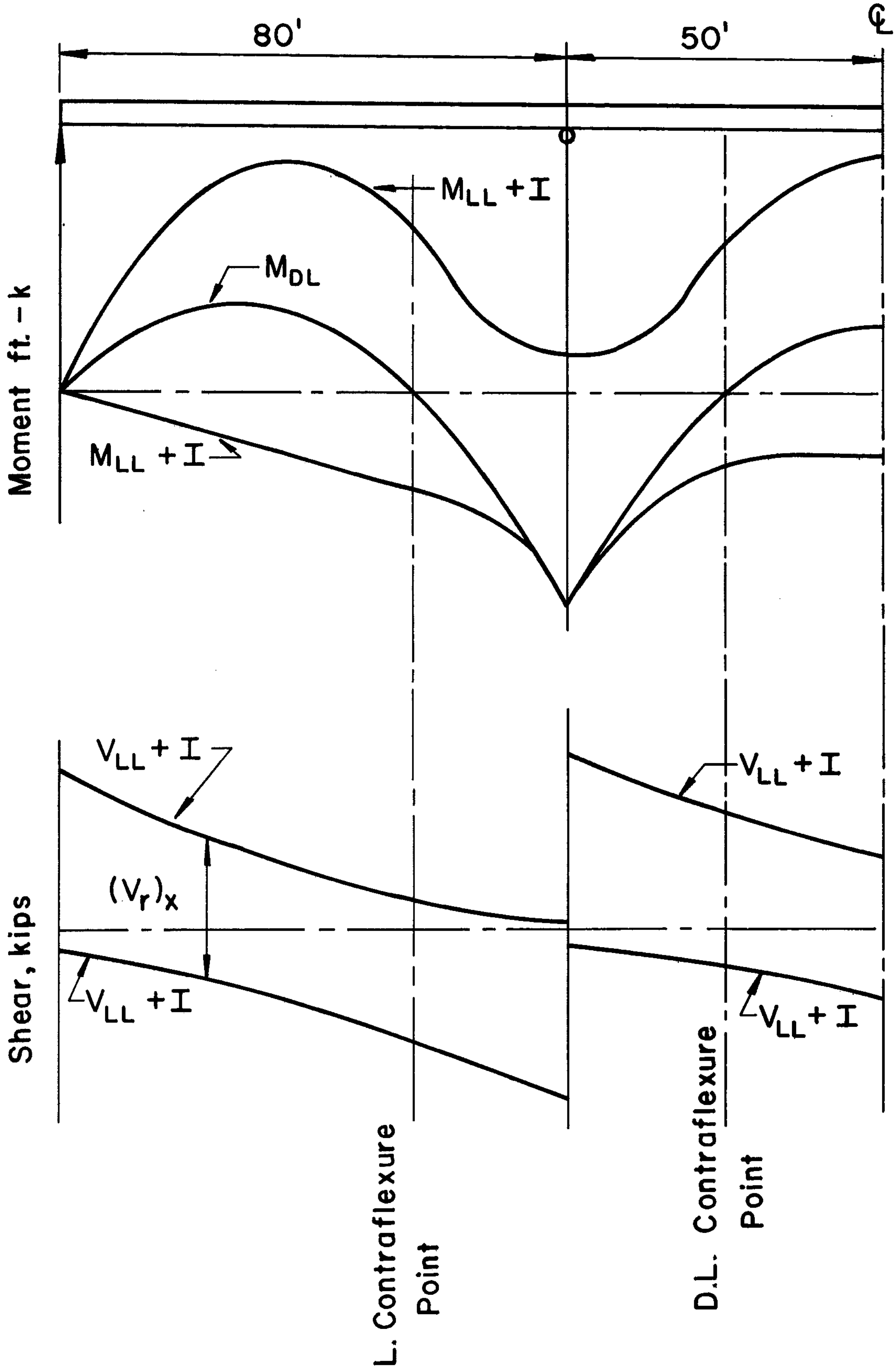


Fig. 32 RANGE OF SHEAR AND MOMENT DIAGRAMS FOR A CONTINUOUS BEAM

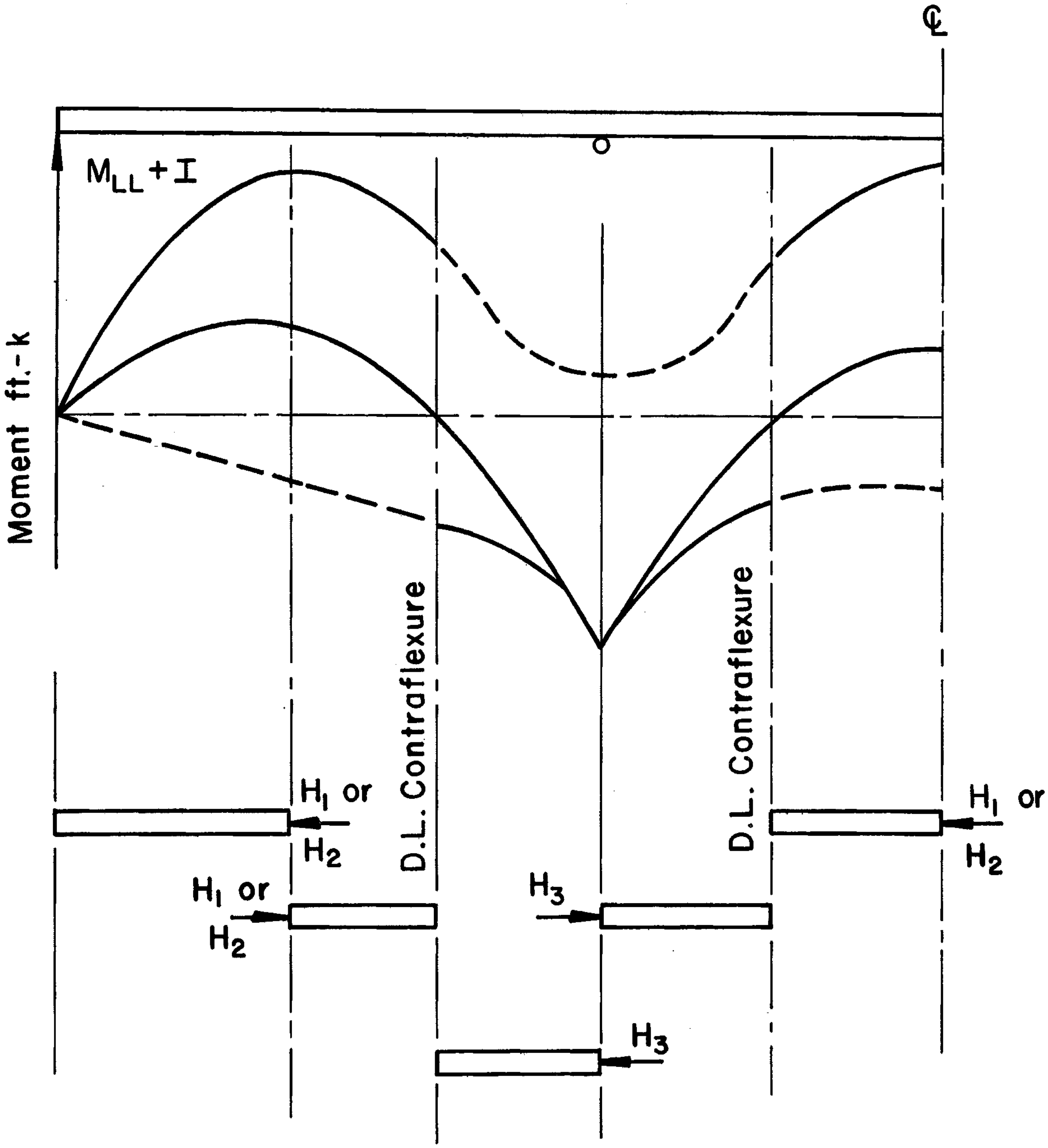


Fig. 33 FORCES IN CONCRETE SLAB OF CONTINUOUS BEAM AT ULTIMATE MOMENT

REFERENCES

1. I. M. Viest
   REVIEW OF RESEARCH ON COMPOSITE STEEL-CONCRETE BEAMS,
   Proc. ASCE, Vol. 86 (ST6) p. 1, June 1960

2. AASHO
   STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES,
   Washington, D. C., 1944

3. AASHO
   STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES,
   Washington, D. C., 1957

4. I. M. Viest, R. S. Fountain, and C. P. Siess
   DEVELOPMENT OF THE NEW AASHO SPECIFICATIONS FOR COMPOSITE STEEL AND CONCRETE BRIDGES,
   Highway Research Board Bulletin 174, National Academy of Sciences, Washington, D. C., 1958

5. C. P. Siess, I. M. Viest, and N. M. Newmark
   STUDIES OF SLAB AND BEAM HIGHWAY BRIDGES, PART III, SMALL SCALE TESTS OF SHEAR CONNECTORS AND COMPOSITE T-BEAMS,
   University of Illinois Engr. Exp. Sta. Bull. No. 396, 1952

6. N. M. Newmark, J. H. Appleton, C. P. Siess, and I. M. Viest
   FULL SCALE TESTS OF CHANNEL SHEAR CONNECTORS AND COMPOSITE T-BEAMS, PART IV,
   University of Illinois Engr. Exp. Sta. Bull. No. 405, 1952

7. D. C. King, R. G. Slutter, and G. C. Driscoll, Jr.
   FATIGUE STRENGTH OF 1/2 INCH DIAMETER STUD SHEAR CONNECTORS,
   Highway Research Record No. 103, Highway Research Board, National Academy of Sciences, Washington, D. C., 1965

8. A. A. Toprac
   FATIGUE STRENGTH OF 3/4 INCH STUD SHEAR CONNECTORS,
   Highway Research Record No. 103, Highway Research Board, National Academy of Sciences, Washington, D. C., 1965

9. H. M. MacKay, P. Gillespie, and C. Leluan
   REPORT ON THE STRENGTH OF STEEL I-BEAMS HAUNCHED WITH CONCRETE,
   Engineering Journal, Eng. Inst. of Canada, Vol. 6, No. 8, pp. 365-369, 1923

10. A. Voellmy
STRENGTH OF ALPHA COMPOSITE SECTION UNDER STATIC AND DYNAMIC STRESSES,
Unpublished report, Swiss Federal Materials Testing Lab., EMPA, Zurich, 1936

11. M. Ros
TRAGER IN VERBUNDBAUWEISE,
Report No. 149, Swiss Federal Materials Testing Lab., EMPA, Zurich, 1944

12. B. Thürlimann
COMPOSITE BEAMS WITH STUD SHEAR CONNECTORS,
Bulletin 174 Highway Research Board, pp. 18-38
Washington, D. C., 1958

13. B. Thürlimann
FATIGUE AND STATIC STRENGTH OF STUD SHEAR CONNECTORS,
Journal ACI, Vol. 30, No. 12, pp. 1287-1302, 1959

14. C. Culver and R. Coston
TESTS OF COMPOSITE BEAMS WITH STUD SHEAR CONNECTORS,
Proc. Structural Div. ASCE, Vol. 87, No. ST2, February, 1961

15. G. M. Sinclair
FATIGUE STRENGTH OF 3/4 INCH WELDED STUD SHEAR CONNECTORS,
Nelson Stud Welding Engineering Test Data,
Lorain, Ohio, September 1955

16. K. H. Dia and C. P. Siess
ANALYTICAL STUDY OF COMPOSITE BEAMS WITH INELASTIC SHEAR CONNECTION,
University of Illinois, Urbana, Illinois, June 1963

17. H. Proctor
ANALYTICAL AND EXPERIMENTAL STUDY OF LIGHTWEIGHT CONCRETE-STEEL COMPOSITE BEAMS,
M.S. Thesis - University of Missouri, August 1963

18. R. G. Slutter and G. C. Driscoll, Jr.
FLEXURAL STRENGTH OF STEEL-CONCRETE COMPOSITE BEAMS,
Journal, Structural Division ASCE, Vol. 91, ST2, April 1965

19. J. W. Fisher and I. M. Viest
FATIGUE LIFE OF BRIDGE BEAMS SUBJECTED TO CONTROLLED TRUCK TRAFFIC,
International Association for Bridge and Structural Engineering, Rio de Janeiro, 1964

20. J. C. Chapman
THE BEHAVIOR OF COMPOSITE BEAMS IN STEEL AND CONCRETE,
The Structural Engineer, Vol. 42, No. 4, April 1964

21. H. Wippel
BERECHNUNG VON VERBUNDKONSTRUKTIONEN AUS STAHL UND BETON,
Springer-Verlag, Berlin, 1963

22. W. M. Wilson, W. H. Bruckner, J. E. Duberg, and H. C. Beede
FATIGUE STRENGTH OF FILLET-WELD AND PLUG-WELD CONNECTIONS IN STEEL STRUCTURAL MEMBERS,
University of Illinois Engineering Experiment Station Bulletin No. 350, 1944

23. M. Hetenyi
BEAMS ON ELASTIC FOUNDATION,
The University of Michigan Press, Ann Arbor, 1946

24. F. E. Richart, A. Brandtzaeq, and R. L. Brown
A STUDY OF THE FAILURE OF CONCRETE UNDER COMBINED COMPRESSIVE STRESSES,
University of Illinois Engineering Experiment Station Bulletin No. 185, November 1928

25. W. Shelson
BEARING CAPACITY OF CONCRETE,
Journal of the American Concrete Institute, Vol. 29, No. 5, November 1957

26. O. H. Basquin
THE EXPONENTIAL LAW OF ENDURANCE TESTS,
Proc. American Society for Testing Materials, Vol. X, 1910

27. W. Weibull
THE PHENOMENON OF RUPTURE IN SOLIDS,
Handlingar Ingeniors Vetenshaps Akademien, No. 153, 1939

28. Bureau of Public Roads
STANDARD PLANS FOR HIGHWAY BRIDGES,
U. S. Department of Commerce, Washington, D. C., June 1952

29. W. M. Wilson, W. H. Munse, and W. H. Bruckner
FATIGUE STRENGTH OF FILLET-WELD, PLUG-WELD, AND SLOT-WELD JOINTS CONNECTING STEEL STRUCTURAL MEMBERS,
University of Illinois Engineering Experiment Station Bulletin No. 380, 1949

30. G. G. Goble
SHEAR STRENGTH OF COMPOSITE THIN FLANGE SPECIMENS,
Unpublished report, Case Institute of Technology, Cleveland, 1965

31. M. A. Miner
CUMULATIVE DAMAGE IN FATIGUE,
J. of Applied Mechanics, Vol. 12, No. 3, September 1945

32. J. W. Fisher
THE EFFECT OF WELDMENTS ON THE FATIGUE STRENGTH OF STEEL BEAMS,
Fritz Engineering Laboratory Report 354.248, Lehigh University, 1966

33. H. S. Reemsnyder
FATIGUE STRENGTH OF LONGITUDINAL FILLET WELDMENTS IN USS "T-1" CONSTRUCTIONAL ALLOY STEEL,
Fritz Engineering Laboratory Report No. 284.6, December 1963

34. R. G. Slutter and J. W. Fisher
A PROPOSED PROCEDURE FOR THE DESIGN OF SHEAR CONNECTORS IN COMPOSITE BEAMS,
Fritz Engineering Laboratory Report No. 316.4, Lehigh University, March 1966

35. AASHO
STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES,
Washington, D. C., 1966

36. C. Culver, P. J. Zarzeczny, and G. C. Driscoll, Jr.
COMPOSITE DESIGN FOR BUILDINGS,
Progress Report No. 2, Fritz Engineering Laboratory Report No. 279.6, Lehigh University, January 1961

37. W. H. Birkeland
DIFFERENTIAL SHRINKAGE IN COMPOSITE BEAMS,
Journal ACI, Vol. 56, pp. 1123-1136, May 1960

38. D. E. Branson, A. M. Ozell, A. L. Miller, H. H. Newland, A. Zaslavsky, W. Zuk, and H. W. Birkeland,
Discussion of DIFFERENTIAL SHRINKAGE IN COMPOSITE BEAMS,
Journal ACI, Vol. 56, pp. 1529-1558, December 1960

39. W. Zuk
THERMAL AND SHRINKAGE STRESSES IN CONCRETE BEAMS,
Journal ACI, Vol. 58, pp. 327-338, October 1961

40. P. E. Branson and A. M. Ozell
A REPORT ON DIFFERENTIAL SHRINKAGE IN COMPOSITE PRESTRESSED CONCRETE BEAMS,
Journal PCI, Vol. 4, No. 5, pp. 61-79, 1959

41. D. E. Branson
TIME-DEPENDENT EFFECTS IN COMPOSITE CONCRETE BEAMS,
Journal ACI, Vol. 61, No. 2, February 1964

42. J. C. Badoux and C. L. Hulsbos
HORIZONTAL SHEAR CONNECTION IN COMPOSITE CONCRETE BEAMS UNDER REPEATED LOADING,
Fritz Engineering Laboratory Report No. 306.1, Lehigh University, August 1965

43. J. W. Baldwin, Jr., J. R. Henry, and C. M. Sweeney
STUDY OF COMPOSITE BRIDGE STRINGERS, PHASE II,
University of Missouri, May 1965

44. J. C. Chapman and S. Balakrishman
EXPERIMENTS ON COMPOSITE BEAMS,
The Structural Engineer, Vol. 42, No. 11, November 1964

45. G. B. Godfrey
COMPOSITE BEAMS FOR STRUCTURES, Directives for Calculations in Design and Supplements and Explanations, German Standard Specification DIN 4239 (translation)
Civil Engineering, January 1960 (London)

46. VERBUNDTRAGER-STRASSENBRUCKEN RICNTLINIEN FUR DIE BERECHNUNG AND AUSBILDUNG, DIN 1078,
Stahlbau-Verlags-Grub H. Koln, Berlin 1959

47. COMPOSITE CONSTRUCTION IN STRUCTURAL STEEL AND CONCRETE,
British Standard Code of Practice, Part 2,
The Council for Codes of Practice, British Standards Institution, 1966 (Tentative)

48. COMPOSITE CONSTRUCTION IN STRUCTURAL STEEL AND CONCRETE,
British Standard Code of Practice CP117, Part 1,
The Council for Codes of Practice, British Standards Institution, 1965

49. H. G. Lehman, H. S. Lew, and A. A. Toprac
FATIGUE STRENGTH OF 3/4 INCH STUDS IN LIGHTWEIGHT CONCRETE,
Center for Highway Research, University of Texas, May 1965

50. R. I. Lewis and J. B. Menzies
COMPOSITE CONSTRUCTION IN STRUCTURAL STEEL AND CONCRETE: THE STATIC AND FATIGUE STRENGTHS OF SHEAR CONNECTORS (2) PUSHOUT TESTS OF HEADED STUD CONNECTORS,
Building Research Station, Ministry of Technology, 1965 (London)

51. V. Vitals, R. J. Clifton, and Tung Au
ANALYSIS OF COMPOSITE BEAM BRIDGES BY ORTHOTROPIC PLATE THEORY,
Journal of Structural Division ASCE, Vol. 89, No. ST4, August 1963

52. Kuang-Han Chu and G. Krishnamoorthy
DISCUSSION - ANALYSIS OF COMPOSITE BEAM BRIDGES BY ORTHOTROPIC PLATE THEORY,
Journal of Structural Division ASCE, Vol. 90, No. ST1, February 1964

53. K. Sattler
COMPOSITE CONSTRUCTION IN THEORY AND PRACTICE,
The Structural Engineer, Vol. 39, No. 4, April 1961

V I T A

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