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A PROPOSED PROCEDURE FOR THE DESIGN OF SHEAR CONNECTORS IN COMPOSITE BEAMS

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and
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This work was carried out as part of the Investigation of Shear Connector Design for Highway Bridges sponsored by the American Iron and Steel Institute.

Fritz Engineering Laboratory
Department of Civil Engineering
Lehigh University
Bethlehem, Pennsylvania

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1. INTRODUCTION

The current design procedure for shear connectors[1] for composite steel and concrete bridge members is based on the static properties of connectors.[2] Recently, attention has been focused on the fatigue properties of shear connectors.[3,4] The new studies indicated that the current design procedure is conservative and that additional economies may be attained by examining separately the static and the fatigue behavior of connectors.[3,4,5]

The results of fatigue tests of composite beams at the University of Texas,[3] Lehigh University,[4] and the University of Illinois[6] have shown that fatigue failure of connectors can be prevented by limiting the magnitude of slip which is the basis of the present design method. However, a more accurate method for preventing fatigue failure of connectors is to limit the magnitude of the maximum shear in the connector at working load. This latter approach is followed in this study.

The second consideration in the design of connectors is concerned with safety against infrequent loads. Failure of connectors on overloading is prevented by providing a sufficient number of connectors so that the composite beam can develop its static ultimate flexural strength. This principle is followed in the current as well as in the proposed design procedure.

The procedure outlined in this paper pertains only to the design of shear connectors; the general provisions of the AASHO Specifications for the design of steel and concrete composite beams may be used in conjunction with it. That is, the cross section of the member may be proportioned by the elastic method in current use. The proposed design procedure results in a smaller number and a different arrangement of connectors as compared to present AASHO designs. The procedure is based on the results of a recent investigation, which provided the data for evaluating the fatigue strength of stud and channel shear connectors,[7] as well as on the results of several earlier studies.[3,4,5,6]

No reference is made to spiral connectors as no new information is available and they are seldom used.

2. DESIGN PROCEDURE

A. Fatigue Considerations

1. For simple beams and for the regions of positive moment in continuous beams, compute the range of horizontal shear from

$$S_r = \frac{V_r Q}{I} \tag{1}$$

in which S_r = the range of horizontal shear per inch of length at the junction of the slab and girder.

V_r = the range of shear due to live loads and impact. At any section the range of shear may be taken as the difference between the maximum and minimum shear envelopes.

Q = the statical moment of the transformed compressive concrete area about the neutral axis of the composite section or the statical moment of the area of reinforcement embedded in the concrete for negative moment.

I = the moment of inertia of the transformed composite girder in positive moment regions and the moment of inertia provided by the steel beam and the area of reinforcement embedded in the concrete in negative moment regions.

The horizontal range of shear for the positive moment regions of continuous beams and simple beams should be computed at the supports or points of dead load contraflexure, and at the midpoint of positive moment regions. (The midpoint of the positive moment region is at midspan for simple span beams and midway between the exterior supports and dead load point of contraflexure in continuous beams.)

For the negative moment regions of continuous beams, compute the range of horizontal shear from Eq. 1 where the value of Q will be the statical moment of the reinforcing steel and the moment of inertia will be that of the steel beam and the reinforcing steel. Compute these values at the points of contraflexure and at the interior supports.

2. Determine the spacing in the outer quarters of the positive moment regions using the range of horizontal shear at the supports or points of contraflexure. The spacing is given by

$$P = \frac{\Sigma Z_r}{S_r} \qquad (2)$$

where S_r = the horizontal range of shear at the supports or points of contraflexure as calculated from Eq. 1.

Z_r = the allowable design range of load in pounds of an individual shear connector (per stud or inch of channel). Allowable values are listed in Table 1. (Note ΣZ_r is the resistance of all connectors at one transverse cross section of the girder.)

P = spacing of shear connectors.

Determine the spacing in the interior half of the positive moment regions from Eq. 2 using the range of horizontal shear computed from Eq. 1 at the midpoint of the positive moment region.*

Determine the spacing in the negative moment regions from the points of contraflexure for half the distance to an interior support from Eq. 2 using the range of horizontal shear at the points of contraflexure. (The horizontal shear values are computed from the Q and I values for the negative moment region described in Step 1.) Determine the spacing in the negative moment regions from the interior

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* An alternate more conservative, yet simpler procedure for the positive moment regions is to consider only the range of shear at the supports of simple beams, and at the exterior supports and points of dead load contraflexure of continuous beams. If these maximum values of range of shear are applied throughout the appropriate span length, a uniform spacing of the shear connectors will result for simple beams and for each positive moment region of continuous beams.

support for the remaining length from the range of horizontal shear at the interior support.**

3. Equation 2 will determine the spacing in most designs. The spacing of connectors should never exceed 24 inches in positive moment regions because connectors also perform the necessary function of holding the concrete slab in contact with the steel beam.

In negative moment regions separation of the slab from the beam is not possible, however, connectors are needed to provide resistance to the range of shear to which they are subjected and to provide a more effective section to distribute the shrinkage forces. The placement of connectors on the tension flange in the negative moment regions affects the fatigue strength of the section. The total number of connectors required to resist the range of shear will be more important than their actual spacing. Hence, the spacing of connectors in the negative moment region can be modified so that the fatigue strength of the beam flange would not govern the design.

B. Flexural Strength Requirements

4. The fourth step in the design procedure is to check whether sufficient connectors are provided for the development of the ultimate flexural strength of the composite section.

** If the range of shear in the regions of negative moments of continuous beams does not vary greatly, an alternate more conservative procedure would result by taking the maximum range of horizontal shear at an interior support. Then determine the spacing throughout the negative moment region between dead load points of contraflexure on either side of an interior support by using this range of shear.

First, determine the value of the force in the concrete slab which must be resisted when the flexural strength is reached. For simple beams and continuous beams between the point of maximum positive moment and the outside supports and the points of dead load contraflexure, take the smaller value obtained from Eqs. 3 and 4.

$$H_1 = A_s F_y \tag{3}$$

$$H_2 = 0.85\ f'_c bc \tag{4}$$

where, A_s = total area of the steel section including coverplates.

F_y = specified yield point of the steel section.

f'_c = compressive strength of concrete at age of 28 days.

b = effective width of concrete slab.

c = thickness of the concrete slab.

For continuous beams between the dead load point of contraflexure and the adjacent interior support determine the maximum horizontal force from Eq. 5.

$$H_3 = A^r_s F^r_y \tag{5}$$

where, A^r_s = total area of longitudinal reinforcing steel at the interior support within the effective flange width.

F^r_y = specified yield point of the reinforcing steel.

The minimum number of shear connectors required between the points of maximum positive moment and the end supports or dead load points of contraflexure, and between points of maximum negative moment and the dead load points of contraflexure is given by Eq. 6.

$$N_i = \frac{H_i}{\phi Q_u} \qquad (6)$$

where N_i = the minimum number of shear connectors between points of maximum positive moment and adjacent end supports or dead load points of contraflexure or between points of maximum negative moment and adjacent dead load points of contraflexure.

ϕ = a reduction factor = 0.85.

Q_u = ultimate shear connector loads given in Table 2.

H_i = smaller value of H_1 and H_2 (Eqs. 3 or 4) for simple beams; continuous bemas between points of maximum positive moment and the end supports; and continuous beams between points of maximum positive moment and points of dead load contraflexure.

= H_3 (Eq. 5) for continuous beams between points of dead load contraflexure and interior supports.

If the number of shear connectors given by Eq. 6 exceeds the number provided by the spacing given by Eq. 2, additional connectors should be added to ensure that the ultimate strength is achieved.

3. DISCUSSION OF DESIGN PROCEDURE

A. Fatigue Consideration

Fatigue is of concern under repeated applications of working load. It is well established that repeated application of loading can lead to fatigue fracture. Since highway structures are subjected to repeated loads, consideration must be given to the fatigue behavior of the shear connection in composite beams.

The magnitude of the shear forces transmitted by individual shear connectors at working loads has been found in a reasonable agreement with values predicted by the elastic theory assuming complete interaction. Tests have indicated that the difference between the computed values based on complete interaction and the experimental measurements is small. Although these measurements have indicated that connectors in regions of constant shear may not transmit equal forces, the maximum stress on any one shear connector seldom exceeds the value predicted from elastic theory assuming complete interaction. Hence, elastic theory assuming complete interaction can be used to evaluate the horizontal shear stress resisted by the shear connection.

The horizontal shear to be transferred by the shear connectors can be computed from Eq. 7.

$$S = \frac{VQ}{I} \qquad (7)$$

where S = horizontal shear per inch of length.

V = shear in kips acting on the composite section.

Q = statical moment of the transformed compressive area about the neutral axis of the composite section, $in.^4$

I = moment of inertia at the composite section, $in.^4$

In regions of negative moment in continuous beams, the value of **Q** is the statical moment of the reinforcing steel and the moment of inertia is that of the steel beam and the reinforcing steel.* In negative moment regions with continuous reinforcement, flexural conformance and action under working loads produces tensile stresses which are sufficiently large to cause cracking of the slab. Also, with passage of time, shrinkage will occur and hairline cracks will form. The composite bridges at the AASHO Test Road showed that even with large numbers of shear connectors, transverse shrinkage cracks formed in the slabs of simple beams which allowed passage of water through the slab.[8] Hence, it appears reasonable to only consider the cracked section of the concrete slab. It should also be noted that under initial loading when the slab may remain uncracked, in all probability high friction forces can be developed due to bond between the steel beam and the concrete slab. Hence, the connectors would not be required to transmit the greater range of shear alone. After cracking this frictional force is reduced with continued application of loads as the bond is destroyed.

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* If an expansion joint is provided at the interior supports of continuous beams so that the steel reinforcement is cut, it is apparent that the force in the reinforcement cannot be developed until flexural conformance is achieved some distance from the joint. Also, when expansion joints are used, the stress in the tension flange of the steel section at the interior supports will usually require the use of coverplates. Since the number of connectors required to develop flexural conformance is not known and connectors would be required near the support with a deleterious effect on the fatigue strength of the steel section, it is recommended that connectors not be used in the negative moment region when expansion joints are provided.

Placing the shear connectors in the negative moment regions should also assist in maintaining flexural conformance throughout the continuous beam. This also prevents the sudden transition from a composite to a non-composite section when they are omitted. Their placement should minimize the large differentials in deformation that might otherwise occur (as in a coverplated beam) and reduces the danger of fatigue failure in connectors adjacent to the negative moment region.

An assessment of the fatigue behavior of various welded details has indicated that minimum stress has a negligible effect on the fatigue strength.[9] Although the fatigue testing of composite beams was generally for a zero-to-maximum loading,[3,4] the study reported in Ref. 7 has shown that minimum stress was not a significant parameter except for the case of stress reversal. This study showed clearly that the magnitude of the fluctuating stress referred to as the stress range accounted for the fatigue strength of the connectors and that it was conservative to neglect minimum stress in the case of stress reversal.

In simple span beams the range of shear stress throughout the span is dependent on the length of span and the type of loading. For spans up to about 70 feet the range of shear varies from a maximum at the end of the span to about 85% of the maximum near midspan. For longer spans this variation is not nearly as great so that the range of shear is nearly constant throughout the span. This is illustrated for spans of 50, 70, and 90 feet by the shear envelopes plotted in Fig. 1.

At the supports, the horizontal range of shear computed from Eq. 1 varies from zero to a maximum value as the live load moves onto the span. As is readily apparent from the shear envelopes plotted in Fig. 1 for simple beams, the range of horizontal shear will vary from zero-to-maximum at the supports to near full reversal at midspan. The dashed curves in Fig. 1 indicate the maximum shear envelopes for loads moving in the opposite direction. At any section along the span the range of shear is the difference between the maximum and minimum shear envelopes and is indicated in Fig. 1 as $(V_r)_x$.

The situation represented by the two outer shear envelopes in Fig. 1 (the upper solid curves and the bottom dashed curves) is that of truck loads passing in both directions in the same lane. Although this is not a realistic condition, it is given here for convenience of discussion. The actual envelopes which apply to range of stress on connectors with traffic in one direction are the two solid or the two dashed curves, depending on the direction which the traffic is moving. The two outer shear envelopes could be used to establish a conservative approximation for the stress range throughout the span. For a 50 ft. span the resulting range of shear at midspan is approximately 84% of the range of shear at the support and for a 90 ft. span it is approximately 97% of the range of shear at the support. The difference in the resulting range of shear and the actual range of shear is usually less than 5% at midspan. This procedure is convenient to use since the actual range of shear is difficult to establish.

For design, several approximations are possible. An average of the actual range of shear at the support and at midspan could be used to ascertain the required number of shear connectors, where the range of shear is the difference in the minimum and maximum shear envelopes for passage of the vehicle. This is shown as $(V_r)_x$ in Fig. 1.

An alternate, more conservative yet simpler procedure would result by considering only the maximum shear at the support. In longer span bridges, the range of shear is more nearly uniform (see Fig. 1) than in the shorter spans so that such an approach would be more conservative for the short span structures. Using the shear at the support as the range of shear throughout the span results in a uniform spacing of the shear connectors. In many structures, the range of shear is nearly uniform and even if the actual shear range were used a nearly uniform spacing would result.

For continuous spans, the variation in the minimum-maximum shear envelopes along the lengths of the spans is usually somewhat greater than in simple spans. Figure 2 shows the moment and shear envelopes for a typical continuous bridge structure.* If the variation in the shear stress range is significant, a variable spacing of the connectors is necessary. The range of stress on the connectors in the positive moment regions can be determined in the same manner that was suggested for simple span structures. The appropriate shear range and the usual composite beam properties of the cross-section would be used.

In negative moment regions, the range of horizontal shear acting on the connectors is caused by the force in the reinforcing steel. This shear range can be evaluated from the shear envelopes $(V_r)_x$ and the cross-sectional properties of the beam in that region. As was noted,

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* Taken from Page 95 of Reference 10

the value of Q will be the statical moment of the area of reinforcing steel and the moment of inertia will be that of the steel beam and the reinforcing steel.

For unusual continuous span combinations, positive moments at an interior support may control and more shear connectors may be required to resist the resulting shear. In such instances, the range of shear would vary from zero to a maximum shear associated with the maximum positive moment. This condition only controls with three of four span continuous beams with odd span ratios such as 10:6:6 or 10:7:7:10.

B. Allowable Stresses for Fatigue Loading

The analysis of the data reported in Ref. 7 has provided a relationship between the applied stresses and the number of cycles to fatigue failure. Prior to this study insufficient fatigue data were available to ascertain the effect of variables such as minimum stress and concrete strength on the fatigue strength of shear connectors.

The analysis of the available laboratory fatigue tests reported in Ref. 7 has shown clearly that minimum stress was only significant for the case of stress reversal and that a conservative estimate of the fatigue life could be made by considering only the range of stress. The S-N curve for the experiment on stud connectors is shown in Fig. 3. The regression curve was obtained by considering only the 2 and 10 ksi minimum stress levels. It is readily apparent that such an analysis will provide a greater margin of safety for the case of stress reversal. This is not considered critical as most connectors will be subjected to a shear loading predominately in one direction. Also, if shrinkage should occur, connectors designed for stress reversal may have residual shear stresses such that little if any reversal of stress takes place.

Similar results were reported in Ref. 7 for channel shear connectors. It was also shown that the concrete strength did not significantly influence the fatigue strength of either the stud or the channel shear connectors.

Figure 4 compares the regression curve for the push-out specimens reported in Ref. 7 with the beam tests reported in Refs. 3 and 4. It is apparent that the lower limit of dispersion for the beam tests (taken as twice the standard error of estimate), overlaps the upper limit of dispersion for the push-out tests. Hence, the lower limit of dispersion of the beam tests is about equal to the mean behavior of the push-out specimens. This finding is reasonable because in the beam tests a loss of interaction was noted which allowed the connector forces to redistribute and resulted in a less severe stress condition than computed from elastic theory assuming complete interaction. In the push-out specimens the loading on the connectors was maintained at a reasonably constant level througout the cycle life. Push-out tests therefore represent a lower bound for connector failure.

Also, it should be noted that the failure criteria for the beam test results plotted in Fig. 4 was taken as the initial fatigue fracture of one or more connectors. The studies reported in Refs. 3 and 4 have shown clearly that the failure of the first connectors has little effect on the beam response and that considerable additional life was available before the beam failed. This is illustrated in Fig. 5 where the curve representing initial fatigue fracture of one of more connectors (plotted in Fig. 4) is compared with the curve relating the cycle life to failure of the connection in a beam as given in Ref. 3. It is apparent that considerable longer cycle life was available before the composite beam failed due to the weakened shear connection.

Because the push-out tests provide a lower bound of fatigue strength it seems satisfactory to consider the mean curve shown in Fig. 4 as the value for the design of stud shear connectors. A suitable design value can be obtained for any desired cycle life. For example, if the expected life is 2,000,000 cycles, the resulting allowable stress range is 10 ksi. The mean curve for push-out tests of stud connectors gives a suitable margin of safety with respect to beam test results.

On the basis of the mean curve in Fig. 4, the design formula for the allowable range of load can be obtained for stud connectors

$$Q_r = \alpha d^2 \tag{8}$$

where Q_r = allowable range of shear force per stud in pounds

d = diameter of stud in inches

α = 13,800 for 100,000 cycles

= 10,600 for 500,000 cycles

= 7,850 for 2,000,000 cycles

Equation 8 is applicable to 3/4 in. and 7/8 in. stud shear connectors.

An examination of Fig. 4 shows that it can be applied conservatively also to smaller diameter stud shear connectors.

For the channel shear connectors, the fatigue failure was generally initiated in one of the transverse fillet welds and propagated through the weld. It was apparent that the critical parameter was the stress on the throat of the connecting fillet welds. For standard channel sections the thickness of the channel web is always equal to or greater than the thickness at the toe of the channel flange which is uniform at 3/16 inches. Since the thickness at the toe governs the weld size, it is to be expected that similar behavior should occur in other channels assuming that the same size 3/16 in. fillet weld is placed at the heel and toe of the channel.

For standard channel sections one should not attempt to place larger welds to provide an increase in the fatigue strength. The beam tests with channel connectors reported in Ref. 6 showed that if larger welds were used then premature failure would occur in the channel web. Obviously an increase in the fatigue strength of channel connectors could only by achieved if larger welds were used with channels having thicker webs.

The data for the channel shear connectors reported in Ref. 7 is plotted in Fig. 6. For convenience the range of shear in kips per inch of channel width is plotted as a function of the cycle life. As was noted previously, the actual failure was due to fracture of the weld so that the actual measure of cycle life is the stress on the throat of the fillet welds. The load per inch of channel width was selected since all standard channels could be expected to behave in a similar manner if 3/16 in. fillet welds were placed at the heel and toe.

On the basis of the test data shown in Fig. 6, a tentative design formula for the allowable range of load for any desired cycle life can be obtained. The lower limit of dispersion was used because of the limited amount of test data and the absence of information on full size beams. Equation 9 is the result for channel shear connectors.

$$Q_r = \beta w \tag{9}$$

where Q_r = allowable range of shear in lbs. per inch of channel.

w = length of a channel shear connector in inches measured in a transverse direction on the flange of a beam.

β = 4000 for 100,000 cycles

= 3200 for 500,000 cycles

= 2600 for 2,000,000 cycles

Shrinkage of the concrete slab imposes forces on the connectors in addition to forces resulting from flexure. For connectors near the support these shrinkage forces are imposed in the direction opposite to flexural forces. The effect of shrinkage forces on the fatigue life of connectors is the same as a change in the minimum stress on the connector. The study reported in Ref. 7 has indicated that a variation in minimum stress over a wide range has little or no effect on the life of connectors. Also the presence of shrinkage forces does not reduce the ultimate strength of members which have flexible connectors. However, shrinkage forces may have to be considered in the design of rigid connectors.

C. Flexural Strength Requirements

The number of shear connectors furnished must assure not only an adequate fatigue strength but also insure that the flexural strength of the composite member can be developed. In most composite beams this second requirement will be satisfied because fatigue considerations are usually critical. However, when substantial dead load is carried by the composite section, the variation in shear stress acting on the connectors may be relatively small so that fatigue strength is not critical. Hence, the maximum shear stress due to dead load plus live load may be critical, so that the governing criterion may be the static ultimate strength. A limitation of the maximum shear stress acting on connectors is necessary for this situation.

Recent research has shown that the flexural strength of composite beams can be developed if sufficient connectors are provided to resist the maximum horizontal force in the slab.[5] This study also confirmed that connector spacing was not critical. Hence, the near uniform spacing which results from the fatigue requirement should also be satisfactory for the development of the ultimate flexural strength.*

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* A few tests have indicated that for beams with uniform connector spacing the maximum load can move into position to produce the flexural failure.[5] However, this problem is currently being studied in greater detail and test results should become available in the near future. Since fatigue governs in nearly all cases, this point is seldom critical.

At the ultimate moment of a composite beam, two stress distributions are possible as illustrated in Fig. 7. The studies reported in Ref. 5 have demonstrated that the horizontal force required for the determination of the number of shear connectors is the compressive force in the concrete slab when the fully plastic stress distribution for the ultimate flexural strength is reached. For the two cases possible, indicated in Fig. 7, the maximum horizontal force is given by

$$H_1 = A_s F_y \tag{3}$$

$$H_2 = 0.85\, f_c' bc \tag{4}$$

For any composite section the ultimate flexural strength will be governed by either Eq. 3 or 4. When the slab is large compared with the beam section, the yield strength of the steel section governs (Eq. 3). When the beam section is large compared with the slab, the ultimate compressive strength of the slab governs (Eq. 4). Obviously for any given composite beam the maximum possible compressive force in the concrete slab would necessarily have to be the smaller of the two values computed from Eqs. 3 and 4. Hence, between a point of maximum positive moment and the end of a beam or point of dead load contraflexure, sufficient connectors should be provided to resist the smaller value given by Eqs. 3 and 4.

For continuous beams an additional horizontal force in the slab between a point of dead load contraflexure and an adjacent interior support must be resisted as indicated in Fig. 8.

Only the yield strength of the reinforcing steel needs to be resisted at the ultimate moment. As the plastic moments of the continuous beam are approached and hinges develop, extensive cracks form over the supports of continuous composite beams. Therefore, in continuous beams the portion of the beam between a point of contraflexure and a point of maximum negative moment must be provided with sufficient shear connectors to resist the horizontal force (H_3) equal to the yield strength of the longitudinal reinforcement of the slab:

$$H_3 = A_s^r F_y^r \qquad (5)$$

It was shown in Ref. 5 that the ultimate strength of stud and channel shear connectors is given by the following expressions:

$$\text{Stud Connectors} \quad Q_u = 930\ d^2 \sqrt{f'_c} \qquad (10)$$

$$\text{Channel Connectors} \quad Q_u = 550\ (h + 0.5\ t)\ w \sqrt{f'_c} \qquad (11)$$

where Q_u = ultimate shear strength in pounds

d = diameter of studs in inches

f'_c = compressive strength of the concrete slab, psi

h = average flange thickness of channel, inches

t = web thickness of channel, inches

To insure the development of the ultimate flexural strength of composite beams, a larger margin of safety against connector failure should be provided than is provided for the beam. Historically, the factor of safety for connections and fasteners has been larger than for the connected members. This assures that the connections do not fail before the main members. This margin can be accomplished by providing a load reduction factor (Ø) to the ultimate shear strength of the shear connector. A Ø value of 0.85 appears to be reasonable. Since the ultimate flexural capacity of composite bridge beams is usually 2.5 or greater than the working load moment, the corresponding margin for the shear connection would be approximately 3 or greater.

The tests reported in Ref. 5 have demonstrated that only a slight deformation in the concrete near the more heavily stressed connectors is needed to redistribute the horizontal shear to other less heavily stressed connectors. A few tests were reported in Ref. 11 in which the loading positions for two point loading were changed in successive tests by moving the loads toward the supports. These tests have indicated that the maximum load can most likely move into the position of maximum moment without premature failure in a more heavily stressed connector because of these redistribution characteristics.[5,11]

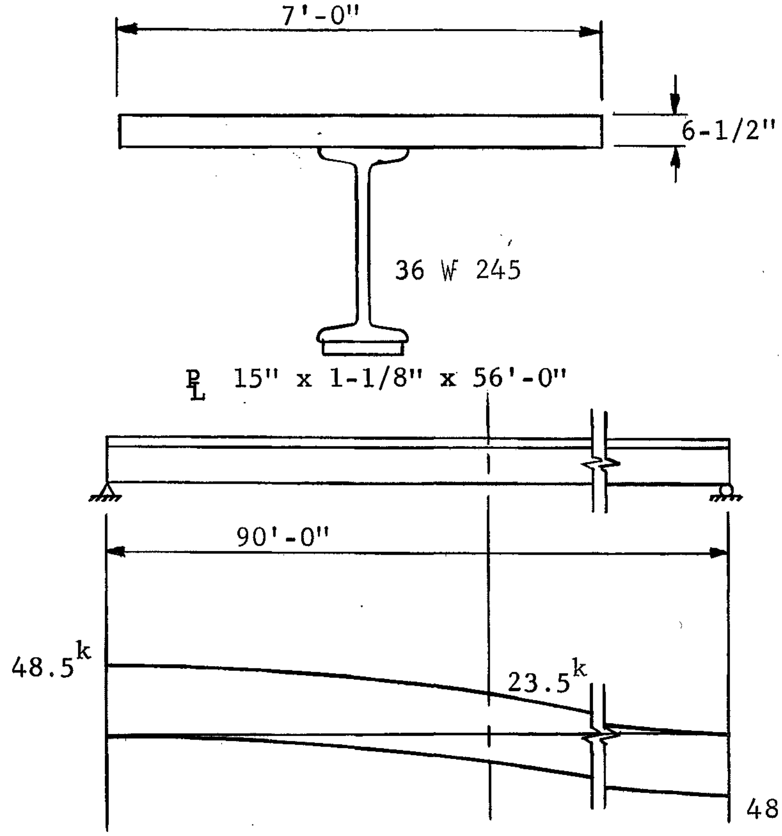
It should be noted that seldom will the maximum load criteria be the governing factor. The number of connectors required by the fatigue criterion will usually far exceed the requirements for ultimate flexural strength.

In instances where the maximum load criterion represents more critical condition, the use of the flexural strength requirements insures that sufficient connectors are present so that excessive local permanent deformation of the concrete in the vicinity of stud connectors are minimized. If this provision was not used local deterioration of the concrete could result which would adversely influence the fatigue strength of stud connectors. Channel shear connectors have sufficient bearing area so that it is doubtful that excessive local permanent deformation would occur.

In simple beams the stress range values for stud and channel connectors will govern the design and it would not ordinarily be necessary to calculate both values since the fatigue criterion is the more critical condition. This is also true in continuous beams for unshored construction.

4. DESIGN EXAMPLE

The proposed procedure for the design of shear connectors is illustrated in the following example for a 90 foot span composite beam with unshored construction and design for 2,000,000 cycles of loading in accord with Section 1.8.3 of the AASHO Specification.



$f'_c = 3$ ksi $n = 10$

Steel - ASTM A36

Live Load H20 S16

D.L. (Composite) = 448 lbs./ft.

$\frac{Q}{I} = 0.02117$ at support

$\frac{Q}{I} = 0.0196$ at midspan

Maximum Shear Envelope

Spacing in the exterior quarters of the span, from Eq. 1

$$S_r = 48.5\ (0.02177) = 1.056^{k/in.} \text{ at the support}$$

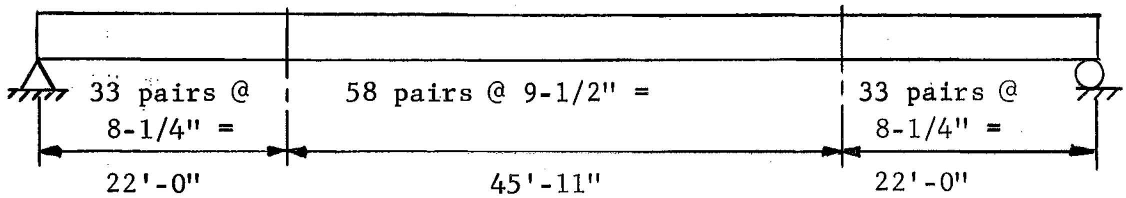
$$P = \frac{\Sigma Z_r}{S_r}$$

$$= \frac{2(4.4)}{1.056} = 8.33 \text{ inches for pairs of 3/4 inch diameter studs}$$

For the interior half of the beam Eq. 1 yields

$$S_r = (23.5 - (-23.5))(0.0196) = 0.9212 \text{ k/in.}$$

$$P = \frac{\Sigma Z_r}{S_r} = \frac{2(4.4)}{0.9212} = 9.55 \text{ inches for pairs of 3/4 in. studs}$$



Total number of connectors required = 248

When the more conservative procedure is used a uniform spacing of 8-1/4 in. would be used throughout the beam span. The resulting total number of connectors required would be 132 pairs or 264 connectors. Check number of connectors required for ultimate strength using Eqs. 3 and 4.

$$H_1 = A_s F_y = 88.91 (36) = 3200.8 \text{ kips}$$

$$H_2 = 0.85 f_c' bc = 0.85 (3) 84 (6.5) = 1392.3 \text{ kips}$$

Number of connectors required for half span

$$N = \frac{H_2}{\phi Q_u} = \frac{1392.3}{0.85 (28.7)} = 57 \text{ connectors}$$

Therefore, flexural strength requires only 114 connectors; fatigue governs.

Using the proposed design procedure for 3/4 inch diameter studs the member requires 248 connectors arranged in pairs with a spacing of 8-1/4 inches in the outer quarters of the span and 9-1/2 inches in the center of the span. The AASHO design using a factor of safety of 4.0 requires 436 connectors of the same size arranged four at a point with spacing varying from 6-3/4 inches at the support to 16-1/2 inches near midspan. If the factor of safety for the AASHO design were reduced to 3.0, 328 connectors would be required. The proposed procedure results in saving 43 percent of the number of connectors required by the AASHO design and a factor of safety of 4.0. The saving is reduced to 25 percent when comparing with an AASHO design having a factor of safety of 3.0.

5. CONCLUDING REMARKS

The uniform spacing of connectors in bridge members is a radical departure from the present designs. This results from the fact that a design procedure for static loading has been used for designs where fatigue criteria should govern. Since the current design procedure is usually conservative, fatigue failures have been prevented.

It is apparent from the study reported in Ref. 7 that allowable stresses cannot be selected for shear connectors based on static strength or a slip criterion alone. Even though the current design procedure is conservative, it is not possible to select an arbitrary, lower factor of safety and be assured that fatigue is not a problem. The use of a design concept which neglects fatigue could result in fatigue failure of shear connectors near midspan even in beams which contain more shear connectors in the complete span than are actually needed for a safe design with uniform spacing.

For simple spans the proposed design procedure will usually eliminate the undesirable variable spacing of shear connectors. Also, it affords economy in terms of the number of shear connectors required.

6. ACKNOWLEDGMENTS

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TABLE 1 VALUES OF ALLOWABLE RANGE OF LOAD

Type 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** (lbs./in. of length)	3 [4.1		4,000	3,200	2,600
	4 [5.4		4,000	3,200	2,600
	5 [6.7		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 2 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 [4.1	10,800	11,700	12,500
	4 [5.4	11,600	12,500	13,400
	5 [6.7	12,500	13,500	14,400

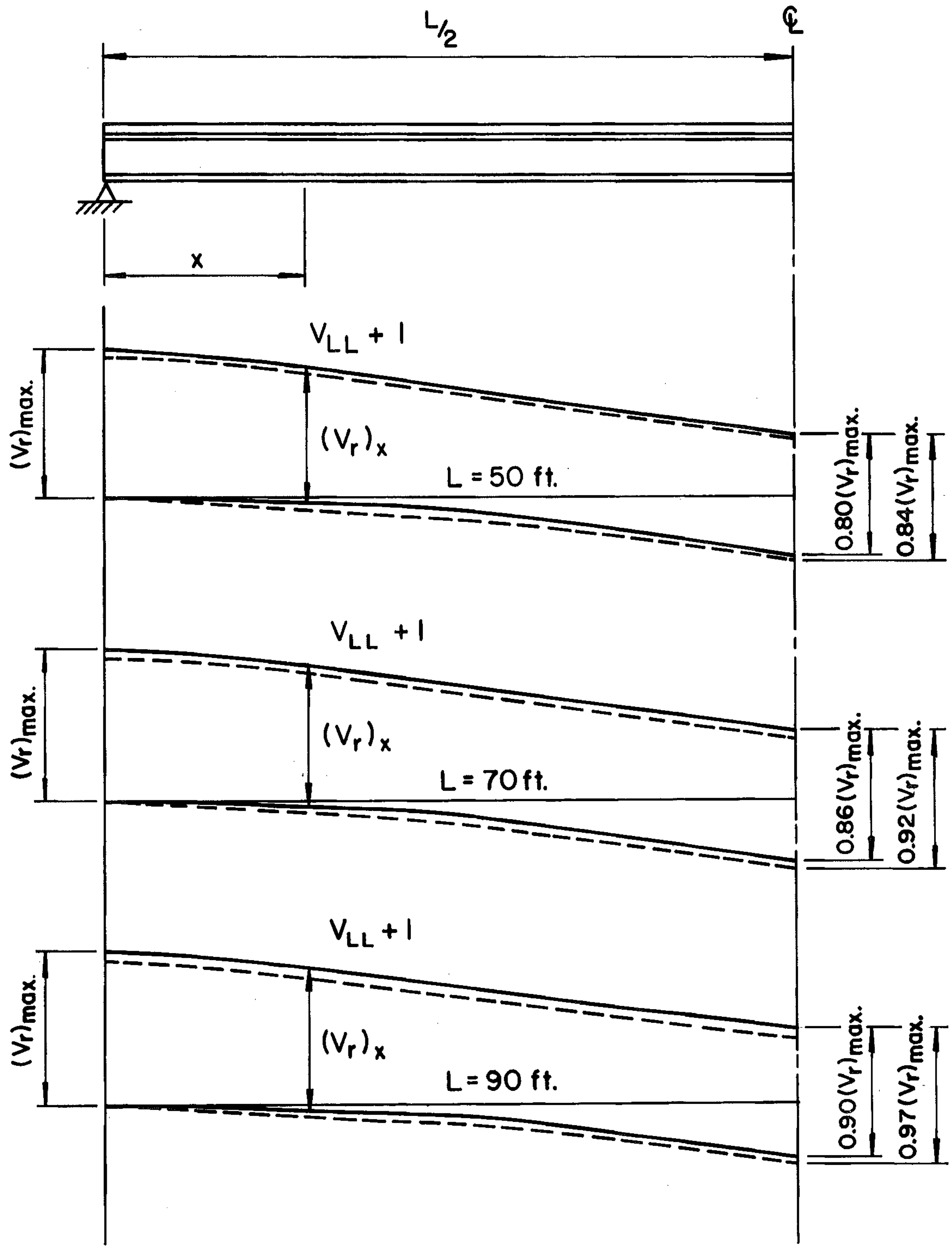

Fig. 1 TYPICAL SHEAR ENVELOPES FOR SIMPLE BEAMS

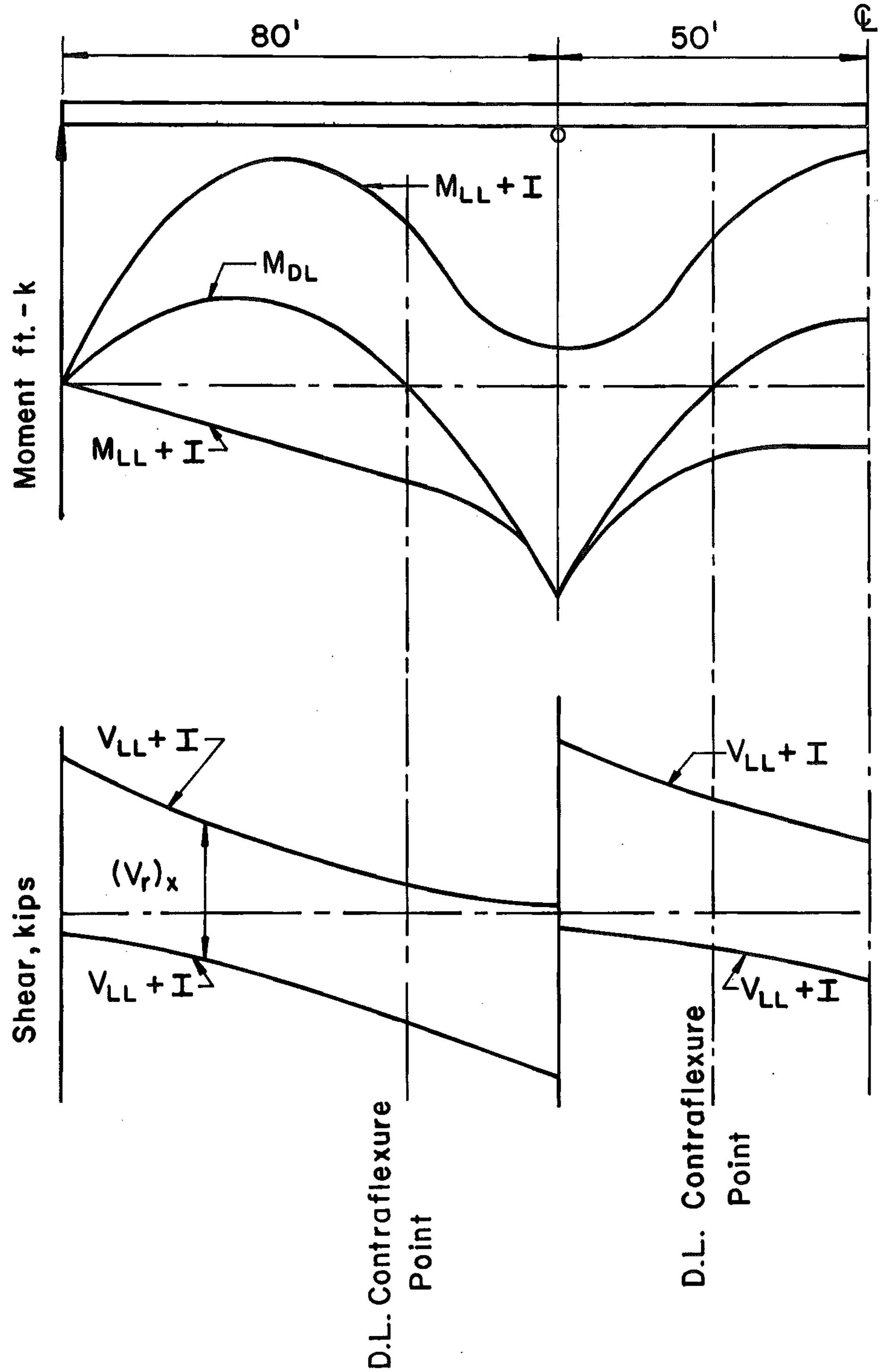

Fig. 2 TYPICAL SHEAR AND MOMENT ENVELOPES FOR CONTINUOUS BEAM

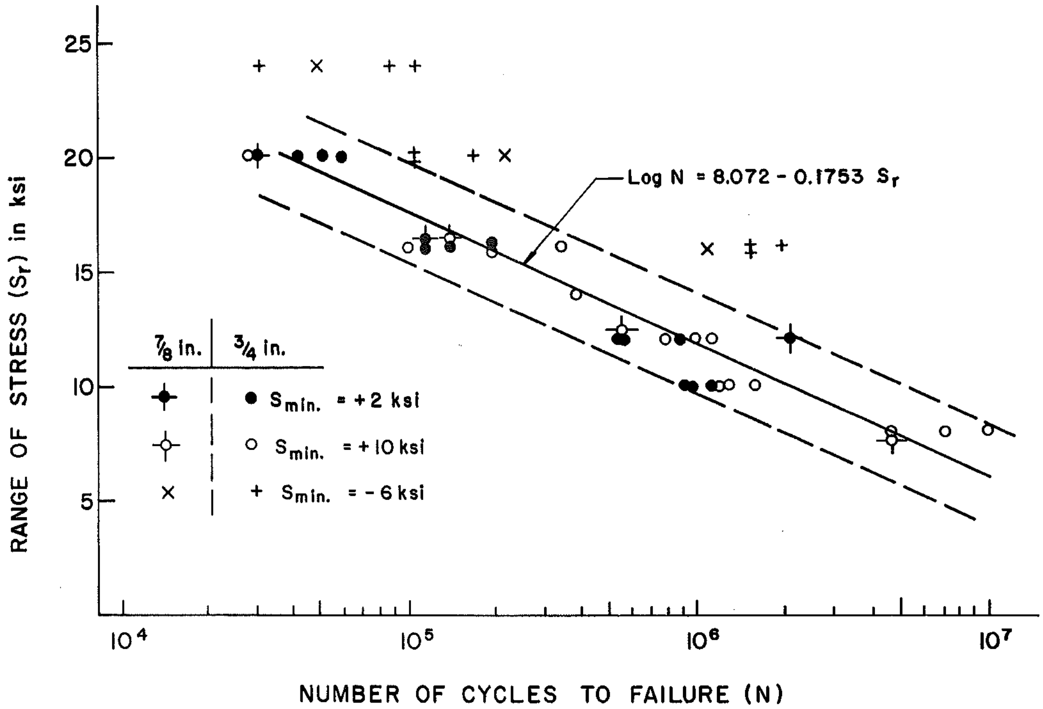

Fig. 3 S-N CURVE FOR STUD SHEAR CONNECTORS

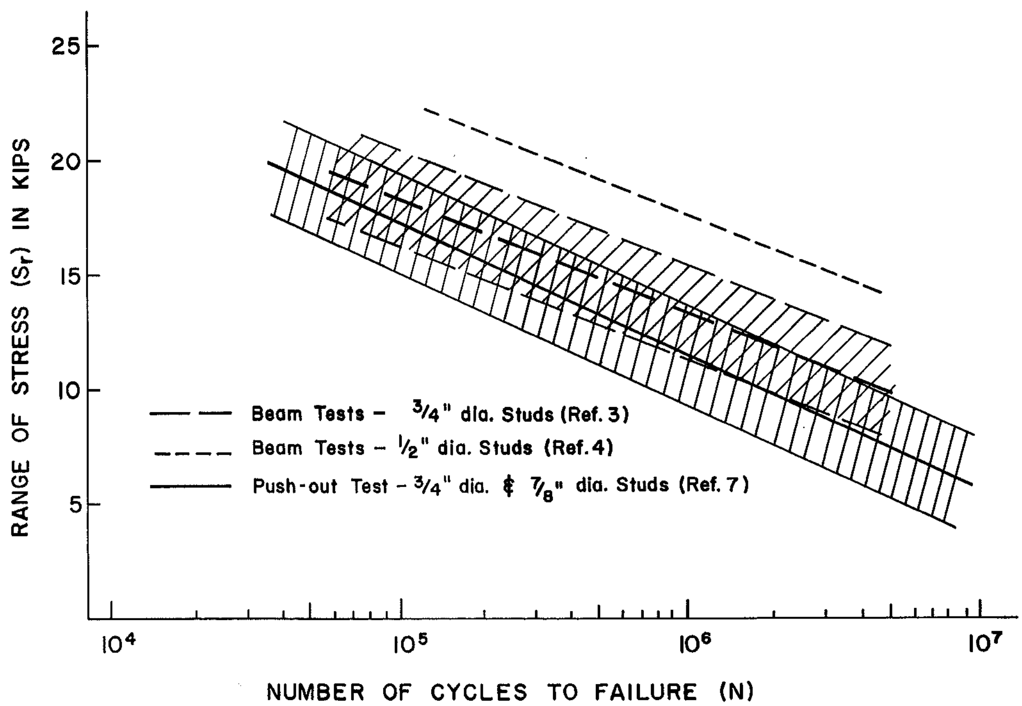

Fig. 4 COMPARISON OF S-N CURVES OF BEAM TESTS AND PUSHOUT TESTS

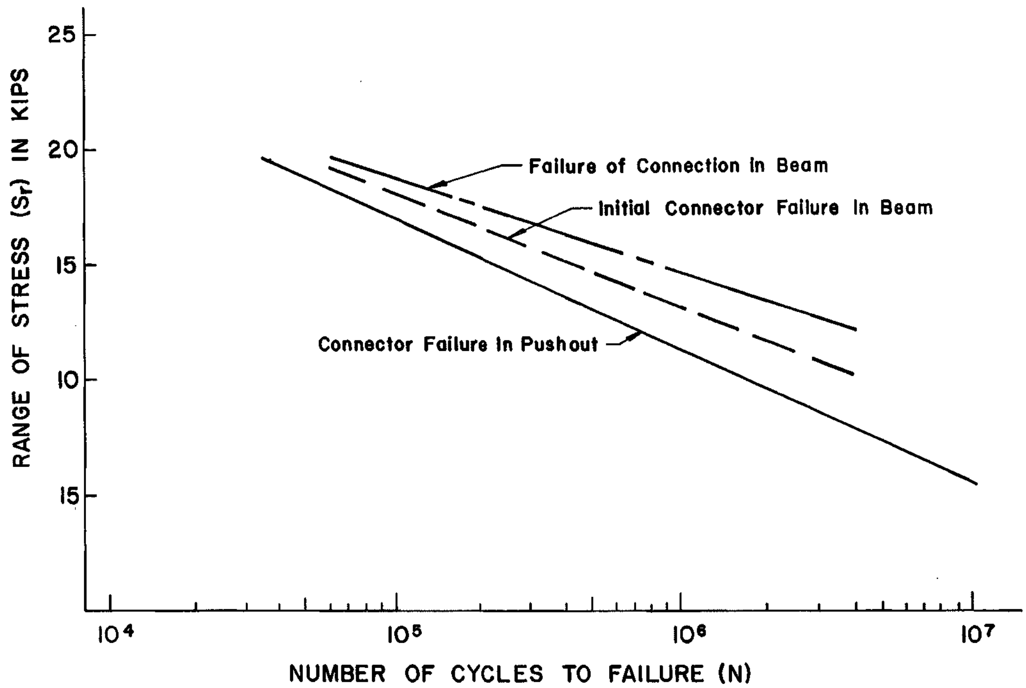

Fig. 5 COMPARISON OF INITIAL CONNECTOR FAILURE AND FINAL FAILURE OF SHEAR CONNECTION IN COMPOSITE BEAM

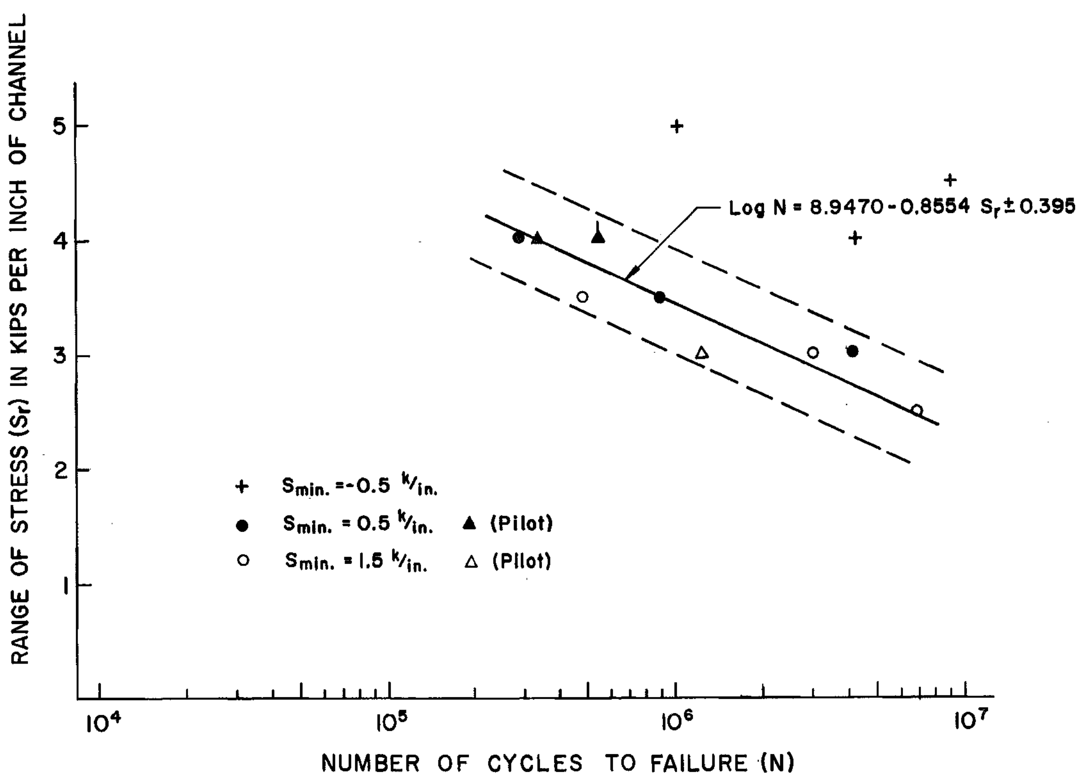

Fig. 6 S-N CURVE FOR 4 in.-5.4 lb. CHANNEL SHEAR CONNECTORS

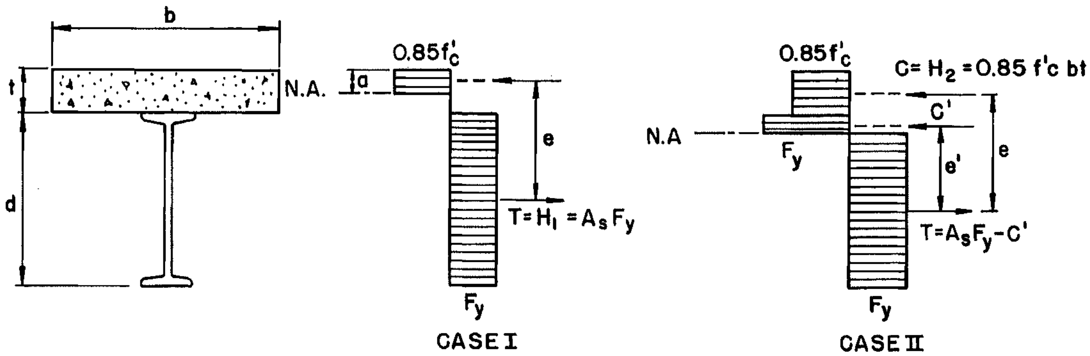

Fig. 7 STRESS DISTRIBUTION AT ULTIMATE LOAD

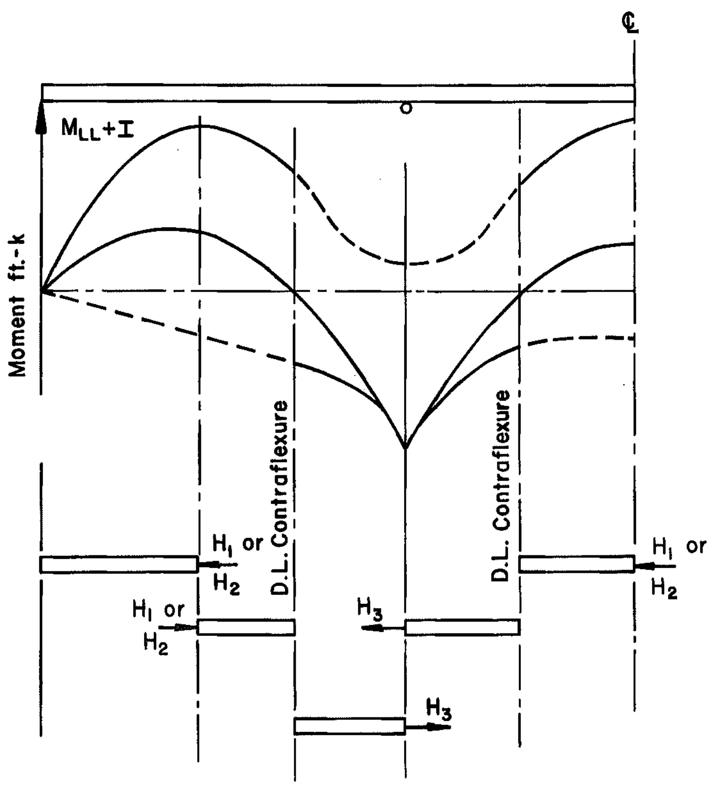

Fig. 8 FORCES ACTING ON THE SLAB OF CONTINUOUS BEAMS

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