Tentative design procedure for shear connectors in composite beams, August 1965

R. G. Slutter
J. W. Fisher

Follow this and additional works at: https://preserve.lehigh.edu/engr-civil-environmental-fritz-lab-reports

Recommended Citation
https://preserve.lehigh.edu/engr-civil-environmental-fritz-lab-reports/240

This Technical Report is brought to you for free and open access by the Civil and Environmental Engineering at Lehigh Preserve. It has been accepted for inclusion in Fritz Laboratory Reports by an authorized administrator of Lehigh Preserve. For more information, please contact preserve@lehigh.edu.
TENTATIVE DESIGN PROCEDURE FOR SHEAR CONNECTORS IN COMPOSITE BEAMS

by

R. G. Slutter
J. W. Fisher

Fritz Engineering Laboratory Report No. 316.1A
TENTATIVE DESIGN PROCEDURE FOR SHEAR CONNECTORS
IN COMPOSITE BEAMS

by

R. G. Slutter
J. W. Fisher

APRIL 1965
1. INTRODUCTION

Shear connectors for composite steel and concrete bridge members have been designed by a procedure in the AASHO Specifications\(^1\) based on the static strength properties of connectors.\(^2\) Recently, attention has been focused on the fatigue properties of shear connectors.\(^3, 4\) It has become apparent that fatigue is an important consideration when designing connectors for highway bridges. Since static criterion does not provide a solution of a fatigue problem, it is necessary to evaluate both the static and fatigue behavior when developing a design procedure.

Recent research has indicated that current design procedures may be too conservative and that additional economy may be possible by examining both the static and fatigue behavior of the shear connectors.\(^3, 4, 5\) By combining both criteria an optimum design procedure can be evolved in an optimum manner. The present design procedure is sufficiently conservative so that it excludes the possibility of fatigue failure of connectors. However, it is not possible to arbitrarily liberalize this method of design without encountering the risk that fatigue failure of connectors would occur under working loads. Preliminary fatigue tests have indicated that the endurance limit for stud shear connectors subjected to zero-to-tension loading is only about 25% of the static shear strength. There is no indication that a general correlation exists between the static shear strength and the fatigue strength.

The necessity to maintain complete interaction of a composite beam (which is reflected in the present method of design\(^2\)) may remain an
important consideration. The results of fatigue tests of beams at the University of Texas, Lehigh University, and the University of Illinois indicate that fatigue failure of connectors can be prevented by limiting the magnitude of slip. Or alternatively, the tests show that if fatigue failure of shear connectors is prevented, the loss of interaction due to slip is not of practical importance.

A second consideration is the necessity to provide a sufficient number of shear connectors such that the theoretical static ultimate strength of the bridge structure is assured.

The proposed design procedure outlined in this paper pertains only to the design of shear connectors; and the general provisions of the AASHO Specifications pertaining to the design of concrete and steel structures would still be applicable. That is, the cross-section of the member would be proportioned by the elastic method that is in current use. The proposed design procedure would result in a different quantity of shear connectors and a different arrangement of connectors as compared to present AASHO designs.

2. DESIGN CRITERIA FOR SHEAR CONNECTORS

The magnitude of the shear force transmitted by individual connectors has been found to agree closely with values predicted by theory within the elastic range. However, these experimental measurements have shown that individual connectors in a region of constant shear do not transmit equal forces. The connectors near the end of the beam are usually subjected to slightly higher stresses than connectors near midspan. However,
the stresses on end connectors seldom exceed the values predicted by the elastic formula. For beams of normal proportions, the difference between the measured and predicted shear stresses is only a nominal amount. Apparently this difference is caused by the lack of complete interaction.

Since fatigue is critical under repeated applications of working load, it is reasonable to determine the variation in shear stress using elastic theory. In other words the design criterion for fatigue is necessarily based on elastic considerations.

If complete interaction is assumed, the horizontal shear to be transferred by connectors for a given loading can be calculated as

\[ S = \frac{V_m}{I} \]  

where

- \( S \) = horizontal shear per inch of length
- \( V \) = shear in kips acting on the composite section
- \( m \) = statical moment of the transformed compressive concrete area about the neutral axis of the composite section, in.\(^2\)
- \( I \) = moment of inertia, in.\(^4\)

In negative moment regions of continuous beams the value of \( m \) will be the statical moment of the area of reinforcing steel.

An assessment of the fatigue behavior of various welded details has indicated that minimum stress may have a negligible effect on the fatigue strength. Although the fatigue testing of composite sections to date was generally for a zero-to-maximum loading, it is assumed herein that minimum stress has a negligible influence on the fatigue strength of shear connectors. In simple span beams the range of shear stress is nearly constant.
throughout the span. At the end of the beam the resulting shear stress 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 maximum shear envelopes in Fig. 1 the range of stress varies from zero-to-maximum at the support to full reversal at midspan. The shear envelopes for 50, 70, and 90 feet spans show clearly that these shear envelopes produce a shear stress range that is nearly uniform along the beam. At any section the range of shear is the difference in the minimum and maximum shear envelopes. Because of this near uniformity in most simple span members, for simplicity the range of stress could be calculated from Eq. 1 by considering only the maximum shear stress at the support. It should be noted that this would be somewhat conservative as the shear stress range may be slightly less than this value.

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 shear envelop for a typical continuous bridge structure. * If the variation in the shear stress range is significant, a variable spacing of the connectors will be necessary. The range of stress on the connectors can be calculated using the properties of the cross-section which are applicable to the positive and negative moments and the appropriate shear range.

When substantial dead load is carried by composite action, the variation in shear stress acting on the connectors may be relatively small so that fatigue strength is not critical. On the other hand, the maximum

* Taken from pg. 95 of Reference 7
shear stress due to dead load plus live load may be critical, so that the governing criteria may be the static ultimate strength. A limitation of the maximum shear stress acting on connectors is necessary for this situation as well as a change in the design criteria.

Maximum allowable shear stresses can be determined by limiting the loss of interaction or by applying a suitable factor of safety to the ultimate strength of the member. The useful capacity could be used if the loss of interaction was the limiting criterion. Or maximum allowable stresses could be obtained by dividing the ultimate strength by a factor of safety. Historically, the factor of safety for connections and connecting media has been larger than that used for the connected members. This assures that the connections and fasteners will not fail before the main members. If a load factor of 3.0 was selected to determine the working values for various types of shear connectors this historical criterion would be satisfied. Table 1 gives the allowable maximum loads per connector that were obtained from the demonstrated ultimate strength of connectors using the load factor of three.

Recent studies have demonstrated that the horizontal shear needed for determination of the number of shear connectors can be computed rationally based on the ultimate strength of the composite beam. The total horizontal shear \( V_h \) to be resisted by the connectors between the point of maximum moment and each end of the beam can be taken as the smaller value of

\[
V_h = \frac{1}{2} A_s f_y
\]  

(2a)

or

\[
V_{h2} = \frac{1}{2} 0.85 f'_c bt
\]

(2b)
where
\[
A_s = \text{total area of steel section including cover plate}
\]
\[
F_y = \text{minimum yield point of the type of steel being used}
\]
\[
f'_c = \text{compressive strength of concrete at 28 days}
\]
\[
b = \text{effective width of the concrete flange}
\]
\[
t = \text{thickness of the concrete slab}
\]

Using the allowable maximum load per connector given in Table 1, the total number of connectors required to develop the static ultimate strength can be determined. The tests reported in Ref. 5 have clearly 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 limited tests have also 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.

It should be noted that seldom if ever 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 strength.

In instances when the maximum load criterion is the more critical situation, excessive local permanent deformation of the concrete in the vicinity of stud connectors should be minimized when a load factor of 3.0 is used. The use of a smaller factor could cause local deterioration of the concrete which would adversely influence the fatigue strength. The other types of connectors have sufficient bearing area so that this problem is not present.
In simple span beams the stress range and maximum stress values will be nearly equal for unshored construction, and it would not ordinarily be necessary to calculate both values since the fatigue criterion is the most critical condition. This is also true in continuous beams for unshored construction. The maximum stress criterion will probably only enter into the design for shored construction.

3. TEST DATA AND TENTATIVE ALLOWABLE SHEAR STRESS FOR FATIGUE LOADING

An analysis of the test data for each type of connector can provide a relationship between shear stress and the number of cycles to fatigue failure. It can also provide limits of dispersion which indicate the scatter of the test data. An appropriate choice of the error term can be made to provide any desired probability of fatigue failure and thus provide the necessary factor of safety. A similar design approach has been suggested for other types of welded details. 8

The S-N curve obtained from available laboratory fatigue tests of connectors are relatively flat as can be seen in Fig. 3. In laboratory tests it has not been possible to find a stress level at which the S-N curve becomes horizontal. It is believed that this is in part due to the high bearing stress on the concrete that results in inelastic deformations which influence fatigue life.

Tentative allowable fatigue stresses for the various types of connectors can be obtained by considering the available test results on connectors and weld details. Nearly all available tests are for a zero-to-maximum fatigue loading. It has been assumed that minimum stress has
a negligible influence on the fatigue strength. This assumption will be checked when the current fatigue study is completed.9

Tentative allowable shear stresses for stud, channel and spiral connectors are shown in Figs. 4, 5, and 6 for two million cycles of loading. Stud connectors do not significantly differ and the same shear stress is proposed regardless of size.

In actual bridge structures the performance of connectors should improve with the passage of time as the concrete properties improve. This should provide an additional margin of safety against fatigue failure of the connectors. Two million cycles of loading was selected as the basis of design because of the flatness of the S-N curve. These values would seem to give an adequate factor of safety for all members.

4. DESIGN PROCEDURE

The first design consideration is based on the fatigue criterion. Necessarily, shear stresses are computed from Eq. 1. The spacing of the shear connectors is given by

$$P = \frac{F_r}{S_{\text{max.}} - S_{\text{min.}}}$$  \hspace{1cm} (3)

where

$$S_{\text{max.}} = \text{maximum horizontal shear per inch as calculated from Eq. 1}$$

$$S_{\text{min.}} = \text{minimum horizontal shear per inch as calculated from Eq. 1}$$

$$F_r = \text{allowable range of horizontal shear stress for the connector}$$

$$P = \text{spacing of shear connectors}$$
(Note: The quantity $S_{\text{max}} - S_{\text{min}}$ is the range of shear stress. As noted in Fig. 1 the range of shear at any location is $V_r$; the range of shear stress could be computed as $V_r m/I$. In continuous beams the range of shear stress is obtained by considering the sum of $V^+_r$ and $V^-_r$ as indicated in Fig. 2.)

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

The second step in the design procedure is to check to see that sufficient connectors are provided to ensure that the ultimate strength of the composite section can be achieved. First, determine the value of total horizontal shear. Take the smaller value obtained from Eqs. 2a and 2b. The number of shear connectors required between the point of maximum moment and the support or point of inflection in continuous beams is given by

$$N = \frac{V_h}{F_m}$$  \hspace{1cm} (4)

where

- $N = \text{number of connectors required between points of maximum and zero moments}$
- $F_m = \text{allowable shear connector load given in Table 1}$

If the number of connectors given by Eq. 4 exceeds the number provided by the spacing given by Eq. 3, additional connectors should be added to ensure that the ultimate strength is achieved.
Tentative values of $F_m$ and $F_r$ for stud, channel, and spiral connectors are given in Tables 1 and 2 respectively. These values will be revised later and are given here merely for the purpose of discussion of the proposed design procedure. The values of $F_r$ in Table 2 are based on Figs. 4, 5, and 6. The values in Table 1 are based on a concrete strength of 3000 psi and a load factor of 3.0. The use of higher concrete strengths for the calculation of these limiting values is difficult to justify.

The procedure for the design of shear connectors is outlined in Fig. 7 for the 85 foot span composite beam with unshored construction. In this example, Eq. 3 results in minimum connector spacing which is uniform throughout the span.

5. DISCUSSION

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.

In some instances the current procedure is unsafe; as an example, the present AASHO Design Procedure for 3/4 inch diameter stud shear connectors at midspan of a simple beam. The useful capacity of this connector in 3000 psi concrete is 10.2 kips per connector. If the factor of safety of 3.0 were used for this design, the allowable load per connector would be 3.4 kips. Because there is stress reversal on this connector, the actual stress range on the connector would be 6.8 kips under design loads.
This exceeds the tentatively recommended maximum allowable stress range of 5.7 kips for this connector.

It is readily apparent 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 factor of safety and be assured that fatigue is no problem. The use of a design concept which neglects fatigue could result in fatigue failure of shear connectors near midspan in beams which actually contain more shear connectors in the complete span than are actually required 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. Figure 7 shows a design of an 85 foot simple span bridge based on the tentative allowable shear values given for the various types of connectors in Tables 1 and 2. In this example the number of shear connectors would be reduced approximately 18% in the case of stud connectors as compared with the AASHO design.

Preliminary analysis has indicated that shear connectors proportioned by the proposed design criteria provide adequate resistance to applied load regardless of their position on the bridge structure.
### TABLE 1. MAXIMUM ALLOWABLE LOAD PER CONNECTOR $F_M$

<table>
<thead>
<tr>
<th>Type of Connector</th>
<th>Allowable Load (kips per connector)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Studs 1/2&quot;</td>
<td>4.2</td>
</tr>
<tr>
<td>5/8&quot;</td>
<td>6.7</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>9.6</td>
</tr>
<tr>
<td>7/8&quot;</td>
<td>13.0</td>
</tr>
<tr>
<td>Channels 3 [ 4.1</td>
<td>3.6</td>
</tr>
<tr>
<td>per 4 [ 5.4 inch</td>
<td>3.8</td>
</tr>
<tr>
<td>of 5 [ 6.7 length</td>
<td>4.1</td>
</tr>
<tr>
<td>Spirals 1/2&quot;</td>
<td>9.9</td>
</tr>
<tr>
<td>per 5/8&quot; turn</td>
<td>12.3</td>
</tr>
<tr>
<td>turn 3/4&quot;</td>
<td>14.8</td>
</tr>
</tbody>
</table>

### TABLE 2. VALUES OF ALLOWABLE STRESS RANGE $F_R$

<table>
<thead>
<tr>
<th>Type of Connector</th>
<th>Allowable Range (kips per connector)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Studs 1/2&quot;</td>
<td>2.5</td>
</tr>
<tr>
<td>5/8&quot;</td>
<td>3.9</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>5.7</td>
</tr>
<tr>
<td>7/8&quot;</td>
<td>7.7</td>
</tr>
<tr>
<td>Channels 3 [ 4.1</td>
<td>2.0</td>
</tr>
<tr>
<td>per 4 [ 5.4 inch</td>
<td>2.7</td>
</tr>
<tr>
<td>of 5 [ 6.7 length</td>
<td>3.0</td>
</tr>
<tr>
<td>Spirals 1/2&quot;</td>
<td>4.0</td>
</tr>
<tr>
<td>per 5/8&quot; turn</td>
<td>6.0</td>
</tr>
<tr>
<td>turn 3/4&quot;</td>
<td>9.0</td>
</tr>
</tbody>
</table>
Fig. 1 SHEAR ENVELOPES FOR SIMPLE SPAN BEAMS
Fig. 2 SHEAR ENVELOPES FOR A CONTINUOUS BEAM

See pp. 95 of Ref. 7
Fig. 3  S-N CURVES FOR BEAM TEST RESULTS
Fig. 4 ALLOWABLE SHEAR STRESS ON WELDED STUD SHEAR CONNECTORS
Fig. 5 ALLOWABLE SHEAR FORCE PER INCH WIDTH OF 4 5.4 lb. CONNECTORS
Fig. 6 ALLOWABLE SHEAR FORCE PER TURN OF 3/4 INCH DIAMETER SPIRALS
fc = 3 ksi  
Steel - ASTM A373  
Live Load - H20-S16  
Dead Load (Composite) = 343 lb./ft.  
m = 626 in.\(^3\)  
I = 29,110 in.\(^4\)

UNSHORED CONSTRUCTION

Shear Connectors to be pairs of 3/4" diameter Studs  
(AASHO Design requires 240 studs with F.S. = 3.0)

Number of Connections by Proposed Method = 196

Check Connectors Required for Ultimate Strength  
\[ V_h = \frac{1}{2} \cdot 0.85(3)78(7) = 696.2 \text{ kips} \]

Number of connectors required for half span =  
\[ \frac{696.2}{9.6} = 72.5 \text{ connectors} \]
REFERENCES

1. **STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES,**
   Eighth Edition
   by American Association of State Highway Officials,
   Washington, D. C., 1957

2. **DEVELOPMENT OF THE NEW AASHO SPECIFICATIONS FOR COMPOSITE STEEL AND CONCRETE BRIDGES,**
   by I. M. Viest, R. S. Fountain, and C. P. Siess,
   Washington, D. C., 1958

3. **FATIGUE TEST RESULTS OF COMPOSITE BEAMS**
   by A. A. Toprac
   Highway Research Board Paper,
   Washington, D. C., January 1965

4. **FATIGUE STRENGTH OF SHEAR CONNECTORS IN COMPOSITE BEAMS,**
   by R. G. Slutter
   Proc. 44th Annual Meeting, Highway Research Board,
   National Academy of Science, Washington, D. C., 1965

5. **FLEXURAL STRENGTH OF STEEL-CONCRETE COMPOSITE BEAMS**
   by R. C. Slutter and G. C. Driscoll, Jr.
   Structural Journal ASCE, Vol. 91, No. ST2,
   April 1965

6. **STUDIES OF SLAB AND BEAM HIGHWAY BRIDGES, PART III,**
   **SMALL SCALE TESTS OF SHEAR CONNECTORS AND COMPOSITE T-BEAMS,**
   by C. P. Siess, I. M. Viest, and M. M. Newmark
   University of Illinois Engineering Experiment Station
   Bulletin No. 396
   Urbana, Illinois, 1952

7. **COMPOSITE CONSTRUCTION IN STEEL AND CONCRETE,**
   by I. M. Viest, R. S. Fountain, and R. C. Singleton,

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

9. **PROPOSAL FOR INVESTIGATION OF SHEAR CONNECTOR DESIGN FOR HIGHWAY BRIDGES,**
   by R. G. Slutter and J. W. Fisher
   Lehigh University, October 1964