Composite Design for Bridges

FATIGUE TESTS OF COMPOSITE BEAMS

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1. INTRODUCTION

The fatigue strength of mechanical shear connections in a composite beam has been studied from the point of view of the adequacy of present specifications, but little has been done from the point of view of determining the minimum number of connectors required. Because of this, an investigation has been initiated at Lehigh University to systematically study the problem of minimum shear connector requirements and in addition to determine whether it is possible to formulate design specifications which would be more liberal than the present AASHO Bridge Specifications.\(^5\) If it is found that a liberalizing of the specifications is possible, bridge construction (which presents a major fatigue consideration) could become more economical than at present.

Although some work has been done on the fatigue properties of certain types of mechanical shear connectors, little has been done on the most common type of shear connector, namely, welded studs. Tests have been conducted in which bare studs were stressed under conditions of completely reversed repeated loading by a constant deflection type machine.\(^1\) Sufficient results were obtained to firmly establish the S-N curve for this type of loading and connector.
In addition to the above tests on bare studs, two composite members with stud shear connectors were tested at Lehigh University. The first member consisted of 11'-0" wide bridge slab with two 18WF50 steel sections. The second member was similar to the test specimens reported herein. In neither case did a fatigue loading result in stud failure.

Some pushout specimens with stud connectors have also been tested under fatigue loading at Lehigh University. Stud failure in fatigue actually occurred in the testing of these specimens.

It is important to note, however, that the results of the above mentioned tests do not provide adequate knowledge for a comprehensive design specification. It is because of this lack of knowledge that the AASHO Specifications are especially conservative. However, adequate testing may well allow the AASHO Specifications to be liberalized.

Before beginning a full scale series of fatigue investigations, it was decided that some preliminary tests should be run. These preliminary tests had several purposes. First, it was necessary to study the performance of a member under fatigue loading which did not have sufficient shear connectors for complete interaction. Second, it was necessary to produce fatigue failure of connectors in a beam and determine if the
fatigue strength of connectors obtained from pushout tests could be used as a basis for design. Lastly, it was felt that a more comprehensive instrumentation and testing procedure could be developed for future tests.

2. DESCRIPTION OF SPECIMENS

The beams for the preliminary tests consisted of a 2'-0" by 3" concrete slab connected to an 8WF17 steel beam as shown in Figure 1. The shear connection consisted of \( \frac{1}{2} \)" diameter "L" shaped welded stud connectors. One member of this type (Beam B-4) was tested in fatigue in 1959 at Lehigh University. The stress in the shear connectors for Beam B-4 was approximately 24 ksi in the final fatigue loading. This stress did not result in shear connector failure prior to fatigue failure of the steel beam. The specimens used in this preliminary investigation were designed similar to B-4 so that results could be compared.

Since the failure of the connectors was paramount in this preliminary investigation, it was found necessary to increase the stud stress above that used on the previous beam test (B-4). Member B-4 was designed with groups of three \( \frac{1}{2} \)" diameter connectors spaced 5 \( \frac{1}{2} \)" apart. The four preliminary beams had \( \frac{1}{2} \)" diameter connectors spaced as follows:
<table>
<thead>
<tr>
<th>Member</th>
<th>Connector Type</th>
<th>Connector Arrangement</th>
<th>Spacing in Inches</th>
</tr>
</thead>
<tbody>
<tr>
<td>BF-A</td>
<td>&quot;L&quot;</td>
<td>Single</td>
<td>7.5</td>
</tr>
<tr>
<td>BF-B</td>
<td>&quot;L&quot;</td>
<td>Double</td>
<td>15.0</td>
</tr>
<tr>
<td>BF-C</td>
<td>&quot;L&quot;</td>
<td>Single</td>
<td>5.5</td>
</tr>
<tr>
<td>BF-D</td>
<td>&quot;L&quot;</td>
<td>Double</td>
<td>11.0</td>
</tr>
</tbody>
</table>

All four test members in this series were fabricated at the Fritz Engineering Laboratory. The concrete slabs were all reinforced with 6" x 6"-#4/#4 welded wire fabric. Concrete for the slabs was taken from a single batch of ready-mix concrete. Concrete strengths were as follows:

<table>
<thead>
<tr>
<th>Age of Concrete</th>
<th>Average Concrete Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>28 days</td>
<td>3,030 psi</td>
</tr>
<tr>
<td>time of testing</td>
<td>3,500 psi (BF-C; BF-D)</td>
</tr>
</tbody>
</table>

Testing was started at 28 days and was completed at 79 days.

3. INSTRUMENTATION

The instrumentation consisted of strain gages at the mid-span under the top and bottom flanges of the steel section, a midspan deflection gage, and slip measuring devices at both ends and near the quarter points. The strain gages were used to check load alignment, to aid in the determination of the degree of interaction, and to serve as an indication of connector failure. The location of these gages is as shown in Figure 2.
The deflection gage was used to determine the need for any load adjustment due to inertial forces, to aid in the determination of interaction, and also to serve as an indication of connector failure. The locations of the deflection gage and four slip gages are shown in Figure 3. The slip gages consisted of 0.001" Ames Dials for the dynamic readings and 0.0001" Ames Dials for the static readings. These gages were used to measure any movement of the slab relative to the steel beam. Whitewash was also liberally applied to the specimens to make easier the identification of any cracks or local yielding.

4. **TEST PROCEDURE**

The specimens were placed on the loading frame and testing was begun after the specimens had been wet cured for two weeks and air cured for a minimum of two weeks. Each specimen was initially loaded to a static value sufficient to break bond between beam and slab. The testing arrangement is shown in Figure 4.

While being loaded statically all SR-4, deflection, and slip readings were taken at intervals of 1.5 kips per jack. After this initial static test each specimen was loaded at 250 cycles per minute. Static tests were taken every 100,000 cycles until failure occurred. Periodically dynamic end slip
and deflection readings were taken. At the end of each cycling period and when failure occurred a static test was run similar to the original one and to a load at least equal to the maximum which was applied during cycling. This procedure was rigidly adhered to except in the case of specimen BF-A where dynamic readings and static tests were run more often. Not all specimens were run at the same load, but the load was never changed in the middle of a test. Throughout the tests care was taken to note all cracking or unusual phenomenon.

5. RESULTS

All of the specimens failed by fatigue of the end shear connectors and all failed in the same manner. For that reason they shall be discussed as a group. All members except BF-D exhibited a transverse crack through the concrete slab in the shear span near the load points at the time of failure.

As each member was cycled in turn, the first indication of failure was an audible banging of the slab upon the steel section. A later analysis of the strain, slip, and deflection readings indicated the actual time of failure. A plot of slip measured at the interior gages versus the number of load cycles is shown for member BF-D in Figure 5. The number of cycles at which the banging commenced corresponds to the point when the studs apparently failed as determined from analysis of data.
After this apparent failure of a connector or connectors, cycling was continued, but no further failures were apparent. In no case was the failure of a sudden type, and in all cases cycling was continued beyond this first failure. In one case (BF-D), cycling was continued for another million cycles before it was decided to drop the attempt to fail more connectors.

A plot of the position of the neutral axis (see Figure 6) in the composite section versus the number of cycles for given static loads indicated that concrete creep plus partial action of the interior connectors (those connectors between the load points) was the cause of this inability to fail additional connectors. Connectors between load points actually carried part of the load attributed to the connectors in the shear span after initial connector failure. The tensile strength of the concrete slab transmitted the load carried by the interior connectors until the concrete tensile strength was exceeded.

The transverse crack in the concrete slab mentioned previously formed when the tensile strength of the concrete was exceeded. Thereafter the member consisted of a steel beam with three individual little concrete slabs connected to it. These slabs were very ineffective as cover plates and added little to the strength of the steel member.
The decrease in effectiveness of the concrete slab, causing a downward movement of the neutral axis and hence a reduction of the loads on the connectors, prevented additional connector failures. It is thought at this time that the normal amount of reinforcing steel usually put in bridge slabs is sufficient to reduce the creep effect of the slab as a whole to a negligible amount, but other tests are required to be certain.

During the static testing it was noted that for a portion of the loading cycle the fatigued member exhibited a partial composite action, while for the higher part of the loading cycle the composite member exhibited a more complete interaction. This may well be the result of crushing of the concrete around the bases of the connectors. At first a connector is only partially effective, but once it has firmly contacted the concrete it becomes more effective. This effect can be seen in the load deflection curves of BF-B, BF-C, and particularly BF-D (see Figures 7, 8, and 9).

It should be pointed out, that as the fatigue tests progressed this tendency towards loss of composite action decreased. A plot of the load at which the slope of the load deflection curve changes versus number of cycles of load for Beam BF-D is given in Figure 10. This indicates that the rate of decrease in effectiveness of the concrete slab does not continue with number of cycles.
The results of this series of tests have been tabulated along with the results of other pertinent tests and are presented in Table 1. The failure stresses were determined from strain readings immediately prior to failure.

Once the tests were completed it was possible to draw an S-N curve using the failure stresses determined from the test. The resulting S-N curve can be seen in Figures 11 and 12. The resulting curve for beams was slightly above the lower bound curve established from pushout specimens. The curve was also well below the upper bound curve established from the bending of bare studs. This would certainly indicate that fatigue results from pushout specimens constitute a safe lower bound.

Because it was decided during testing that it would be more valuable to test the specimens at different loads, no comparison can be made with respect to connector spacing or arrangement. However, indications are that connectors in pairs are more efficient. More tests must be conducted to establish this relationship.

Specimen BF-D was tested near the AASHO useful connector capacity. The AASHO Bridge Specifications indicate that a satisfactory factor of safety to be applied to this useful connector capacity is four. However, the fact that specimen

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# See 1.9.5, Reference 5.
BF-D, tested at the useful connector capacity, was cycled for 820,000 cycles before first failure occurred and another 1,200,000 without additional failures, would indicate that this factor of safety may well be overly conservative.

As was mentioned earlier, the end connectors consistently failed first. Additional tests are required to determine the cause of this, but the addition of connectors at the ends of the specimen may well increase the fatigue life of the composite member.

6. CONCLUSIONS

1. The amount of slip does not appear to be an important fatigue consideration.

2. Connector failure does not result in total failure of the composite structural member, but merely an increase in the deflection and steel section stresses.

3. S-N Curves established from the fatigue of stud shear connectors in pushout specimens serve as a good lower bound to the actual connector fatigue strength in composite members.

4. The AASHO Bridge Specifications are overly conservative with respect to fatigue, and a factor of safety of two (2) against fatigue failure of stud shear connectors in composite members may be sufficient.
ACKNOWLEDGEMENTS

This work has been carried out through the Institute of Research of Lehigh University under the sponsorship of the American Institute of Steel Construction. Technical guidance for the project was supplied by the AISC Committee on Composite Design (Mr. T. R. Higgins, Chairman).

Appreciation is expressed to the technical staff of Fritz Engineering Laboratory for their work in fabrication, instrumentation, and testing of members and to Mrs. L. Morrow for typing the manuscript.
APPENDIX
NOMENCLATURE

$A_s = \text{steel area}$

$s_{st} = \text{distance from neutral axis of composite section to extreme fiber of steel in tension}$

$b_c = \text{width of concrete slab}$

$d_c = \text{depth of concrete slab}$

$d_s = \text{depth of steel section}$

$f'_c = \text{cylinder strength of concrete at time of testing}$

$f_y = \text{yield stress of steel beam}$

$I = \text{moment of inertia of composite section, concrete transformed to equivalent steel area}$

$I_s = \text{moment of inertia of steel beam}$

$P = \text{load per jack}$
SECTION PROPERTIES

1. Concrete Slab
   \( b_c = 24 \text{ in.} \)
   \( d_c = 3 \text{ in.} \)
   \( f_c = 3500 \text{ psi} \)

2. Steel Beam (8WF17)
   \( A_s = 5.00 \text{ in.}^2 \)
   \( d_s = 8.00 \text{ in.} \)
   \( I_s = 56.4 \text{ in.}^4 \)
   \( f_y = 33.0 \text{ ksi}^* \)

3. Studs (L-connector)
   \( \text{diameter} = \frac{1}{2} \text{ in.} \)
   \( \text{height} = 2.25 \text{ in.} \)
   \( \text{area} = 0.196 \text{ in.}^2 \)

4. Composite Section
   \( a_{st} = 7.48 \text{ in.} \)
   \( I = 156.0 \text{ in.}^4 \)

* Specified minimum of ASTM A-7 Steel
Figure 1. Dimensions of Test Specimens

- Slab
- Mesh: 6" x 6" +4/4
- Cross Section
- Stud Connector

Dimensions of Test Specimens:
- BF-A: S = 7.5"
- BF-B: S = 15.0"
- BF-C: S = 5.5"
- BF-D: S = 11.0"
4 SR-4 Type A-I gages on each of 4 composite beam specimens.

SR-4 gages bottom of top flange and bottom of bottom flange at midspan.

Figure 2. Strain Gage Location
Figure 3: Slip and Deflection Gage Location
Figure 4. Test Setup
Figure 5. Interior Static Slip vs. Cycles. BF-D
Figure 6. Movement of Neutral Axis Under Static Load as Fatigue Progresses. BF-D
Figure 7. Load-Deflection Curve - BF-B
Figure 8. Load-Deflection Curve - BF-C
Figure 9. Load-Deflection Curve - BF-D
Figure 10. Decrease in Rate of Loss of Composite Action as Cycling Progresses
Figure 11. S-N Curve for Stud Shear Connectors
Figure 12. S-N Curve for Stud Shear Connectors
<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Reference Type</th>
<th>Stud Type</th>
<th>Load Range Maximum Stress* (psi)</th>
<th>Minimum Stress* (psi)</th>
<th>Cycles to Failure</th>
<th>Specimen Type</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>4</td>
<td>L-Connector</td>
<td>22,300</td>
<td>2900</td>
<td>223,200</td>
<td>pushout</td>
<td>Stud fracture</td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>&quot;</td>
<td>17,800</td>
<td>2200</td>
<td>134,200</td>
<td>pushout</td>
<td>&quot;</td>
</tr>
<tr>
<td>6</td>
<td>4</td>
<td>&quot;</td>
<td>17,800</td>
<td>2200</td>
<td>261,000</td>
<td>pushout</td>
<td>&quot;</td>
</tr>
<tr>
<td>7</td>
<td>4</td>
<td>&quot;</td>
<td>15,600</td>
<td>1900</td>
<td>1,748,000</td>
<td>pushout</td>
<td>&quot;</td>
</tr>
<tr>
<td>B-4</td>
<td>3</td>
<td>&quot;</td>
<td>21.0</td>
<td>1.5</td>
<td>619,900</td>
<td>beam</td>
<td>no stud fracture</td>
</tr>
<tr>
<td>B-4</td>
<td>3</td>
<td>&quot;</td>
<td>24.0</td>
<td>1.5</td>
<td>122,400</td>
<td>beam</td>
<td>no stud fracture</td>
</tr>
<tr>
<td>BF-A</td>
<td>&quot;</td>
<td>13.50</td>
<td>23,900**</td>
<td>5730</td>
<td>50,300</td>
<td>beam</td>
<td>Stud fracture</td>
</tr>
<tr>
<td>BF-B</td>
<td>&quot;</td>
<td>11.30</td>
<td>32,600</td>
<td>4900</td>
<td>55,400</td>
<td>beam</td>
<td>&quot;</td>
</tr>
<tr>
<td>BF-C</td>
<td>&quot;</td>
<td>11.65</td>
<td>27,700</td>
<td>3600</td>
<td>78,000</td>
<td>beam</td>
<td>&quot;</td>
</tr>
<tr>
<td>BF-D</td>
<td>&quot;</td>
<td>7.00</td>
<td>20,100</td>
<td>2160</td>
<td>820,000</td>
<td>beam</td>
<td>&quot;</td>
</tr>
</tbody>
</table>

* Load divided by cross-sectional area of all studs.
** Stress determined from strain readings after failure occurred. This point is not considered a valid data point and has been omitted from Figures 11 and 12.
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