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Yao-Ching Wu

R. G. Slutter

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CONTINUOUS COMPOSITE BEAMS UNDER FATIGUE LOADING

by
Yao-Ching Wu
R. G. Slutter

This research was conducted by
Fritz Engineering Laboratory
Lehigh University
for
Pennsylvania Department of Transportation
in Cooperation with
Federal Highway Administration

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Pennsylvania Department of Transportation or the Federal Highway Administration.

Fritz Engineering Laboratory
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<table>
<thead>
<tr>
<th>TABLE OF CONTENTS</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABSTRACT</td>
<td>1</td>
</tr>
<tr>
<td>1. INTRODUCTION</td>
<td>2</td>
</tr>
<tr>
<td>2. BEHAVIOR OF CONTINUOUS BEAM</td>
<td>5</td>
</tr>
<tr>
<td>2.1 Positive Moment Region</td>
<td>5</td>
</tr>
<tr>
<td>2.2 Negative Moment Region</td>
<td>6</td>
</tr>
<tr>
<td>2.3 Previous Studies on Fatigue Strength</td>
<td>7</td>
</tr>
<tr>
<td>3. DESCRIPTION OF TEST SPECIMENS</td>
<td>11</td>
</tr>
<tr>
<td>3.1 Design Criteria</td>
<td>11</td>
</tr>
<tr>
<td>3.2 Design Details and Fabrication</td>
<td>12</td>
</tr>
<tr>
<td>4. PROPERTIES OF TEST BEAMS</td>
<td>14</td>
</tr>
<tr>
<td>4.1 Rolled Steel Beams</td>
<td>14</td>
</tr>
<tr>
<td>4.2 Reinforcing Bars</td>
<td>14</td>
</tr>
<tr>
<td>4.3 Stud Shear Connectors</td>
<td>15</td>
</tr>
<tr>
<td>4.4 Cross-section Properties</td>
<td>15</td>
</tr>
<tr>
<td>5. INSTRUMENTATION</td>
<td>17</td>
</tr>
<tr>
<td>6. TEST PROCEDURE</td>
<td>19</td>
</tr>
<tr>
<td>7. TEST RESULTS</td>
<td>22</td>
</tr>
<tr>
<td>7.1 Deformation of the Continuous Beams</td>
<td>22</td>
</tr>
<tr>
<td>7.2 Strain Measurements in the Steel Beam in the</td>
<td>23</td>
</tr>
<tr>
<td>Negative Moment Region</td>
<td></td>
</tr>
<tr>
<td>7.3 Strain Measurements in Reinforcement over</td>
<td>24</td>
</tr>
<tr>
<td>Negative Moment Region</td>
<td></td>
</tr>
<tr>
<td>7.4 Strain Measurements under Stud Connectors</td>
<td>25</td>
</tr>
<tr>
<td>7.5 Slip Measurements</td>
<td>26</td>
</tr>
<tr>
<td>7.6 Cracking of Slabs in the Negative Moment Region</td>
<td>26</td>
</tr>
</tbody>
</table>
# TABLE OF CONTENTS (continued)

8. ANALYSIS OF TEST RESULTS

8.1 Stresses and Bending Moments in Continuous Composite Beams 28
8.2 Slab Force and Slip in the Negative Moment Region 30
8.3 Force on Stud Connectors in the Negative Moment Region 32
8.4 Behavior of a Cracked Slab Under Tension Forces 33

9. SUMMARY AND CONCLUSIONS

ACKNOWLEDGMENTS 41
TABLES AND FIGURES 42
REFERENCES 70
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Details of Composite Beams of CC-3F and CC-4F</td>
<td>46</td>
</tr>
<tr>
<td>2</td>
<td>Typical Beam Cross Section for Beams</td>
<td>47</td>
</tr>
<tr>
<td>3</td>
<td>Details of the Steel Beams for CC-3F and CC-4F</td>
<td>48</td>
</tr>
<tr>
<td>4</td>
<td>Strain Gage Locations on W21X62 for Beams CC-3F and CC-4F</td>
<td>49</td>
</tr>
<tr>
<td>5</td>
<td>Locations of Deflection, Rotation, and Slip Gages for Beam CC-4F</td>
<td>50</td>
</tr>
<tr>
<td>6</td>
<td>Beams CC-3F and CC-4F Test Setup</td>
<td>51</td>
</tr>
<tr>
<td>7</td>
<td>Fracture of Reinforcement at the West Span Near the Center Support of Beam CC-3F</td>
<td>52</td>
</tr>
<tr>
<td>8</td>
<td>Sound Connectors at the West Side Near the Dead Load Point of Contraflexure in the Negative Moment Region of Beam CC-3F</td>
<td>52</td>
</tr>
<tr>
<td>9</td>
<td>Fracture of Studs at the East Span Near the Dead Load Point of Contraflexure in the Negative Moment Region of Beam CC-4F</td>
<td>53</td>
</tr>
<tr>
<td>10</td>
<td>Load-Deflection Curve for CC-3F (West Span)</td>
<td>54</td>
</tr>
<tr>
<td>11</td>
<td>Typical Strain Distribution in Negative Moment Regions for Beams</td>
<td>55</td>
</tr>
<tr>
<td>12</td>
<td>Location of Neutral Axis at Section 4 in Negative Moment Regions (Typical)</td>
<td>56</td>
</tr>
<tr>
<td>13</td>
<td>Average Load-Strain Curves of Reinforcing Bars at Section 3 of Beam CC-3F</td>
<td>57</td>
</tr>
<tr>
<td>14</td>
<td>Average Load-Strain Curves of Reinforcing Bars at Section 3 of Beam CC-4F</td>
<td>58</td>
</tr>
<tr>
<td>15</td>
<td>Strain Under Studs in Negative Moment Region (West Span of CC-4F)</td>
<td>59</td>
</tr>
<tr>
<td>16</td>
<td>Strain Readings versus Cycles - Beam CC-4F</td>
<td>60</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>--------</td>
<td>-----------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>17</td>
<td>Slip Distribution on Beams CC-3F and CC-4F</td>
<td>61</td>
</tr>
<tr>
<td>18</td>
<td>Distribution of Slab Cracking in Negative Moment Regions of Beams</td>
<td>62</td>
</tr>
<tr>
<td>19</td>
<td>Moment Distribution for Beam CC-3F</td>
<td>63</td>
</tr>
<tr>
<td>20</td>
<td>Force in Longitudinal Reinforcement of Beam CC-3F</td>
<td>64</td>
</tr>
<tr>
<td>21</td>
<td>Force in Longitudinal Reinforcement of Beam CC-4F</td>
<td>65</td>
</tr>
<tr>
<td>22</td>
<td>Slab Force Curve for Beam CC-4F</td>
<td>66</td>
</tr>
<tr>
<td>23</td>
<td>Slab Force Curve for Beam CC-3F</td>
<td>67</td>
</tr>
<tr>
<td>24</td>
<td>Slab Force Curve for Beam CC-2F</td>
<td>68</td>
</tr>
<tr>
<td>25</td>
<td>Slab Force Curve for Beam CC-1F</td>
<td>69</td>
</tr>
</tbody>
</table>
ABSTRACT

The results of fatigue and static tests of two continuous composite beams are reported. These results and earlier experimental results are compared with computer studies using a mathematical model to simulate the continuous composite member which has a cracked slab in the negative moment region. Satisfactory correlation was obtained between predicted and experimental values for slab force, loads on shear connectors, and strain distribution in the cross section.

The study indicated that the mathematical model can be used to evaluate the effect of important parameters that influence the behavior of the negative moment region. These parameters have been identified as area of longitudinal reinforcing steel, number of longitudinal bars, and spacing of shear connectors. It has been shown that the stiffness of the cracked slab in the negative moment region is greater than the area of the longitudinal reinforcing steel.

The study indicates that a beam with one percent longitudinal slab reinforcement in the negative moment region performed well with shear connectors omitted from a large portion of the negative moment region. The stiffness of the slab in tension was observed to equal the area of the longitudinal reinforcement plus approximately 20 percent of the concrete area.
Composite construction of steel and concrete composite beams has been used with increasing frequency in both buildings and bridges. However, the design of continuous composite members has been hampered by the lack of a thorough study of the behavior of such members. Currently, the AASHO Specifications,\(^1\) the AISC Specifications,\(^2\) and the British Codes of Practice\(^3\) cover fairly general methods of composite design for simple span members that consider both strength and performance. The provisions for the design of the negative moment region of continuous beams are not as fully developed as provisions for the positive moment region.

Design criteria\(^4\) for shear connectors in bridge structures require that adequate static and fatigue strength be provided. The 1969 AASHO Specifications are based on this criteria. Sufficient connectors are needed to avoid fatigue failure and to insure that the static ultimate strength of the composite beam can be achieved. In general the design of a bridge structure is governed by fatigue requirements and the member must be very carefully proportioned. The allowable range of stresses must not be exceeded in the steel section, the longitudinal reinforcement of the slab, or the shear connectors.

For many years continuous beams were designed to be composite in the positive moment region and non-composite in the negative moment
region. From recent experimental studies on continuous composite beams \(^{(5,6)}\) it was concluded that shear connectors are required to resist the slab tensile force developed by the curvature of the continuous longitudinal reinforcement in the negative moment region even though the slab was not connected to the steel beam.

When shear connectors are omitted in the negative moment region, the shear connectors in the positive moment regions near the inflection points must resist the additional force developed in the longitudinal reinforcement extending from the negative moment region. This additional force may cause premature fatigue failure of the shear connectors located near the inflection points. Enough shear connectors must be provided for anchorage of the continuous longitudinal reinforcement whether or not the negative moment region is considered to be composite or not. The 1969 AASHO Specification has provisions that consider this fact. The latitude allowed to the designer in this regard may result in shear connectors that will be overstressed in fatigue.

The participation of the concrete slab over the negative moment region has been examined in the past by two theories. One considers the composite beam to have complete interaction between the steel beam and reinforcement. The second method assumes the longitudinal reinforcement to act as though anchored near the points of contraflexure. Neither of these theories adequately accounts for the behavior of composite beams in the negative moment region and consequently the stresses in the longitudinal reinforcement and shear connectors can not be accurately determined.
Available tests on continuous beams and other experience have indicated that fatigue can be critical in: (a) the tension flange of the steel section at points where shear connectors are welded; (b) the top layer of longitudinal reinforcement over the interior support; (c) the shear connectors between the interior support and the dead load point of contraflexure.

Premature fatigue failure is prevented by a reduction in the allowable stress permitted in the tension flange when shear connectors are welded to it.\(^{(1)}\) Unfortunately the shifting of shear connectors from the region of maximum negative moment toward the dead load point of contraflexure may result in a greater probability of other types of failure.

This study is intended to evaluate the performance of continuous composite beams which have stud shear connectors proportioned by the criteria suggested in Refs. 4 and 5 which were used to develop the 1969 AASHO Specification provisions. Both the fatigue and static behavior were considered and evaluated to ascertain the applicability of the suggested design concepts to continuous composite beams with stud shear connectors. This research program made extensive use of the earlier work on connector and beam behavior.\(^{(4,5)}\) Only the results of fatigue studies on continuous composite beams are discussed herein.
2. BEHAVIOR OF CONTINUOUS BEAM

2.1 Positive Moment Region

For a simply supported composite beam, static tests have shown that the interaction of the steel beam and the concrete slab is never entirely complete. Although slip results in incomplete interaction, the decrease of interaction has little effect on stresses and deflections. The experimental load-strain and load-deflection curves are nearly identical to values computed using a transformed section, and complete interaction. Fatigue tests of composite beams have shown that an adequate fatigue strength was provided for simple beams by the design criteria suggested in Ref. 4.

In the fatigue tests of two-span continuous composite beams, bond failure in the end portions of positive moment regions was observed to initiate at the end of the member and progress toward the loading points. In the positive moment regions adjacent to the dead load points of contraflexure the bond failure also progresses from the points of the contraflexure toward the load points. It was observed that the critical shear connectors were near the ends or points of contraflexure of the positive moment regions.

The tensile strength of concrete is only equal to between one-eighth and one-twelfth of the concrete compressive strength. Hence, relatively small tensile stresses cause cracking in the concrete slab.
Tensile stresses in the concrete slab of continuous composite beams can result from shrinkage or loading. If shrinkage cracking occurs in the positive moment regions the cracks normally close on application of load.

Shrinkage can produce a sizeable dimensional change in the slab which is diminished by creep over a period of time. In composite beams, the concrete deck slab is interconnected with the steel beam by the mechanical shear connectors. Deformation of the concrete slab induces deformation into the steel beam producing stresses in the shear connectors, concrete slab, and steel beam. In the positive moment regions the concrete slab may be subjected to permanent compressive stresses if dead loads act on the composite section. If creep occurs it increases the compressive stresses in the top flanges and the tensile stresses in the bottom flanges of the steel beams and decreases the compressive stresses in the slab. Creep also decreases the loads acting on shear connectors. Thus, the critical loading of shear connectors may occur immediately after construction before substantial creep can take place if shored construction is used or substantial dead loads are carried by the composite section.

2.2 Negative Moment Region

Besides shrinkage, loading can also cause cracks to form in the negative moment region of a continuous composite beam. Hence, the concrete slab in the negative moment region has been considered ineffective in resisting any stresses. On application of load, tensile
cracks tends to open wider in the negative moment region. This is different from the situation in the positive moment region where application of load tends to close the cracks.

The presence or absence of shear connectors in the negative moment region of a continuous composite beam has little effect on the distribution of strains in the positive moment region. (7) For beams with or without shear connectors in the negative moment regions, the change in the bending moment distribution and the loss of interaction between the concrete slab and the beam due to fatigue loading was not significant for the positive moment region. (5)

Continuous composite beams without shear connectors in the negative moment region still develop a considerable tensile force in the longitudinal reinforcement throughout the negative moment region due to the curvature induced in this region. (5) When shear connectors were placed in the negative moment region, the slab reinforcement was observed to be fully effective. It was concluded that the forces developed in the slab over interior supports with or without shear connectors in the negative moment region was of the same order of magnitude, but the interaction between the slab and steel beam was quite different.

2.3 Previous Studies on Fatigue Strength

Fatigue tests were available on four two-span continuous beams. Two were designed for 2,000,000 cycles of load application and two were designed for static strength conditions and then loaded for
500,000 cycles. Only beams CC-1F and CC-2F which were designed for 2,000,000 cycles of load application are of interest and will be briefly described in this section. 

Beams CC-1F and CC-2F had two equal spans of 25 feet and consisted of a 60 inch wide by 6 inch reinforced concrete slab. The slab was connected to a W21X62 ASTM A36 steel beam with 3/4 inch diameter by 4 inch stud shear connectors. Two pulsating concentrated loads located 10 feet from each end of each span were applied symmetrically with respect to the interior support by two Amsler jacks. The load varied from near zero to a maximum load which was approximately the working load for each beam. The loading rate was constant for the two beams at 250 cycles per minute. Both beams were designed according to the 1965 AASHO Bridge Design Specifications except for the shear connectors which were proportioned in accordance with the procedure recommended in Ref. 4. Beam CC-1F had no shear connectors in the negative moment region. Beam CC-2F had shear connectors in the negative moment region and could be directly compared to beam CC-1F.

Beam CC-1F had 1,907,000 cycles of load applied and 2,079,000 cycles of load were applied to beam CC-2F. Observations made during the testing of beam CC-1F showed that studs were failing in fatigue at about 500,000 cycles in the east span and that a continuous deterioration of the shear connectors was occurring. The deterioration of the beam had progressed to the point where the fatigue test was stopped short of the desired number of cycles so that a static test of the beam to ultimate load could be carried out.
Beam CC-2F had connectors placed in the negative moment region except for a length of 22 inches on either side of the interior support. It was concluded that the presence or absence of shear connectors in the negative moment regions of beam CC-2F and beam CC-1F had little effect on the distribution of strains in the positive moment regions. It was also readily apparent that the steel beam of CC-1F was interacting with the longitudinal reinforcement in the negative moment region. Since a considerable tensile force was developed in the longitudinal steel due to the curvature in the negative moment region, little shear transfer could take place between the slab and the beam of CC-1F after the bond had been broken because no transfer devices were present. In addition, frictional forces and flexural conformance could not exist in the negative moment region because of slab separation.

It was shown that a higher degree of flexural conformity was present in beam CC-2F, but there was much less than assumed on the basis of complete interaction. Although the static load behavior of beams CC-1F and CC-2F was similar, the dynamic response of beam CC-1F was inferior compared to that of beam CC-2F. It was difficult to maintain the maximum dynamic load during the test of beam CC-1F due to the uneven deflection characteristics of each span after 1,000,000 cycles of load. The longitudinal reinforcement in the negative moment region of beam CC-1F acted like tendons in an unbonded post-tensioned beam. Therefore, the studs near the points of contraflexure were subjected to shear forces considerably higher than those in beam CC-2F. This caused premature failure of the studs in this region of beam CC-1F.
In beam CC-1F the initial slab force was about equal to the theoretical value for a cracked section with complete interaction over the interior support. This rapidly decreased to a level slightly below the value determined for a cracked slab without interaction of the longitudinal reinforcement and the steel beam. The slab force was nearly uniform over the entire negative moment region and only varied because of the frictional forces. The slab force in beam CC-2F maintained a level near the interior support which was greater than observed in beam CC-1F. Because of the presence of shear connectors in the negative moment region of beam CC-2F, a higher initial slab force was observed. The slab force decreased in beam CC-2F in both spans at about the same rate. The force stabilized near the theoretical value for a cracked slab with complete interaction after about 200,000 cycles. At the inflection point of beam CC-2F the slab force approached zero as flexural conformance required. It was apparent that shear transfer was taking place in beam CC-2F between the slab and the steel beam in the negative moment region as a result of the presence of the shear connectors in that region.
3. DESCRIPTION OF TEST SPECIMENS

Beams CC-3F and CC-4F were similar to CC-1F and CC-2F. Each of the members was 50 ft.-10 in. long with two equal spans of 25 ft.-0 in. between bearings. Symmetrical concentrated loads were applied 10 ft.-0 in. from the exterior support in each span. The beam consisted of a reinforced concrete slab 60 inches wide and 6 inches thick interconnected to W21X62 steel beam with 3/4 x 4 inch stud shear connectors. The rolled beams were all supplied from the same heat of A36 steel. Details of the continuous composite beams are shown in Figs. 1 and 2.

3.1 Design Criteria

The design of the composite beams was based on the transformed area method. The concrete slab was transformed to an equivalent area of steel. A modular ratio of steel to concrete of 10 was assumed in the design. Table 1 summarizes the design stresses for the steel beams. A considerable space between shear connectors at the interior support of beams CC-3F and CC-4F was made so that the tensile stress in the top flange adjacent to stud shear connectors did not exceed the limit given in the AASHO Specification for 2,000,000 cycles.

The transverse reinforcing steel in the slab was designed so that the percentage of steel was the same as typical bridge decks designed for an H20-S16 truck. In beam CC-3F the longitudinal reinforcement was made continuous through the entire beam length. The
amount of longitudinal steel used for beam CC-3F was 66.7% of the required transverse reinforcement and was equal to 0.61% of the cross-sectional area of the concrete slab. In the negative moment region of beam CC-4F the amount of longitudinal reinforcing steel was increased to 1.025% of the cross-sectional area of the concrete slab. The placement of the reinforcing steel is shown in Fig. 2. The #4 bars were lapped and welded to provide continuous reinforcement throughout the length of the beams.

3.2 Design Details and Fabrication

Each of the two beams was cut from a 57 ft. length of rolled section by a local fabricating shop. The excess pieces were marked and delivered to the laboratory to provide material for tension tests of the steel section and studs. Bearing plates were fitted and welded to the beams by the fabricating shop. Web stiffeners were installed to prevent premature failure of the beams during the static ultimate load tests.

All 3/4 in. studs were placed in pairs except for the studs in the negative moment region as illustrated in Fig. 3. Single studs were staggered throughout the negative moment region with extra studs near the point of contraflexure in beam CC-4F. Before the studs were welded to the test beams, the stud welding equipment was calibrated by welding several studs to the extra pieces of W21X62 steel beam. The quality of the welds was verified using the welding and inspection procedure outlined in Ref. 8. The transverse reinforcement in the
beams was provided by two layers of #5 bars placed at 6 in. centers throughout the beam.

To assist in moving these large test specimens in the laboratory, 2 in. pipe sleeves were cast into the slabs approximately 13 ft. on each side of the interior support. They extended the full depth of the slab and were located 10 in. on each side of the centerline of the beam as shown in Fig. 1. The lifting points were selected so that the concrete tensile stresses were minimized during handling and erection.

Construction of the continuous composite T-beams began with the erection of a structural steel beam on supports at Fritz Engineering Laboratory. Plywood forms for the slab of the T-beam were suspended from the steel beams. The concrete for the slabs was transit-mixed and proportioned for a 28 day compressive strength of 3000 psi.

The concrete in the slabs of the two beams was moist-cured for seven days with the exposed surface covered with burlap and a plastic sheet. The forms were removed approximately 14 days after casting and the specimens were allowed to cure under normal laboratory ambient conditions thereafter.
4. PROPERTIES OF TEST BEAMS

A detailed test program was conducted to determine the physical characteristics of the materials used in the T-beams. Also, the physical dimensions were obtained to help ascertain the section properties of the composite beams.

4.1 Rolled Steel Beams

The mechanical properties of the structural steel were determined from tests of tensile coupons cut from a 2 ft. piece of the beam that had been flame cut from the original 57 ft. length. The coupons were tested in tension at a speed of 0.025 in. per minute up to the onset of strain hardening and then at a speed of 0.05 in. per minute to fracture. In all tests, the yield point, static yield level, and ultimate load were recorded. The values of the yield point, static yield stress, and the ultimate strength are listed in Table 2.

4.2 Reinforcing Bars

The mechanical properties of the #4 deformed longitudinal reinforcing bars were determined by tension tests of 3 ft. lengths of reinforcement. The deformed bars were of intermediate grade conforming to ASTM A615 steel. The average yield point and tensile strength are given in Table 2.
4.3 Stud Shear Connectors

The stud shear connectors were made from a single coil of material. The properties of the studs were determined from tensile tests on full sized studs. These studs had been welded to a short length of W21X62 steel after proper welds were obtained as noted earlier. A 3-1/2 in. x 3-1/2 in. square of the beam flange containing a stud was cut out and the unit tested in a special test jig. The average ultimate strength is also included in Table 2.

4.4 Cross-section Properties

The cross-section properties of W21X62 rolled steel beams are given in Table 3. The properties are given for both measured dimensions and handbook values.

The section properties of the composite sections in the positive moment regions were computed on the following assumptions:

1. The effective width of the slab was taken as the full 5 ft. width.
2. The ratio of the modulus of elasticity of the steel beam to the concrete was assumed to be $n = 10$.

The section properties of the composite sections in the negative moment regions were computed based on the steel beam and the area of the longitudinal reinforcing steel. It was assumed that the concrete slab was cracked throughout its depth.

Table 4 gives the moments of inertia and the distance to the neutral axis from the bottom of the beam for each test specimen.
The values given in this table are based on measured dimensions of the member.

The concrete properties are summarized in Table 5. The splitting tensile strength and compressive strength are given at 28 days and the compressive strength at the time of testing was also obtained.
5. INSTRUMENTATION

The instrumentation for beams CC-3F and CC-4F was essentially the same as that used previously in testing CC-1F and CC-2F. Figures 4 and 5 summarize the instrumentation used on CC-3F and CC-4F. A combination of electrical resistance strain gages, dial gages, and level bars was used.

Figure 4 shows the location of the electrical resistance strain gages which were used to determine the flexural strains in the steel beams. These strain gages were placed on the underside of both the top and bottom flanges of the steel beam as well as on both sides of the web. They were located at 7 sections for beam CC-3F and at 10 sections for beam CC-4F as shown in Fig. 4.

In addition electrical resistance strain gages were attached to the top surface of all the #4 bars longitudinal and the #5 transverse bars at sections 1, 2 and 3. These gages were waterproofed and wrapped before the concrete was placed. The wrapping broke the bond over a length of about 4 inches. The lead wires were passed through the bottom of the wood form during construction.

Electrical resistance strain gages were also placed on the underside of the top flanges near some of the stud connectors. These strain gages were used to help detect connector failure.

Figure 5 shows the locations of dial gages that were used to measure slip and deflection. Dial gages were placed under the beams at the
load points to measure vertical deflection. These gage readings were
used to adjust the maximum dynamic load level at the beginning and
throughout the duration of each beam test. Dial gages (0.001 in.) were
also used to measure slip at each end of the two continuous beams and
at various sections along the beam spans.

A large compression dynamometer was used at the interior support
to measure the center reaction as indicated in Fig. 6. This reaction
was used together with the known loads to determine bending moments
along the beam. A check was thus provided on the bending moments
computed from the strain measurements on the steel beam. A 50 power
microscope was used to measure the width of cracks in the slab in the
negative moment region. In addition, the crack patterns were photographed.
6. TEST PROCEDURE

Each two-span continuous beam was supported at three points resulting in two 25 ft. spans. Load was applied to each span by hydraulic jacks located 10 ft. from each exterior support. The load was distributed across the width of the concrete slab by a 4 ft. long loading beam. The test setup is shown schematically in Fig. 6.

The interior support of the beams consisted of a compression dynamometer. To insure stability at this point, a fixed assembly of beams and plates was mounted on either side of the dynamometer as shown in Fig. 6. Grooved plates were fitted to the web stiffener to maintain longitudinal stability and the bottom flange of the W21X62 beam was braced to provide lateral stability. A small clearance was provided at all points so that the reaction could be measured by the compression dynamometer. Lead shims from 1/4 in. to 3/8 in. thick were placed between the interior bearing plates and the beam to prevent a high stress concentration resulting from flexure over the bearing plate. During the fatigue tests, the lead shims deformed to a round surface thus providing even bearing of the beams.

The exterior supports of all continuous beams were provided by bearing plates mounted on high rocker plates. End braces were provided over each rocker support to provide lateral stability and to prevent excessive transverse vibration of the beams during the fatigue tests.
Additional shims were used to bring the three supports to the same elevation.

The fatigue testing of beams CC-3F and CC-4F was started 137 and 154 days respectively after the concrete slabs were cast. Initially, each beam was loaded statically to the maximum load to be applied during the fatigue testing. The load was applied in increments of 10 kips up to a maximum of 60 kips at each load point. All instrumentation was read at each load increment. The cyclic load applied during the fatigue tests was governed by the deflection obtained under the maximum static load. The deflections at the two load points were nearly the same for each beam. Frequent checks were made of the deflections at the initial stages of the test and adjustments to the dynamic load were made as necessary. Since the bending stiffness of each span was changing because of bond failure and other causes, the deflections gradually changed but stabilized early in the test. Frequent static tests from zero to maximum load were run so that adjustments in the dynamic load could be made.

The maximum dynamic load was always less than the maximum static load because of the dynamic amplification effect. For the two beams the dynamic load correction was usually from 1.5 to 3.9 kips at each load point. The minimum dynamic load varied from 4.5 to 7.0 kips for the beams. The minimum load was the smallest that could be applied without separation of the beam and loading jack during cyclic loading. Hence the actual loading cycle was nearly zero to maximum throughout all the tests based on static load levels. The applied dynamic
loading placed all connectors in the two beams at the design shear level suggested in the AASHO Specification for 2,000,000 cycles of load.

Due to shrinkage of the concrete slab prior to testing, each beam had an initial deflection over the 50 ft. length so that the center support was initially about 1/2 inch lower than the end supports in order to achieve the proper live load reaction at the three supports. During the first 100,000 cycles the slab developed a cracking pattern in the negative moment region which made it possible to raise the center support level with the end supports. The static test data taken immediately after the raising of the center support was somewhat erratic. However, the application of additional cycles of live load restored the behavior of the beam to what it had been prior to elevation of the center support.

After completion of fatigue testing the two continuous beams were loaded to determine the static ultimate strength. After the static strength tests were completed the concrete slab of CC-3F near the interior support was removed to check the fracture of reinforcing bars near the interior support. The concrete slab of CC-4F in the negative moment region was removed and each connector in this region was bent with a hammer to ascertain whether or not fatigue cracks were present. Several connectors were found to be fractured. Photographs made of the reinforcement and connector fractures in the negative moment region are shown in Figs. 7, 8 and 9.
7. TEST RESULTS

During fatigue testing 2,012,200 cycles of working load were applied to CC-3F and 2,027,300 cycles of the same level to CC-4F. Beam CC-3F had a slab crack near the center support of the West span before the fatigue loading was started, caused by shrinkage of the concrete slab. It was observed that reinforcement at the crack was failing during the test of beam CC-3F in fatigue. A continuous deterioration of the reinforcement occurred after 1,100,000 cycles of applied load. Figure 7 shows fracture of the reinforcement at the vicinity of the center support of the West span.

After the ultimate strength test of beam CC-4F, concrete in the negative moment region was removed. It was found that four connectors in the West span and six connectors in the East span were partly fractured during the fatigue loading of 2,027,000 cycles as shown in Fig. 9.

7.1 Deformation of the Continuous Beams

Deflection measurements under the loading points were taken at each static test which was conducted at intervals throughout the period of the fatigue loading. Figure 10 shows the load deflection characteristics of the West span of beam CC-3F. The curves are plotted for the start of fatigue testing, for two intermediate cyclic levels, and at the end of the test. For comparative purposes the theoretical
curves based on the cases of complete interaction and no interaction of the steel beam and concrete slab are also shown.

It is apparent that the deflections at the loading points of beam CC-3F increased as the cycles of applied load increased. This is partly due to the fracture of reinforcement during the fatigue test. The deflections at the loading points of beam CC-4F were little affected by the cycles of applied load.

The use of lead and steel shims at the center and exterior supports of the two beams was discussed previously. It was obvious that after completion of the test a small amount of support settlement was observed. The support settlements were assumed to stabilize after the initial zero to maximum static loading. The load-deflection results were corrected for this settlement.

7.2 Strain Measurements in the Steel Beam in the Negative Moment Region

Electrical resistance strain gages were applied at three sections on beam CC-3F and at six sections on beam CC-4F. Typical strain distributions are shown in Fig. 11. Strain distributions near the center support of beams CC-3F and CC-4F for a static load of 60 kips are compared with the results of beam CC-2F. The effect of the increased area of longitudinal reinforcement in beam CC-4F can be observed by comparing the strain distribution for beam CC-3F and CC-4F. It is also apparent that the strain distributions for beams CC-2F and CC-4F are essentially identical.
Figure 12 shows the location of the neutral axis as a function of the loading cycles for a typical section in the negative moment region. The position of the neutral axis was not sensitive to the progress of the fatigue test or slight changes in the reinforcement area.

7.3 Strain Measurements in Reinforcement over Negative Moment Region

Electrical resistance strain gages were applied on the reinforcement at three sections over the negative moment region for both beams CC-3F and CC-4F. Strain gage readings were taken at intervals during the fatigue test. Figures 13 and 14 summarized the load strain characteristics in reinforcement over the negative moment region for both beams. Figure 14 shows that strain in the top layer of beam CC-4F was little affected by the cycles of applied load. The strain was much smaller than observed in beam CC-3F (See Fig. 13), which had a lesser amount of longitudinal reinforcement in the negative moment region.

Five of the six reinforcing bars in the top layer of beam CC-3F fractured during the fatigue testing. After some reinforcing bars on the top layer were broken, the reinforcing bars at the bottom layer were subjected to more force as shown in Fig. 13. The average strains in the top and bottom layers of beam CC-4F were much smaller than the strains observed in beam CC-3F. The measurements of strains in the reinforcing bars of various sections in the negative moment regions
359.2

of beams CC-3F and CC-4F indicated that the full width of concrete slab was effective.

7.4 Strain Measurements under Stud Connectors

Electrical resistance strain gages were installed on the bottom of the upper beam flange in the immediate vicinity of stud connectors. The strains obtained at various static load levels were plotted as a function of the applied cyclic load.

Figure 15 shows strain versus applied cyclic load at the maximum load level for all stud connectors in the negative moment region of the West span of beam CC-4F. The two studs near the center support show evidence of being cracked at 2 million cycles. The strains decreased for studs nearer the dead load points of contraflexure. The figure also shows that strains increased as the fatigue test progressed, indicating that greater load was being carried by stud connectors as bond was broken and friction reduced. Four connectors were partly fractured in the West span and six connectors in the East span at the end of the test. The crack initiated on the side of the stud closer to the center support.

Strain readings obtained from gages located under one row of connectors near the point of contraflexure are plotted in Fig. 16 for intervals throughout the fatigue test. The curve for gage 23 indicates the formation of a fatigue crack prior to 2,000,000 cycles. It is interesting to note that the other connectors indicate an increase in load as the connector opposite gage 23 developed a crack.
7.5 Slip Measurements

Slip and uplift measurements were taken at intervals during the fatigue test throughout the length of the continuous beams. Figure 17 shows the range of slip and the direction of the range of slip between 0 and 60 kips at various cycles of applied load along the beam for both beams CC-3F and CC-4F.

In the negative moment region the range of slip was much greater than that at the exterior supports. The range of slip increased throughout the fatigue test. It was apparent that after bond had been broken the range of slip at the exterior supports changed very little. The range of slip in the negative moment region of beam CC-3F was much greater than that of beam CC-4F. The range of slip for both beams at dead load points of contraflexure was not zero. This was in agreement with the observed level of strain under stud connectors at the same locations.

At the center support the range of slip was almost zero for both beams, and the maximum range of slip occurred at the midway point between the center support and the first stud nearest the center support in the negative moment region.

7.6 Cracking of Slabs in the Negative Moment Region

Beam CC-3F had a transverse crack due to shrinkage near the center support in the West span before the initial static loading was applied. Beam CC-4F was relatively free of cracks at the beginning of the test. Some cracks formed during the initial static loading.
A few additional cracks appeared during the fatigue testing. The distribution of the slab cracks is shown in Fig. 18 for all four continuous beams. Crack widths were measured at a static load of 60 kips following the cyclic loading.

During the initial static loading of beam CC-3F to 60 kips only one more crack occurred near the interior support. Three more cracks were observed elsewhere. The cracks were approximately perpendicular to the beam axis and were observed to increase in width as fatigue testing progressed.

More longitudinal reinforcing bars were placed in the negative moment region of beam CC-4F than that of beam CC-3F and the crack pattern of beam CC-4F was substantially different from that of beam CC-3F. The slab of CC-4F was relatively free of cracks at the beginning of the test but cracked during the initial static loading of the beam. Nine cracks were formed in the negative moment region between the points of contraflexure during the initial static loading of the beam as indicated in Fig. 18. The cracks were approximately uniformly distributed over the negative moment region indicating approximately uniform tensile force in the reinforcing bars over the negative moment region. The cracks were approximately perpendicular to the beam axis. A few additional cracks appeared during the fatigue test.
8. ANALYSIS OF TEST RESULTS

8.1 Stresses and Bending Moments in Continuous Composite Beams

Historically the theoretical analysis of a simple composite beam has been based on the well-known transformed area method with complete interaction between the concrete slab and the steel beam. Later the elastic analysis was improved by taking account of the effect of slip between the concrete slab and the steel beam.\(^9\) Recently, with the aid of high speed computers, numerical analysis was developed to take account of the non-linear relationship of connector load and slip.\(^10\) The behavior of a simple composite beam can be evaluated satisfactorily if the stress versus strain relationship for both slab and beam materials along with the connector load versus slip relationship are known. However, the behavior of a continuous composite beam is more difficult to determine than that of a simple composite beam. Very little information about the behavior of continuous composite beams is available. A theoretical study of the behavior of continuous composite beams will be discussed in detail in a later report. Only the results related to the test beams will be briefly reported here.

The dead load stresses for CC-3F and CC-4F were calculated by assuming a unit weight of materials and using measured section properties and dimensions of the member. Table 6 lists the computed dead load
stresses at the top and bottom of the steel beam under loading points and at the center support.

The strain distribution throughout the depth of the steel beam was evaluated by data from the electrical resistance strain gages. Figure 11 summarized the strain distribution for typical cross-sections in the negative moment regions of both beams CC-3F and CC-4F. The computed strains based on the transformed area method are also shown for comparison.

Figure 12 indicated that no significant shift in the neutral axis occurred during the fatigue test. In both CC-3F and CC-4F the steel beam was interacting with the longitudinal reinforcement in the negative moment region. This was expected since a considerable tensile force was induced into the longitudinal reinforcement due to the curvature in the negative moment region.

The shear connectors for negative moment of both beams were located near the dead load point of contraflexure at both ends of the negative moment region, as shown in Fig. 3. This arrangement prevented the tensile stress in the beam flange adjacent to a stud shear connector from exceeding the specification value.\(^{(1)}\) The reinforcement acted as an unbonded tie bar over the negative moment region. Since no shear transfer can take place between the slab and the beam after bond has been broken except a very small amount due to frictional forces, flexural conformance can not exist in the negative moment region for both beams. Figure 17 shows that there was a large amount of slip in the negative moment region of both beams. Beam CC-4F, which had more
longitudinal reinforcement over the negative moment region, provided
greater flexural conformity than beam CC-3F, but much less than the
assumption of complete interaction would require.

Figure 18 compares the theoretical and experimental bending
moment distribution in beam CC-3F. Stress resultants were computed
for both the flexure and tie bar modes. Complete interaction and
points of dead load contraflexure were assumed. Comparable results
were obtained for beam CC-4F.

In general, the stiffness of the negative moment region was
decreased with increased cycles due to cracking of the concrete slab
and loss of bond between the slab and the beam. Measurements of the
center reaction by a load cell and the strain readings throughout the
depth of the steel beam at various sections of the continuous beam
indicated a change in bending moment distribution between the begin­
ing and end of the fatigue test. This was also apparent from the
measured strains and reactions. The bending moment at the center
support decreased whereas the bending moment at the loading points
increased throughout the test.

8.2 Slab Force and Slip in the Negative Moment Region

The degree of interaction for composite beams can be estimated
from the slip between the concrete slab and the steel beam. Flexural
conformance can be maintained completely only if a very large number
of shear connectors are provided to reduce the slip to zero. Some
degree of incomplete interaction and lack of flexural conformance
exists in all composite beams.
The degree of interaction and flexural conformance are of particular interest in the negative moment regions of continuous composite beams with a finite number of shear connectors. To evaluate the degree of interaction in both beams CC-3F and CC-4F, the forces in the slab at various sections over the negative moment region were calculated from the strain measurements on the reinforcement and steel beam. Theoretical analysis considering the tie bar mode, the flexural mode, and incomplete interaction were made and compared with the test results.

Figures 20 and 21 summarize the slab force developed in the negative moment region of both beams as the fatigue test progressed. The slab force was maximum near the center support and decreased toward the dead load point of contraflexure. Theoretically the maximum slab force should occur at the center support. Although no shear connectors were provided from the center support to a point about 5 feet from the center support, the slab force decreased an appreciable amount in this region for beam CC-4F. This may be due to loss of interaction and friction. The slab force at the dead load point of contraflexure was almost zero for CC-3F but not for CC-4F as illustrated in Fig. 21. The shear connectors in the negative moment region apparently were not sufficient to remove all of the force in the slab of CC-4F.

The maximum range of slip occurred at a point midway between the center support and the first stud illustrated in Fig. 17. This is due to the fact that the force developed in the slab was nearly constant from the center support to the first connector, whereas, the
external moment gradient was very steep, being maximum at the center support and nearly zero at the dead load point of contraflexure. In order to satisfy the compatibility condition, curvature must be smaller at the mid-point than at the center support.

A theoretical analysis was developed to evaluate the behavior of the composite beams over the negative moment region. From a study of the change in slab force and the corresponding slip data in the region of the dead load point of contraflexure a general picture of the variation in the force per connector could be obtained. A suitable load-slip curve for the shear connectors in the negative and positive moment regions of the beams was obtained from a study of the load-slip data from the four continuous composite beams and other experimental programs.

8.3 Force on Stud Connectors in the Negative Moment Region

The curves of Fig. 15 reveal that the force on most of the shear connectors in the negative moment region increases slightly as the number of cycles of loading increases. This is due to progressive changes in the stiffness of the member from cracking of the slab and the gradual change in the shear transfer by bond and friction. The curves reflect the relative magnitude of the force on the shear connectors through the region. There is a variation in the force along the length of the beam, and also some variation transversely due to the staggered pattern of the connectors.
The average force per shear connector for the negative moment region of beams CC-3F and CC-4F are given in Table 7 for intervals during the test. The design value for the connector force is also given. The test value was obtained as the change in the tensile force in the slab divided by the number of connectors between two sections. The tensile force developed in the slab at each section was determined from the strain gages on the longitudinal reinforcing steel. The force obtained in this manner was checked by computing the slab force from the strain gages on the steel section. Very good correlation was obtained between the values of slab force computed by these two methods.

Although the average connector force is considerably less than the design value the curves of Fig. 15 indicate that the maximum force on a connector is slightly greater than the design value. The force tends to become more evenly distributed as the number of cycles increases. At 1,000,000 cycles the maximum force per connector is approximately 1.75 times the average force. In beam CC-4F the maximum connector force was approximately 5.6 kips per connector on the first cycle and approximately 4.6 kips per connector at 1,000,000 cycles.

8.4 Behavior of a Cracked Slab Under Tension Forces

The slabs of beams CC-1F and CC-2F were cast in two pours with the negative moment regions being cast a week earlier than the remainder of the slab. In these members the effect of shrinkage of the concrete was insignificant. The slabs of beams CC-3F and CC-4F were cast in a single pour and the start of testing was delayed considerably longer
than had been the case for the other two members. The effect of shrinkage of the concrete was quite significant for beams CC-3F and CC-4F. In Fig. 18 the crack pattern of CC-3F is drastically different from that of the earlier members even though the amount of slab reinforcement is the same. It is obvious that beam CC-4F had a sufficient amount of reinforcing steel in the negative moment region to develop a satisfactory crack pattern.

The cracks in CC-3F were wide at the beginning of the test and remained unchanged throughout the test. There were no new cracks formed during fatigue loading whereas fatigue loading produced new hairline cracks in the other three members. The upper layer of reinforcing steel failed in fatigue eventually at one of the cracks near the center support. The shear connectors in CC-3F did not develop cracks due to failure of the reinforcement.

The other three beams exhibited shear connector failure. In the case of CC-1F the failure of connectors was obvious and was easily detected during testing. In the case of CC-2F and CC-4F the slabs had to be removed to find the fatigue cracks in connectors. Particularly in the case of CC-4F there was no indication of connector failure from the test data.

In designing the shear connectors for the four members, the number of connectors was selected by considering only the area of the reinforcing steel. The magnitude of the slab force observed in these members was consistently greater than that determined by considering only the reinforcing steel. Therefore, the number of shear connectors
must be increased if fatigue failure of connectors is to be prevented. A procedure for finding the magnitude of the force in the cracked slab does not exist in the literature. Such a procedure had to be developed as a part of this investigation.

The bending stiffness of the cracked slab has little effect on the distribution of forces in the member but the stiffness of the slab in tension is important because the slab must conform to the curvature of the steel beam. The tensile stiffness of the slab can be determined by a mathematical model which considers the slab as consisting of only reinforcing steel at each crack and a composite slab between cracks. With this model the effective stiffness of a cracked slab can be considered and the slab force determined.

8.5 Comparison of Beam Performance

It is apparent from the previous discussion that none of the four beams tested had a sufficient number of connectors in the negative moment region. From the point of view of design, this problem could be solved very easily by requiring that the number of shear connectors provided be computed on the basis of the area of longitudinal steel plus some percentage of the area of the concrete slab. It was found in the testing of beam CC-4F that even a percentage figure of zero was nearly satisfactory so long as the amount of longitudinal steel was increased to 1 percent.

One of the goals of this project was to determine an optimum design for a continuous composite beam. Beam CC-4F was very nearly the
optimum beam. The improved performance of CC-4F as compared to CC-3F is not only due to the increase in the amount of longitudinal steel but also due to an increase in the effectiveness of the concrete slab. It is rather impractical to test other members with smaller or larger percentages of steel because a difference in performance may not be measurable.

From the knowledge gained in testing these four members, the effect of changes in other variables can be studied using the mathematical model by computer. Among the variables actually studied by this manner were changes in longitudinal reinforcement, variation in shrinkage coefficient and various spacings of shear connectors. It was found that changing the spacing of shear connectors also changes the effectiveness of the concrete slab. This provides the reason for the relatively good performance of beam CC-2F even though the amount of longitudinal steel in the slab was the same as for CC-1F and CC-3F.

One guideline which provides an obvious lower bound for the percentage of longitudinal reinforcement in any region of a continuous beam where the slab will be in tension under live load is that the maximum tension force that can be developed in the concrete should not be sufficient to yield the steel when the slab cracks. This is necessary to control the size of cracks and to retain the effectiveness of the concrete after cracking at a level corresponding to composite behavior of the concrete and reinforcing steel. If the steel yields, only a tie bar exists at the crack and the danger of a disastrous result such as was experienced in beam CC-3F develops.
Figure 22 shows the force in the concrete slab in the negative moment region of CC-4F which was obtained using the mathematical model with two different load-slip curves and two values for the effective stiffness of the concrete slab. Cracks in the concrete slab were assumed at intervals of 6 inches. The lower curve was obtained by considering only reinforcing steel while the upper two curves were obtained by taking the effective stiffness of the concrete slab between cracks to be 20 percent of the stiffness of an uncracked slab. A load-slip curve which represented initial loading of a member was used to generate the upper curve, and a load-slip curve which represented the load-slip behavior after numerous cycles of loading was used to generate the middle curve. The slope at the right shows the change in slab force at each shear connector except the last increment which represents three connectors at the same cross section. The pattern of shear connectors (see Fig. 3) is nine single connectors at a spacing of 1-11/16 inches followed by three connectors at the next location which is 1-11/16 inches from the ninth connector.

The experimental results obtained from strain gages on the reinforcing steel and steel beam are plotted in Fig. 22 for 0 and 2,000,000 cycles of loading. The experimental results for the initial cycle show the effect of at least partial bond between the slab and beam which was not treated by the mathematical model. For cycles of loading after bond failure, the values from the mathematical model and experimental results correlated very well.

Because of deterioration of the members during fatigue loading, it was difficult to compare the results of the mathematical model with
test results for the other beams. However, the results for the initial load cycle can be compared to establish correlation. Figure 23 shows two curves for beam CC-3F which were obtained using slab stiffness values of 0 and 12 percent with a load-slip curve for initial loading. The slab force obtained from strain measurements on the member was close to that predicted by the upper curve.

For beam CC-2F the corresponding curves are somewhat different because the shear connectors were spread out over most of the negative moment region. The results for static loading are given in Fig. 24 for slab stiffness values of 0 and 12 percent. The upper curve predicts the slab force fairly well, but it underestimates the slab force for the initial cycle because bond has not been considered in the mathematical model.

Beam CC-1F offered the greatest challenge for simulation by the mathematical model because it had no shear connectors in the negative moment region. Figure 25 shows the results for static loading with slab stiffness values of 0 and 12 percent. The upper curve correlates very well with the slab force obtained from strain measurements near midspan. The curve predicts very high shear connector forces on the first shear connector, and this was also found in the test.

One of the most interesting observations is that in Fig. 24 where the curve predicts high shear connector loads on shear connectors located at the dead load point of contraflexure. These connectors were the ones that failed in the fatigue test. At the time beam CC-2F was tested it came as a surprise that these rather than the end shear connectors failed.
9. SUMMARY AND CONCLUSIONS

The results of fatigue testing two continuous beams, CC-3F and CC-4F, have been presented. The results of earlier tests on two similar beams, CC-1F and CC-2F have also been reviewed. The testing program for all four members consisted of 2,000,000 cycles of working load followed by an ultimate load test.

In all of the beams except CC-4F the amount of longitudinal reinforcing steel in the slab over the negative moment region was essentially the amount required by the 1969 AASHO Specifications. None of these beams performed satisfactorily although beam CC-2F was better than the others. Beam CC-4F exhibited satisfactory behavior due to the increased participation of the concrete slab in the negative moment region which was brought about by increasing the percentage of longitudinal steel to 1 percent. The analysis of test results has revealed that all beams should have had additional shear connectors in the negative moment region. However, the additional shear connectors would not have improved the behavior of beams CC-1F, CC-2F, or CC-3F to an appreciable extent. For CC-4F the additional connectors would have prevented fatigue cracking of connectors.

The experience gained in these tests has shown that the amount of longitudinal reinforcing steel should be greater than the minimum required by the 1969 AASHO Specifications. Beam CC-4F provides an apparently satisfactory design, but a complete study of the other
variables involved must be made before design criteria are finalized.
The following conclusions were reached as a result of this study.

1. The number of shear connectors in the negative moment region should be computed on the basis of the stiffness of the cracked concrete slab. The stiffness of the slab is dependent on the number and size of the longitudinal reinforcing bars and the spacing of the connectors.

2. Shrinkage of the concrete in the slab is an important factor in determining the minimum percentage of longitudinal reinforcing steel required in the negative moment region. The magnitude of the shrinkage force is dependent upon construction procedure, span length, and material properties.

3. In beam CC-4F the effective stiffness of the cracked concrete slab in tension was approximately 20 percent of the uncracked slab. The corresponding value for the other three beams was approximately 12 percent.

4. Beam CC-4F was provided with 1 percent of longitudinal slab reinforcement and provided satisfactory performance.
ACKNOWLEDGMENTS

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**TABLE 1**

**SUMMARY OF DESIGN STRESSES**

<table>
<thead>
<tr>
<th>Test Beam No.</th>
<th>Live Load</th>
<th>Dead Load</th>
<th>Loads</th>
<th>Flexural Stress (DL + LL)</th>
<th>At Load Points</th>
<th>At Interior Support</th>
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</thead>
<tbody>
<tr>
<td>CC-3F*</td>
<td>60</td>
<td>0.437</td>
<td>20.93</td>
<td>-3.12</td>
<td>-0.70</td>
<td>-21.76</td>
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<tr>
<td>CC-4F*</td>
<td>60</td>
<td>0.437</td>
<td>20.37</td>
<td>-2.98</td>
<td>-0.67</td>
<td>-21.23</td>
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<tr>
<td>CC-3F**</td>
<td>60</td>
<td>0.437</td>
<td>20.06</td>
<td>-3.10</td>
<td>-0.69</td>
<td>-20.84</td>
</tr>
<tr>
<td>CC-4F**</td>
<td>60</td>
<td>0.437</td>
<td>19.72</td>
<td>-2.94</td>
<td>-0.66</td>
<td>-20.52</td>
</tr>
</tbody>
</table>

* W21X62 Values by measurement

**TABLE 2**

**MATERIAL PROPERTIES OF STEEL**

<table>
<thead>
<tr>
<th>Type of Specimen</th>
<th>No. of Tests</th>
<th>Yield Point (ksi)</th>
<th>Static Yield Point (ksi)</th>
<th>Tensile Strength (ksi)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean</td>
<td>Std.Dev.</td>
<td>Mean</td>
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<tr>
<td>CC-3F Flange</td>
<td>3</td>
<td>33.57</td>
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<td>31.77</td>
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<tr>
<td>CC-4F Flange</td>
<td>3</td>
<td>35.42</td>
<td>0.37</td>
<td>33.72</td>
</tr>
<tr>
<td>CC-3F Web</td>
<td>3</td>
<td>34.42</td>
<td>1.93</td>
<td>32.94</td>
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<tr>
<td>CC-4F Web</td>
<td>3</td>
<td>36.87</td>
<td>2.08</td>
<td>35.26</td>
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<tr>
<td>3/4 inch Studs</td>
<td>2</td>
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</tr>
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</table>
### TABLE 3

**PROPERTIES OF W21X62 BEAMS**

<table>
<thead>
<tr>
<th>Beam</th>
<th>Area in.²</th>
<th>Depth in.</th>
<th>Flange Width in.</th>
<th>Web Thickness in.</th>
<th>Moment of Inertia in.⁴</th>
</tr>
</thead>
<tbody>
<tr>
<td>CC-3F*</td>
<td>17.47</td>
<td>21.05</td>
<td>8.25</td>
<td>0.584</td>
<td>1267.3</td>
</tr>
<tr>
<td>CC-4F*</td>
<td>17.71</td>
<td>21.05</td>
<td>8.25</td>
<td>0.585</td>
<td>1276.4</td>
</tr>
<tr>
<td>CC-3F**</td>
<td>18.23</td>
<td>20.99</td>
<td>8.24</td>
<td>0.615</td>
<td>1326.8</td>
</tr>
<tr>
<td>CC-4F**</td>
<td>18.23</td>
<td>20.99</td>
<td>8.24</td>
<td>0.615</td>
<td>1326.8</td>
</tr>
</tbody>
</table>

* Measured values.
** Handbook values.

### TABLE 4

**PROPERTIES OF COMPOSITE BEAMS**

<table>
<thead>
<tr>
<th>Beam</th>
<th>Moment of Inertia in.⁴</th>
<th>Position of Neutral Axis From Bottom in.</th>
<th>Moment of Inertia in.⁴</th>
<th>Position of Neutral Axis From Bottom in.</th>
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<tr>
<td>CC-3F*</td>
<td>3462.18</td>
<td>19.79</td>
<td>1630.13</td>
<td>12.03</td>
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<tr>
<td>CC-4F*</td>
<td>3535.44</td>
<td>19.89</td>
<td>1896.06</td>
<td>12.97</td>
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<tr>
<td>CC-3F**</td>
<td>3576.01</td>
<td>19.61</td>
<td>1689.56</td>
<td>11.94</td>
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<tr>
<td>CC-4F**</td>
<td>3620.86</td>
<td>19.75</td>
<td>1946.69</td>
<td>12.88</td>
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* **
<table>
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<tr>
<th>Beam</th>
<th>Batch</th>
<th>No. of Tests</th>
<th>Age (days)</th>
<th>Tensile Strength T (psi)</th>
<th>Compressive Strength $f_c$ (psi)</th>
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<td>Mean</td>
<td>Mean</td>
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<tr>
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<td>1</td>
<td>28</td>
<td>552</td>
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<td>3</td>
<td>28</td>
<td></td>
<td></td>
<td>5795</td>
</tr>
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<td>1</td>
<td>28</td>
<td>543</td>
<td>5435</td>
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<tr>
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<td>3</td>
<td>28</td>
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<td>3</td>
<td>28</td>
<td>5647</td>
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<td>2</td>
<td>1</td>
<td>28</td>
<td>473</td>
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<td>28</td>
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<td>5501</td>
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<tr>
<td>CC-4F</td>
<td>1</td>
<td>3</td>
<td>176</td>
<td>6744</td>
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<td>2</td>
<td>2</td>
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<td>6557</td>
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</tbody>
</table>
### TABLE 6

DEAD LOAD STRESSES IN STEEL BEAM

Flexural Stress in W21X62 (D.L.)

<table>
<thead>
<tr>
<th>Test Beam No.</th>
<th>Dead Load (k/ft.)</th>
<th>At Load Points BTM (ksi)</th>
<th>Top (ksi)</th>
<th>At Interior Support BTM (ksi)</th>
<th>Top (ksi)</th>
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<tbody>
<tr>
<td>CC-3F*</td>
<td>0.437</td>
<td>1.91</td>
<td>-1.91</td>
<td>-3.41</td>
<td>3.41</td>
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<td>0.437</td>
<td>1.90</td>
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<td>0.437</td>
<td>1.82</td>
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<td>3.24</td>
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<tr>
<td>CC-4F**</td>
<td>0.437</td>
<td>1.82</td>
<td>-1.82</td>
<td>-3.24</td>
<td>3.24</td>
</tr>
</tbody>
</table>

*W21X62 values by measurement

**W21X62 values by a handbook of AISC

### TABLE 7

AVERAGE FORCES ON STUD SHEAR CONNECTORS

IN NEGATIVE MOMENT REGION

<table>
<thead>
<tr>
<th>Cycles (x10^6)</th>
<th>CC-3F</th>
<th>Design</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Test</td>
<td>(kips)</td>
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<tr>
<td>0</td>
<td>1.89</td>
<td>4.40</td>
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<tr>
<td>0.1009</td>
<td>2.36</td>
<td>4.40</td>
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<tr>
<td>0.6012</td>
<td>2.59</td>
<td>4.40</td>
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<tr>
<td>1.102</td>
<td>2.89</td>
<td>4.40</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Cycles (x10^6)</th>
<th>CC-4F</th>
<th>Test</th>
<th>Design</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(kips)</td>
<td>(kips)</td>
<td>(kips)</td>
</tr>
<tr>
<td>0</td>
<td>2.72</td>
<td>4.40</td>
<td></td>
</tr>
<tr>
<td>0.1994</td>
<td>2.71</td>
<td>4.40</td>
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<tr>
<td>0.6078</td>
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<tr>
<td>1.0133</td>
<td>2.62</td>
<td>4.40</td>
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</tr>
</tbody>
</table>
Sections A and B are shown on fig. 2

Fig. 1 Details of Composite Beams of CC-3F and CC-4F
Fig. 2  Typical Beam Cross-Section for Beams
Fig. 3 Details of the Steel Beams for CC-3F and CC-4F
Fig. 4 Strain Gage Locations on W21X62 for Beams CC-3F and CC-4F
Fig. 5 Locations of Deflection, Rotation, and Slip Gages for Beam CC-3F
Fig. 6 Beams CC-3F and CC-3F Test Setup

Supports for Longitudinal Stability

Sections A-A
Fig. 7 Fracture of Reinforcement at the West Span Near the Center Support of Beam CC-3F

Fig. 8 Sound Connectors at the West Side Near the Dead Load Point of Contraflexure in the Negative Moment Region of Beam CC-3F
Fig. 9  Fracture of Studs at the East Span Near the Dead Load Point of Contraflexure in the Negative Moment Region of Beam CC-4F
Fig. 10 Load-Deflection Curve for CC-3F (West Span)
Fig. 11 Typical Strain Distribution in Negative Moment Regions for Beams
Fig. 12 Location of Neutral Axis at Section 4 in Negative Moment Regions (Typical)
Fig. 13  Average Load-Strain Curves of Reinforcing Bars at Section 3 of Beam CC-3F
Fig. 14 Average Load-Strain Curves of Reinforcing Bars at Section 3 of Beam CC-4F
Fig. 15 Strain Under Studs in Negative Moment Region (West Span of CC-4F)
Fig. 16 Strain Readings versus Cycles - Beam CC-4F
Fig. 17 Slip Distribution on Beams CC-3F and CC-4F
Fig. 18 Distribution of Slab Cracking in Negative Moment Regions of Beams
Fig. 19 Moment Distribution for Beam CC-3F
Figure 20: Force in Longitudinal Reinforcement of Beam CC-3F
Fig. 21 Force in Longitudinal Reinforcement of Beam CC-4F
Experimental Results:

- △ 0 Cycles
- △ 2,000,000 Cycles

Fig. 22 Slab Force Curve for Beam CC-4F
Fig. 23 Slab Force Curve for Beam CC-3F
Experimental Results:

- \( \Delta \) 0 Cycles
- \( \Delta \) 2,000,000 Cycles

Fig. 24 Slab Force Curve for Beam CC-2F
Experimental Results:

\[ \Delta \text{ Zero Cycles} \]

Fig. 25 Slab Force Curve for Beam CC-1F
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