AN ASSESSMENT OF FATIGUE DAMAGE IN THE
NORFOLK AND WESTERN RAILWAY BRIDGE 651
AT HANNIBAL, MISSOURI

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TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>1.1 Purpose</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Description of Bridge</td>
<td>1</td>
</tr>
<tr>
<td>1.3 History of Modifications and Repairs</td>
<td>3</td>
</tr>
<tr>
<td>1.4 Objectives of Study</td>
<td>5</td>
</tr>
<tr>
<td>2. FIELD INSPECTION</td>
<td>6</td>
</tr>
<tr>
<td>2.1 Inspection of Welded Lap Splices</td>
<td>6</td>
</tr>
<tr>
<td>2.2 Cracks in Members of Bottom Lateral System</td>
<td>9</td>
</tr>
<tr>
<td>2.3 Cracks in Floor Beam Triangular Patch Plate Welds</td>
<td>10</td>
</tr>
<tr>
<td>2.4 Summary</td>
<td>12</td>
</tr>
<tr>
<td>3. STRAIN GAGING AND FIELD MEASUREMENTS</td>
<td>13</td>
</tr>
<tr>
<td>3.1 Strain Gaging</td>
<td>13</td>
</tr>
<tr>
<td>3.2 Field Measurements and Testing</td>
<td>14</td>
</tr>
<tr>
<td>4. GLOBAL ANALYSIS OF BRIDGE SPANS</td>
<td>16</td>
</tr>
<tr>
<td>4.1 Modeling Assumptions</td>
<td>16</td>
</tr>
<tr>
<td>4.2 Support Conditions</td>
<td>18</td>
</tr>
<tr>
<td>4.3 Loading Conditions</td>
<td>19</td>
</tr>
<tr>
<td>4.4 Comparison of Measured and Analytical Responses</td>
<td>19</td>
</tr>
<tr>
<td>5. FIELD MEASUREMENTS AND GLOBAL ANALYSIS RESULTS</td>
<td>23</td>
</tr>
<tr>
<td>5.1 Measured-Strain Observations</td>
<td>23</td>
</tr>
<tr>
<td>5.2 Analytical Response of Truss Members</td>
<td>25</td>
</tr>
<tr>
<td>Section</td>
<td>Page</td>
</tr>
<tr>
<td>------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>5.3 Analytical Response of Floor Beams</td>
<td>26</td>
</tr>
<tr>
<td>5.4 Influence of Bottom Laterals on Overall Span Behavior</td>
<td>29</td>
</tr>
<tr>
<td>6. ANALYSIS OF FLOOR BEAM - HANGER - BOTTOM LATERAL CONNECTION</td>
<td>32</td>
</tr>
<tr>
<td>6.1 Refined Global Analysis</td>
<td>32</td>
</tr>
<tr>
<td>6.2 First Level Substructure Modeling of Floor Beam 7</td>
<td>35</td>
</tr>
<tr>
<td>6.3 Results of First Level Substructure Analysis</td>
<td>37</td>
</tr>
<tr>
<td>6.4 Measured Floor Beam Stresses and Behavior</td>
<td>39</td>
</tr>
<tr>
<td>6.5 Correlation of Substructure Analysis Results to Measured Test Strains</td>
<td>41</td>
</tr>
<tr>
<td>6.6 Second Level Substructure Analysis of Web Gap</td>
<td>43</td>
</tr>
<tr>
<td>7. TRAFFIC ANALYSIS AND PREDICTED STRESS CYCLES</td>
<td>47</td>
</tr>
<tr>
<td>7.1 Traffic Study</td>
<td>47</td>
</tr>
<tr>
<td>7.2 Stress Cycle Assessment</td>
<td>53</td>
</tr>
<tr>
<td>8. FATIGUE RESISTANCE OF WROUGHT IRON SPLICES</td>
<td>56</td>
</tr>
<tr>
<td>8.1 Repair Details</td>
<td>56</td>
</tr>
<tr>
<td>8.2 Physical Properties</td>
<td>57</td>
</tr>
<tr>
<td>8.3 Fatigue Tests</td>
<td>58</td>
</tr>
<tr>
<td>8.3.1 Testing of Fabricated Specimens</td>
<td>59</td>
</tr>
<tr>
<td>8.3.2 Testing of Span F Hangers</td>
<td>61</td>
</tr>
<tr>
<td>8.4 Examination of Results</td>
<td>62</td>
</tr>
<tr>
<td>8.5 Summary and Conclusions</td>
<td>64</td>
</tr>
</tbody>
</table>
9. EVALUATION OF CUMULATIVE DAMAGE IN WELDED AND RIVETED WROUGHT IRON MEMBERS

  9.1 Span D
  9.2 Span F
  9.3 Reinforcement Members
  9.4 Summary and Conclusions

10. FATIGUE CRACKING IN THE FLOOR BEAMS

  10.1 Welded Reinforcement Along Patch Plates
  10.2 Retrofitting the Floor Beam Web at End Connections

11. RETROFIT AND INSPECTION RECOMMENDATIONS

  11.1 Welded Lap Splices
  11.2 Riveted Wrought Iron Hangers
  11.3 Bottom Laterals in Spans C, F, and G
  11.4 Floor Beam - Hanger - Lateral Connections
  11.5 Handrails of Truss Members

12. SUMMARY AND CONCLUSIONS

  TABLES

  FIGURES

  REFERENCES
1. INTRODUCTION

1.1 Purpose

The purpose of the studies reported herein was to examine the welded repairs and strengthening of details on the members of Norfolk and Western Railway Bridge 651 at Hannibal, Missouri. These weld repairs were known to result in low fatigue strength details. Since the structural members were wrought iron with welded steel reinforcement, it was desired to evaluate how serious the resulting welded details were and to assess the degree of cumulative damage that may have occurred. In addition, the reinforcement of some of the floor beams was carried out, because fatigue cracks had developed in the original riveted members. The causes of any fatigue cracks was to be assessed, and recommendations for corrective action and retrofitting are to be made.

1.2 Description of Bridge

The bridge is an eight span, 1582 ft. long single track railroad bridge, owned and operated by the Norfolk and Western Railway Company. It crosses the Mississippi River and is located on the east side of Hannibal, Missouri, about 100 miles north of St. Louis. The bridge is part of Norfolk and Western's main rail corridor through the midwest.

The structure consists of four identical simply supported Pratt trusses (Spans A through D) from east to west, each with a span of 176 ft. - 4 in., two simply supported through trusses (E and F) with spans of 246 ft. - 3 in. and 176 ft. - 4 in., respectively, a swing span
through truss (G), 358 ft. - 3 in. long, and a plate girder approach span (H), 68 ft. - 3 in. long. Figure 1.1 shows an elevation sketch of the bridge. Figures 1.2 and 1.3 show views looking northeast and west, respectively.

The bridge was built in 1888 by the Detroit Iron and Bridge Works and was constructed of riveted built-up wrought iron sections and eye bars.

Trusses A, B, C, and D each consist of nine panel points L0 through L8, 22 ft. - 6 in. apart. The truss heights and widths are 28 ft. and 19 ft. - 6 in., respectively. The upper chord members, end posts, and vertical hangers are constructed of built-up channels, angles, plates, and lattice bracing, as shown in Fig. 1.4. The diagonal and lower chord members consist of either two or four eye bars, as shown in Figs. 1.4, 1.5, and 1.6. Counters which run from U4 to L3 and L5, respectively, consist of two wrought iron eye bars with turnbuckles and two welded steel bars that were added in the 1930's. Floor beams L1 through L7 are 42 in. deep with web plates and riveted flange angles, as shown in Fig. 1.7. The top lateral cross bracing frames into the chord panel points, and the bottom lateral cross bracing frames into the bottom flanges of the floor beams, as shown in Figs. 1.7 and 1.8.

Span E, replaced in 1982 (because of a barge collision) with a welded steel truss, consisting of built-up and rolled sections is slightly higher than Spans A, B, C, D, and F. Span F, which has the same type of members and a similar floor beam-lateral-stringer system as spans A through D, is 36 ft. - 0 in. high, 19 ft. - 6 in. wide and consists of ten panel points L0 to L9.
The Swing Span G contains eighteen panel points L0 to L17, of varying height. Member construction is similar to those of Spans A, B, C, D, and F, with the exception of the lower chord which was fabricated using built-up channels and lattice bracing in place of eye bars.

1.3 History of Modifications and Repairs

The original structure consisted of Spans A through G with Spans E and F each 246 ft.-3 in. long and Span G being on the extreme west bank of the river.

In 1912 Span F was shortened to its present length of 176 ft.-4 in. The Swing Span G was moved away from the shore, and Span H was added. This was done to accommodate wider barge traffic.

Between 1923 and 1937 several counters, diagonals, vertical hangers, and lower chord members in each of the spans were apparently loose and were shortened by cutting the eyebar and splicing the section with welded steel double lap splices. Examples of these repairs are shown in Figs. 1.5, 1.9, and 1.10. During the same period many of these members were strengthening or replaced using welded steel bars and splice plates.

In 1943 cracks were discovered in a number of floor beams in Spans A through F, at the beveled angles in the corners of the bottom flange to vertical hanger connections, as shown in Fig. 1.11. Triangular shaped patch plates were welded onto both sides of the floor beam webs at these corners to strengthen the cracked regions. One such patch plate can be seen in Fig. 1.7. At the same time the pin-connected bottom laterals,
which were connected to the floor beam bottom flanges, were replaced with rolled carbon steel tee sections (WT8x22.5) and steel gusset plates. These are also shown in Fig. 1.7, although the stem of the tee was pointed up at their installation in 1943.

In 1975 the original stringer system in all eight spans was replaced with two rolled steel sections (W33x116), field bolted to the existing floor beams using connection angles. The bottom laterals were reversed at that time and bolted to the bottom flanges of the stringers at points of intersection in all spans, as shown in Fig. 1.7. Web doubler plates were also installed with high strength bolts, on both faces of the floor beam between the stringers for all eight spans.

The original stringer system, which was built-up wrought iron members with web plates and flange angles, consisted of two interior main stringers 2 ft. - 6 in. deep, and two outer stringers 2 ft. - 0 in. deep (see Fig. 2.9). The outer stringers helped support a bridge deck which carried highway traffic up until 1936 when a highway bridge was erected downstream.

In May of 1982 Span E was rammed by a barge and was destroyed. The span was replaced with the present welded steel truss in August 1982.

During replacement of Span E, inspections of the other spans revealed cracks at several welded splices on hangers and diagonals and at several of the floor beam triangular patch plates. Since the reoccurrence of the cracks in the floor beams implied that their strengthening by using patch plates was not effective, and that the cracks could lead to possible interruption of service on the bridge, a thorough evaluation of the cracking was initiated.
1.4 Objectives of Study

In order to evaluate the influence of the welded reinforcement on the wrought iron members, the following studies were carried out:

1. Field inspection of bridge structure and repaired details.


3. Fatigue tests of welded wrought iron bars in order to establish their fatigue behavior.

4. Analysis of the bridge spans under train loads and evaluation of stresses in the component members and details.

5. Evaluation of the cumulative damage in the various members.

6. Development of retrofit recommendations and corrective procedures.
2. FIELD INSPECTION

During the period October 28 to November 5, 1982 a detailed field inspection and data acquisition program was carried out. The areas of interest were members containing welded lap splices which were present in Spans A, B, C, D, F, and G, the floor beam to hanger connections and the bottom lateral system.

Spans E and H were not inspected in detail, because Span E had just been erected, and previous inspections of girder span H revealed no crack problems.

2.1 Inspection of Welded Lap Splices

The most serious cracks were discovered at the welded double lap splices of the outside bars of vertical hangers M1-U1 and M8-U8 of Span F. These hangers consist of two eyebars each which were shortened and reconnected by welding and adding double lap steel splice plates, as shown in Fig. 1.9. This reinforcement was carried out in 1937. It was observed that the load was being carried in the outside bars of the first and last hangers in both the north and south trusses. The inside bars at each of these four locations were loose and did not carry any load. Each inside bar could be moved during passage of a train.

Figure 2.1 shows the crack in the wrought iron hanger at the upper end of the outside upstream splice plate of M8-U8 in the north truss. The crack had coalesced over the full width of the weld toe. A crack was also observed at the weld filled center of the double lap splice joint,
as shown in Fig. 2.2. This crack did not appear to have propagated into the splice plates. The gap in the cut and spliced wrought iron eyebar was found to be only partially filled with weld metal.

The gap in the lap joint was typical of all members which had welded lap splices. Small cracks were found in other eyebars which had lap splices, but none of the cracks appeared to have penetrated into the splice plates.

The outside spliced bar at M8-U8 in the downstream truss was found to have cracks at each end of the splice plate at the weld toe. These cracks can be seen in Figs. 2.3 and 2.4. Similar cracks were also found in the outside bar of member M1-U1 of the downstream truss for Span F. Hence, all three hanger members with double lap splice plates experienced cracking at the weld toe with penetration into the wrought iron bars. Hanger M1-U1 of the upstream truss did not contain the repair detail.

Examinations of the diagonals which had splice plates, revealed small toe cracks at several of the weld splice details but did not appear to penetrate the spliced bars a significant amount. Figure 2.5 shows a small crack at the weld toe of diagonal L4-U3 of the upstream truss in Span B. This was typical of the cracks found at these details.

Strengthening of the counters of Spans A, B, C, D, and G was accomplished by adding steel bars which were connected to the panel points by U-shaped parts and welded splice plates, as shown in Fig. 2.6. Many of these details contained either plug or slot welds on the back side of the plates. Inspection of these welds revealed small cracks at the weld toes.
and in the weld metal. However, as in the welded lap splices of the diagonals, the cracks had not penetrated into the base metal. Figure 2.7 shows a small crack in the slot weld on counter L1-U2 of the downstream truss in Span G. It was observed that many of these added reinforcement members were loose and not fully sharing the load.

As was the case with the crack in the slot weld, inspection of the bridge details was difficult due to the recent painting of the structure. On many welded details it was necessary to sandblast and burn away the paint in order to expose the small cracks. Liquid penetrant was also used to enhance the crack.

While inspecting the built-up vertical hangers of Spans A, B, C, D, and F it was observed that handrails had been welded to the channel flanges and lattice bracing. Small cracks were found at the weld toes at several of these locations. An example is shown in Fig. 2.8. This was true primarily with hangers L1-U1 and L7-U7 of Spans A through D and in hangers L1-M1 and L8-M8 of Span F. The interior verticals, as determined by the arrangement of the counters, would be in compression under live load. This was later verified by analysis.

As was the case with the counters and diagonals, the cracking at the handrail connections did not appear to penetrate very far into the wrought iron members and posed no immediate problem.

It was also noticed that on Span E (the new welded span) the handrails were welded to the verticals at several locations. Although no cracks were detected, the possibility of cracking in the future is present, given a sufficient number of stress cycles.
2.2 Cracks in Members of Bottom Lateral System

Examination of the floor beam bottom lateral system revealed fatigue cracking in these component members. It was found that most of the bottom laterals in Spans C, F, and G had a flame-cut notch in the web of the tee section. These notches were apparently made in 1943 when installation of the stringer bracing system called for the notching of the upright stem at the stringer brace as shown in Fig. 2.9.

In 1976, when the stringers were replaced, the laterals of Spans F and G and the laterals in the middle panels of Span C were inverted, thus pointing the notched stem down. This was done in order to clear the new stringers and permit the laterals to be bolted to the bottom flanges of the stringers.

Figure 2.10 shows a view of one set of intersecting bottom laterals in Span G. The flame-cut notch in the stem can be seen near the intersection. Figure 2.11 shows an oblique view of a flame-cut notch with a small fatigue crack on the left side. The crack can be seen better in Fig. 2.12, which is a close-up view of the reentrant corner.

Nearly all the flame-cut notches which were inspected had cracks. Figures 2.13 and 2.14 show two of the deeper notches where the cracks had propagated up the stem into the flanges of the tees.

Spans A, B, and D also had new laterals installed in 1943. The stems of these tees were continuous and pointed down, thus no notches were made and no cracks were detected.
Several large fatigue cracks were observed in the bottom lateral connection plates at end panel points L0 and L8 of Spans A through D. Figures 2.15 and 2.16 show the configuration of the connections and the cracks that formed at the reentrant corners where the connecting weld terminates.

2.3 Cracks in Floor Beam Triangular Patch Plate Welds

Many of the bottom corners of the floor beam angles to connection angle junctions, as shown in Fig. 2.17, showed signs of cracking along the edges of the welded triangular patch plates. Figure 2.18 shows a crack forming out of the reentrant corners of the beveled intersection of the bottom flange angle and connection angle on the northeast face of Floor Beam 2 in Span D.

Cracking was also observed along the horizontal and vertical patch plate welds. Figure 2.19 shows a close-up view of a horizontal crack which formed at the intersection of the 45° weld and horizontal weld. The crack had coalesced along the horizontal weld between the bottom flange angle and reinforcement patch plate. None of the cracks, however, appeared to penetrate into the flange angle. The cracks remained along the fusion line. A crack in the vertical weld of the connection angle to reinforcement patch plate connection on the upstream west face of Floor Beam L3 in Span B can be seen in Fig. 2.20. The crack appeared to grow out of the beveled intersection of the connection angle and bottom flange angle. Most of these beveled intersections had a gap, as can be seen in Figs. 2.17, 2.18, and 2.20.
In addition to the cracks forming at the lower end of the vertical welds, cracking also developed at the upper end of the vertical welds between the reinforcement patch plate and the connection angle. Figure 2.21 shows a patch plate on the upstream east side of Floor Beam 3 in Span D which developed a crack at the weld termination. The arrow points toward the crack. A close-up view of the crack which extends into and beyond the rivet hole is shown in Fig. 2.22.

Figure 2.23 shows a similar crack that formed at the top of the patch plate on the northeast face of Floor Beam 3 in Span C. The crack extends from the weld termination into the rivet hole, as shown by the close-up view in Fig. 2.24. These cracks were typical of the connection angle cracks that had formed.

Cracks in the original connection angles have led to their replacement at several locations. The riveted connection to the hangers were replaced with new steel connection angles and high strength bolts. Vertical welds were then made between the new connection angles and the patch plates. An example of a replaced connection angle is given as Fig. 2.25. Inspection of one of these repairs on Floor Beam 4 of Span B revealed that cracks had reinitiated at the top corner of the patch plate to connection angle weld, as shown in Fig. 2.26.

At a number of the floor beams the beveled angle gap was filled with weldment. Figure 2.27 shows the filled-in bevel of Floor Beam L7 in Span D and a short weld between the remaining portion of the coped bottom flange angle and connection angle. A small crack, highlighted by rusting, can be seen in this short weld.
In the attachment of the floor beam to the vertical hangers, the original plan called for the coping of the bottom flange angles, so flame-cut right angle notches were made. Small cracks, as shown in Fig. 2.28, have formed at the corners of many of these notches. The attachment by welding of the end of the coped angle to the hanger likely increased the cyclic stresses at the notch causing the crack to form.

2.4 Summary

Of all the cracks found, only the cracks in the weld splices of the outside eyebars at M1-U1 and M8-U8 in Span F appeared to be large. These cracks were subsequently found to penetrate into the bars for about 1/8 in.

Furthermore, there were two eyebars at each of these four locations, with the inside bars being loose and not carrying any load. Should sudden fracture of an outside cracked eyebar occur, it would shift the load to the inside eyebar. Because of this redundancy, the existence of these cracks did not endanger the structure.
3. STRAIN GAGING AND FIELD MEASUREMENTS

Concurrent with the field inspection was the strain gaging and monitoring of selected members in Spans C, D, F, and G. The inspection of these spans helped to establish the members and the approximate locations of the gages which were to be mounted. A total of 53 electrical resistance strain gages were installed.

3.1 Strain Gaging

While inspecting the counters and diagonals of Spans C, D, and F, which had been shortened and strengthened, some of the bars which comprise the overall member were found to be loose and carrying little or no load. In order to determine the stress distribution and variations of these members, gages were installed on each bar. The members selected are listed in Table 3.1. Figures 3.1 to 3.5 show some of the members and gage locations.

The second group of members whose behavior was of concern were in the floor beam-hanger-bottom lateral system which was the same for Spans A, B, C, D, and F. The inspection of the cracks in the welds of the floor beam patch plates and connection angles suggested that the cause was due to out-of-plane distortion of the floor beams at the bottom flange-to-lateral connection.

The bottom laterals in Spans A, B, C, D, and F frame into the lower chord panel points through gusset plates which are attached only to the bottom flanges of the floor beams, as can be seen in Fig. 3.6. In addition, to allow for the attachment of the floor beam to the hanger, the
bottom flanges were coped to clear the channel flanges, as was shown in Figs. 2.27 and 2.28. As a result, the top and bottom flanges of the floor beams were not connected to the hangers. This arrangement would permit out-of-plane bending of the floor beams if differential forces exist in the laterals. In order to monitor the behavior of the bottom flange-to-lateral bracing connection, strain gages were mounted in these areas.

The floor beam-hanger-bottom lateral system between Panel Points L6 and L8 on Span D were chosen due to the availability of a shed to house the strain recording equipment. Thirty strain gages were mounted on the floor system of Span D and two gages on the floor beam web of Span G. Figures 3.6 to 3.8 show samples of gage locations on Floor Beams L6 and L7 and on a lateral gusset. Figures 3.9-3.11 show the exact gage locations on the instrumented floor beams. Gage locations are also summarized in Table 3.2.

3.2 Field Measurements and Testing

From October 31 to November 5 strains from twenty eastbound and westbound trains were recorded. The direction, number of engines, cars and passage time were recorded for each train. These data are summarized in Table 3.3.

Strain traces were recorded on ultraviolet light sensitive paper using two nine channel Honeywell CRT Visicorders. Because only eighteen gages could be recorded at any one time, several groups of gages were monitored during the test period. Figure 3.12 shows the recording equipment and temporary shed.
In order to explore the cyclic stress conditions in the floor beam-lateral system prior to the stringer replacement in 1975, laterals between Panel Points L6 and L8 of Span D were unbolted and disconnected from the stringers. This was done after several trains had been recorded with the laterals connected and before the test train runs.

A test train of known axle weight and wheel spacing was used to establish a controlled data base. This enabled correlation of the field measured stresses with the computed values under the same loading conditions. It also provided a means to establish load-stress relationships and determine the stress distribution among the bridge members at a given instant under known load conditions. By operating the same test train at different speeds the effect of impact on the bridge could be examined. The possible directional effects of eastbound and westbound trains due to traction forces on the stringer-lateral system could be detected.

The test train is shown in Fig. 3.13 and consisted of three diesels, two 150 ton rated freight cars and a caboose. It was run across the bridge in both directions, each at 15 mph and 30 mph. This set of four test train passages was performed three times in order to record strains for all gages. Table 3.4 summarizes the test train directions and speed. A typical strain trace for Gage 68R on Hanger M1-U1S in Span F, which yielded the largest strains among all gages, is shown in Fig. 3.14.

The results of the field measurements are discussed in Chapters 4, 5, and 6.
4. GLOBAL ANALYSIS OF BRIDGE SPANS

The results from the inspection of the floor beam-lateral bracing system of Spans A, B, C, D, and F suggested that fatigue cracks in and around the welds of the patch plates were being caused by out-of-plane distortions, induced by the lateral connection. In order to determine the effects of this eccentric connection, a finite element analysis was required.

Since the floor systems of the spans were identical and because they all had experienced cracking, analysis of only one span was required. A three-dimensional space frame analysis of Span D was performed using Program SAP IV(2), because the span was the most extensively strain gaged.

4.1 Modeling Assumptions

In order to keep the total number of finite element nodal points reasonable and to reduce computing time, symmetry about the longitudinal center plane of the bridge was employed. Thus, only the north (upstream) half of the span was modeled. A total of 173 nodal points were used in conjunction with truss, beam, and plate bending elements. Figure 4.1 shows a computer generated plot of the finite element model.

Several assumptions regarding the modeling of the members and connection details were made:

1. Each of the eyebars which comprised the lower chord and diagonal members were considered fully effective in sharing the member load. However, only the two steel reinforcing bars
of the counters were considered effective in carrying load. This was decided because of the loose outside original bars found during the inspection. The cross-sectional areas of the tight bars in each member were added together to form the area of equivalent truss element.

2. The upper chord members which were fabricated using web plates, channels, angles and lattice bracing were also modeled as truss elements with the contribution of the lattice bracing being ignored.

3. The main end posts, vertical hangers, interior verticals, and top portal struts were modeled as beam elements with equivalent section properties.

4. The stringers were modeled using plate bending elements for the webs and beam elements for the flanges. The stringer depth and section properties were modified in order to incorporate the lateral bracing connections. In addition, stringer to floor beam connections were considered simply supported against out-of-plane rotation.

5. The floor beams were also modeled using plate bending elements for the webs and beam elements for the flanges. Out-of-plane (horizontal) restraint between the floor beam top and bottom flange and hanger were assumed simply supported as in the floor beam-stringer connection.
6. The top and bottom lateral bracing members were modeled as beam elements with the top laterals framing into the top chord panel points. The bottom laterals were attached to the bottom flanges of the stringer between Panel Points L0 to L6 and to the bottom flanges of all floor beams at a distance of 14.25 in. from the panel points. Between Panel Points L6 and L8 the bottom laterals were not connected to the stringers to simulate the condition of the bridge span during test measurements.

7. The axial and bending stiffness contributions of the rails and ties were ignored.

4.2 Support Conditions

Consideration was given to the span end support conditions in order to examine the effects on member stresses. Other studies of bridges have indicated that their effects could be significant\(^{3,4}\). Original design specifications for the spans called for hinges at the east end of the trusses and roller supports at the west end. Equivalent support conditions were also used for the end stringers at the piers.

In 1975, with the replacement of the stringers, neoprene bearing pads were inserted under the bottom flanges at each pier with the east end pads having regular holes and the west end pads having short slotted holes for the anchoring bolts. Thus, any longitudinal forces exerted on the stringers would be resisted by the supports at the east end. No unusual conditions of the supports were noticed during the field inspection and measurement period.
The computer analysis of Span D indicated that the use of simple supports for both stringer and truss, with hinges at the east end and rollers at the west end gave the best agreement with the measured traces of overall bridge response. Thus, these support conditions were used for all subsequent analyses.

4.3 Loading Conditions

In order to correlate the analytical results with the measured test train strain versus time variations, 21 static load cases were used which simulated the movement of the test train across the span. The loads were applied as concentrated node loads acting directly on the top flanges of the stringers at the intermediate nodes and on the top nodes of the floor beam-stringer connections. Figure 4.2 shows the engines and cars used for the test train. Wheel spacing was adjusted in order to load the stringer nodes.

The output stresses from the computer were plotted for select members, versus the position of the first axle to form stress-time curves (influence curves).

4.4 Comparison of Measured and Analytical Responses

Accuracy of the analysis was examined by comparing the theoretical stress versus load position (time) response of the gaged members to the actual strain responses. The analog traces corresponding to the westbound passage of the test train at 15 mph were used. This not only corresponded to the load conditions of the computer analysis but also approximated a
static live loading of the real bridge (the effects of train velocity will be discussed in Chapter 5).

Figure 4.3 gives the comparison of the measured and theoretical stresses in lower chord member L4-L5N. Excellent agreement between the measured and analytical responses was observed. Examination of the measured strain traces for the gages on this member revealed an equal stress distribution among the six component bars which made up the member. The measured peak stress was 7.6 ksi, whereas the theoretical peak stress was 7.3 ksi.

Strain measurements were also acquired for diagonal member L4-U5S in the downstream truss of Span C. Since Spans A through D were identical and each span is symmetrical, direct comparison with the theoretical stress response of L4-U5N in the computer model was possible. Figure 4.4 shows the computed influence curve and the measured equivalent stress-time record. A live load stress reversal was predicted and observed indicating a partial unloading of the dead load stress in the bars. Examination of the strain records from the gage readings on the two eyebars showed a maximum difference of 2.5 ksi between the two bars with the inside upstream eyebar having the higher stress. The strain traces for the two eyebars are compared in Fig. 4.5. Comparison of traces for the two parallel gages on the steel splice plate of the inside eyebar (Fig. 3.1) indicated a strain gradient corresponding to a stress differential of 2 ksi suggesting bending of the eyebar and fixity in the joint. These traces are given in Fig. 4.6.
Figure 4.7 gives measured and theoretical influence curves for counter L3-U4N. As in diagonal L4-L5S, a live load stress reversal is evident. Furthermore, the strain distribution among the four bars was not equal. Examination of the traces for the first recorded train revealed that the outside (upstream) eyebars was carrying no load. Subsequently, a new gage (54R) was mounted on the second bar, directly opposite an existing gage (54W) in order to obtain the strain distribution across the thickness. Test traces revealed a strain gradient across the thickness indicating that the member bent while the span was carrying load. Figure 4.8 compares the traces for the second bar and for the other two effective bars which made up the member.

Figures 4.9 and 4.10 show the "influence curve" comparisons between measured and theoretical stresses for laterals L7N-L8S and L8N-L7S. Stresses for both the top flange and stem of the tees are plotted. The two figures show fairly good agreement between the measured and analytical responses. The live load stress distribution across the depth of the laterals for any position of the train can be deduced. The top flanges of the laterals are always in tension, while the bottoms of the stems are in compression. This shows the presence of both axial and bending stresses. Figures 4.11 and 4.12 show the influence curve comparisons for laterals L6N-L7S and L7N-L6S. Only the top flanges of these members were measured. They too show good agreement between measured and theoretical responses.

Figures 4.13 and 4.14 give the measured and predicted responses of the channel flanges for vertical hanger L7-U7N. The theoretical stresses were computed by adding up the concurrent stresses due to axial force,
in-plane bending and out-of-plane bending for each load case. Comparison of the traces for each flange of the hanger show the west flanges starting off with reduction in stress as the train enters the east end of the span, then increasing in stress as the wheels pass over the panel points. This indicated the presence of bending moments in the hangers.

Figure 4.15 shows measured and analytical stress-time responses of the bottom flange tips of Floor Beam L7. The theoretical stresses were calculated using the axial force and out-of-plane moments from the finite element analysis. Good agreement regarding stress magnitudes and the type of fluctuations was obtained. The stress distribution across the bottom flange for any load position shows horizontal out-of-plane bending of the floor beams as being a significant part of the total stress in the bottom flange. Explanations of this behavior will be discussed in the next chapter.

The agreement between computed and measured stresses also existed for members in Span F, which differs from Spans A, B, C, and D. Figure 4.16 shows the excellent correlation of strain variation for the eyebar of M1-ULS in Span F. This agreement assures the adequacy of evaluating truss member stresses and the floor system member stresses as a result of other loadings while the structure was in service.
5. FIELD MEASUREMENTS AND GLOBAL ANALYSIS RESULTS

Chapter 4 compared the analytical stress-time responses of several members with the measured test responses and showed that the global model gave a good representation of the overall behavior of Span D. This chapter will evaluate the measured data and use the results of the global analysis to examine the interaction of the truss and floor system. Also, two additional cases will be examined to determine the influence of the bottom laterals on the predicted response of the span.

5.1 Measured-Strain Observations

The following conclusions were reached based on the field measurements.

1. Train direction had no measurable influence on the behavior of either the truss or floor system. Examination of strain traces for lower chord L4-L5N and diagonal L4-U5S of Span C for both east and west passages of the test train revealed strains which were similar in magnitude and sign. Figures 5.1 and 5.2 show comparisons of the traces in both directions for each member. They produce the mirror image expected from the loading. Comparisons of strain traces for bottom laterals L7N-L8S and L8N-L7S for the two directions also revealed strains of similar magnitudes and sign. Figures 5.3 and 5.4 give the comparisons for the top flange and stem of the two bottom lateral members. These traces indicate that the traction force of the diesels and cars due to rolling friction did not greatly influence the behavior of either the truss or floor system. The effects of braking of the train were not measured.
2. The effects of impact on the magnitude of strain in the bridge members due to train velocity were negligible. Comparison of the strain-time responses for 15 mph and 30 mph showed no difference in member response. Figure 5.5 gives the strain traces for vertical hanger M1-U1 of Span F which displayed the highest strain variations of all gaged members. The peak stresses for both train speeds were 13.5 ksi, thus implying no measureable impact. Because of the proximity of the bridge to a 90° crossover with another track and to a tunnel just beyond, the higher test train speed is the maximum velocity which can be attained by any train. Therefore, no significant effect is expected as a result of impact for any members of the bridge.

3. The addition of bolted doubler plates, which are located on both sides of the floor beam web between the stringers for all spans, created small vertical gaps between the plates and stringer connection angles. Previous studies\(^{(4,5)}\) have found that out-of-plane distortion can cause high bending stresses to develop in web. Such a condition likely resulted in the cracking that developed at the intersection of the beveled angles. However, gages mounted horizontally in the web gap on the east face of Floor Beam 7 in Span G, produced maximum web gap stresses of only 5 ksi under normal train traffic. Figure 5.6 gives a portion of the traces for the two gages showing the strain variations produced by diesel locomotives. Since the cyclic stresses were low, cracking of the webs along the gap would not be expected.
5.2 Analytical Response of Truss Members

The analytical stress-time responses for each of the lower chord members, counters, diagonals and verticals were examined to determine the members which exhibited the highest stress variations under test train loading. The analytical responses were compared based on the condition of the bottom laterals being disconnected from the stringer between panel points L6 and L8.

Most of lower chord members had computed stress variations of similar magnitude. The maximum live load stress was 8.2 ksi in member L5-L6. This stress was slightly higher than the maximum predicted stress of 7.3 ksi in member L4-L5.

Comparison of the analytical stress-time responses for the diagonals and counters revealed that the end diagonals were subjected to the highest stress ranges (about 10 ksi). However, unlike the intermediate diagonals in the spans, these members did not experience any live load stress reversals. Figure 5.7 shows the predicted response of end diagonal L6-U7. The intermediate diagonals U2-L3 and L5-U6 behaved similar to the end diagonals but had slightly lower stresses. Counter U4-L5 exhibited live load stress excursions into compression similar to the measured stresses in counter L3-U4 as was depicted in Fig. 4.8. The stress fluctuated from 7.3 ksi to 3.5 ksi.

The predicted axial stress and bending in the plane of the floor beam for vertical hangers L1-U1 and L7-U7 were similar. The axial stresses and bending stresses for both members had the same magnitude and sign with peak values of 9.2 ksi and 1.7 ksi, respectively, with the bending stress
producing tension on the floor beam side of the hanger. Examination of the axial stresses in the interior verticals verified that the arrangement of the counters and diagonals resulted in their always being in compression. Thus, the interior verticals were not of concern with respect to possible fatigue cracking.

Comparison of the highest predicted bending stresses in the plane of the truss for each of the vertical hangers revealed bending stresses which steadily increased from a minimum of 0.3 ksi for hanger L1-U1 on the east end of the span to a maximum of 4.3 ksi for vertical L6-U6 near the west end. Hanger L7-U7 was predicted to have a bending stress of 3.2 ksi. This unusual pattern resulted in a compressive bending stress for the west side of each member. Figure 5.8 gives a plot of the computed maximum bending stresses in each member versus its respective panel point location. The causes of this bending, which can be related to the displacement of floor system, will be discussed later.

5.3 Analytical Response of Floor Beams

During the reduction of measured test train data for gages 64R and 64W, which were located on the west and east bottom flange tips of Floor Beam L7 near the stringer connection, it was noticed that the bottom flange was subjected to large stress gradients causing compression on the west edge. The flange tip stresses and the gradients fluctuated with the relative position of the train. Comparisons of traces for each gage for both directions of train movement showed that the gage response reversed itself when the train direction was reversed. Figure 5.9 shows the strain traces of both gages for the two directions.
It can be seen in Fig. 5.9a that as the train enters the span from the west end (Panel Point L8), both sides of the bottom flange are in tension. However, as the train moves further onto the span and induces more load in the bottom chord members, the west side of the flange changes into live load compression while the east side of the flange remains in tension. The average stresses in the flange increase (tension) when a set of axle loads pass directly over the floor beam. As the end of the train leaves the east end of the span, the live load stresses return to zero. The exact opposite pattern, with respect to time, occurs when the train enters the span from the east but the magnitudes of stresses for a given train position remains the same. Figure 5.10 shows the stress gradients across the bottom flange at various time increments for the two directions of the train.

This directional behavior was verified by the comparison of the measured response and the analytical response of Floor Beam 7, as was shown previously in Fig. 4.16 (under the condition of the bottom laterals being disconnected from the stringers between Panel Points L6 and L8).

A comparison of horizontal "out-of-plane" bending moments in the bottom flanges of each floor beam at the stringer connections revealed that each of the floor beams were bending in the same direction, causing compression on the west flange, but with different magnitudes. Floor Beam 1 displayed the lowest flange moment with a peak value of 18.6 k-in., while Floor Beam 7 had the highest peak flange moment of 140 k-in., as shown in Fig. 5.11. This difference in horizontal bending indicated that the stringers and support conditions were influencing the behavior of the floor beams. Earlier analysis on railway truss bridges have shown this to be true (6,7).
To examine this phenomenon further the computed lateral displacements (in the direction of the train) of the floor beam bottom flanges were compared. Table 5.1 lists the midspan displacements and end (at hanger) displacements for each of the floor beams with respect to the hinge supports at the east end of the bridge. Also listed in the last column are the relative displacements between the ends and midspan of the floor beams. The comparison was based on train position which produced the largest displacements. The relative displacements vary from a minimum of 0.0224 in. for the Floor Beam 1 near the hinge supports to a maximum of 0.222 in. for Floor Beam 7 near the truss and stringer roller supports at the west end of the bridge. A second comparison was made in examining the change in horizontal displacement against position of the train. The horizontal displacements of the stringer bottom flange to floor beam connection and the floor beam to hanger connection at Floor Beam 7 are plotted in Fig. 5.12 and compared to show the displacement patterns of the two points. The difference in displacement between the two points for any given load position represent their relative displacements. It is seen that the lower chord panel point always displaces more than the stringer to floor beam connection.

From these results it was concluded that the relative difference in stiffness between the trusses and stringers was causing the out-of-plane lateral movement of the floor beams when the bridge span was under load. The stringers, essentially behaving as two continuous beams with hinge supports at the east end of the span and roller supports at the west end, have less longitudinal displacements in the direction of the span than do
the lower chord panel points of the trusses. Consequently, all floor beams are bent horizontally concave to the west, with Floor Beam 7 being deformed the most. Furthermore, this relative displacement was also the cause of twisting of the floor beams or bending of the hangers in the plane of the truss.

5.4 Influence of Bottom Laterals on Overall Span Behavior

The global analysis has revealed the overall behavior of Span D based on the condition of the span during the test train measurements, that is, the bottom laterals being disconnected from the stringers between Panel Points L6 and L8. This condition existed prior to 1975 but is not the current state of the bridge in which all the bottom laterals are attached to the stringers. To simulate the current condition, a separate global model was made in which all the bottom laterals were attached to the stringers and ends of the floor beams. An analysis was performed using the same load conditions described in Section 5.3. This analysis is referred to as Case 1.

A second analysis based on the lateral arrangement in Span G was performed to determine the effects of attaching the bottom laterals to the panel points as opposed to attaching through the floor beam bottom flanges. Span G has the laterals directly attached to the lower chord at the panel points, as shown in Fig. 5.13, and did not experience cracking in the bottom corners of the floor beams. The modeling of this lateral arrangement is referred to as Case 2. The analysis of Span D with bottom laterals disconnected between Panel Points L6 and L8 is referred to as Case 3.
Table 5.2 summarizes the results of the three cases for the truss and floor system showing comparisons based on computed peak live load stresses.

The different arrangements of bottom laterals had small effects on the predicted stresses in the truss members. The largest difference occurred in lower chord member L6-L7 between Cases 1 and 3 in which the laterals were connected and disconnected respectively. The peak stress for Case 1 was 6.3 ksi, whereas the peak stress for Case 3 was 7.9 ksi. The counters, diagonals and vertical hangers exhibited very small changes in stresses.

The members in the floor system showed significant changes in stresses for the three cases. Comparisons of the bottom laterals, between Panel Points 6 and 8, showed the axial stresses in members L6N-L7S and L7N-L8S to increase when the bottom laterals were disconnected from the stringers (Cases 1 to 3), whereas the axial stresses for laterals L7N-L6S and L8N-L7S decreased. The stresses in the members were generally lower if connected directly to the panel points (Case 2). The bending stresses on the other hand were not necessarily lowered. Significantly, the existence of large forces and stresses in the lateral bracing members, when the bridge span is under traffic load, indicates that these bracing members participate in carrying train loads, not just wind loads and traction forces as normally assumed in design.

During the field measurements, strain versus time responses were recorded for laterals L7N-L8S and L8N-L7S under conditions corresponding to Cases 1 and 3. Plots of the stress distribution across the depths of the tees were made at various time frames for both laterals in order to

-30-
visualize their behavior. The strain measurements were made under normal traffic but different trains, thus only indirect comparisons could be made. The computed analytical stress distributions for various load positions were also plotted and compared to the measured stress distribution for Cases 1 and 3. Figures 5.14 and 5.15 show comparison of the measured and computed stress distributions across lateral L7N-L8S at various instances for the two cases. Similar comparisons for L8N-L7S are given in Figs. 5.16 and 5.17. Connecting the bottom laterals to the stringers reduced the difference of stresses in the flange and at the tip of the stem but increased the average (axial) stresses. This condition most likely contributed to the development of cracks at the flame-cut notches in the laterals as it increased the tensile stress at the notch tip.

Connecting the bottom laterals directly to the truss panel points reduced the lateral bending stresses in the bottom flanges a significant amount (Case 2). Both Case 1 and Case 3 introduced lateral bending stress into the bottom flange that was six to ten times greater than predicted for Case 2. In fact, releasing the stringer-lateral bracing connection actually increased the bending in the floor beam flanges which can be seen from the results summarized in Table 5.2.

Hence, the analysis indicates that the most significant effect of the bottom lateral bracings system is on the floor beam end connection.
6. ANALYSIS OF FLOOR BEAM - HANGER - BOTTOM LATERAL CONNECTION

The measured and theoretical structural response of Span D was in good agreement. The computer analysis and measurements indicated that the stringers were essentially continuous and that the bottom laterals had a major influence on the behavior of the floor beams.

The finite element mesh that modeled the structure did not permit an evaluation of the localized stresses and distortions in the patch plate region of the repaired floor beam connection nor in the web gap of the original connection.

In order to determine these stresses and distortions, a detailed finite element analysis of the floor beam-hanger-bottom lateral connection was performed. Floor Beam 7 was chosen for the study, because it was predicted to develop the highest out-of-plane stresses and deformations, and because it was the most extensively strain gaged floor beam. This would allow for correlation between measured and computed stresses. A three step analysis requiring a refined global analysis and two levels of substructuring was employed.

6.1 Refined Global Analysis

To make the global model more compatible to substructuring, a refined finite element mesh was used employing additional nodal points and elements for the floor beam-hanger connection at Panel Point 7. Figure 6.1 shows a plot of the refined global model. Seventy-six additional nodal points were employed.
As in the original global model the bottom laterals were modeled as beam elements framing into the bottom flanges of the floor beam via point connections 14.25 in. from the centerline of the hanger, ignoring the contribution of the gusset plate.

The results of the global analysis discussed in Chapter 5 indicated that the highest stresses and out-of-plane displacements along the bottom flange of Floor Beam 7 occurred during load case 10, and this load case was used to assess the floor beam.

Stresses and displacements in select cross-sections of Floor Beam 7 were examined to determine how the eccentric lateral connection effected the stress distribution and deformation patterns in the web. Live load induced web surface stresses for both longitudinal (horizontal) and transverse (vertical) directions were computed using Eqs. 6.1 and 6.2

\[
\sigma_x = S_{xx} \pm \frac{6 M_{xx}}{t^2} \quad (6.1)
\]

\[
\sigma_y = S_{yy} \pm \frac{6 M_y}{t^2} \quad (6.2)
\]

where \( S \) is the computed membrane stress and \( M \) the corresponding plate bending moment. Figure 6.2 gives the distribution of longitudinal surface stress on the east face of the floor beam near the stringer and shows a stress variation of 2 ksi in tension near the bottom of the web to -1.2 ksi in compression near the top of the web.
Plots of the longitudinal web surface stresses at the end of the floor beam near and along the hanger connection angle (Figs. 6.3 and 6.4) indicated that the stresses were less than 2.5 ksi. These figures show the stress distribution at the two cross-sections with the highest magnitude of stresses occurring in the connection angle near the top and bottom flanges. The transverse web surface stresses for the same three cross-sections are shown in Figs. 6.5 to 6.7. The plots show that the vertical stresses throughout the floor beam web are low with a peak stress of 1.5 ksi occurring near the bottom flange of the stringer (see Fig. 6.5). However, there was a definite change in the magnitude across the depth. This suggested that the web could be subjected to transverse vertical bending or torsion.

Horizontal out-of-plane displacements and rotations along the floor beam web-bottom flange junction and web-connection angle junctions were examined to see if abrupt changes occurred. Displacements (in the direction of the track) of the nodes which lie along the junction of the floor beam web to vertical leg of the bottom flange angle are plotted in Fig. 6.8. No abrupt changes in displacement near the bottom lateral connection are apparent, although the restraining effects of the stringer on the floor beam out-of-plane movement is evident. This indicated that the displacement mode was not a significant contributor to the change of vertical bending stresses in the web. On the other hand, examination of the nodal rotations about the floor beam longitudinal axis for the same junction are plotted in Fig. 6.9 and show an abrupt change near the bottom lateral connection. This indicated that the laterals were preventing
the region around the connection from moving while the rest of the lower portion of the floor beam was allowed to rotate, resulting in a relative twisting of the bottom flange region.

Horizontal out-of-plane displacements (in the direction of the train) of nodes along the web to connection angle junction are given in Fig. 6.10 and show no unusual displacement patterns. The bottom of the floor beam web displaced more than the top, and very little vertical bending of the floor beam web is apparent. A plot of the rotations about the vertical axis for the same junction is given in Fig. 6.11, and small changes along the depth of the web with the lower portion rotated slightly more than the top but less than at near mid-depth. However, the changes were not as large and as abrupt as along the web bottom flange junction.

The results of the refined global analysis do not provide sufficiently detailed stress and displacement conditions at the floor beam-hanger connections. The stresses and displacements were examined further by using substructure models.

6.2 First Level Substructure Modeling of Floor Beam 7

A first level substructure analysis was performed in order to determine the nominal stress distribution in the floor beam-hanger-lateral connection for both the original condition and the patch-plated condition. This involved the generation of a new finite element model which included a more detailed mesh of the floor beam bottom corner.
The analysis for the original condition of the floor beam was based on the present system of two stringers and gusset plate connected bottom laterals even though cracking of the beveled web gaps occurred while the old system of four stringers and pin connected bottom laterals was still used. This assumption affected the forces and displacements in the floor beam, but the localized behavior of the floor beam bottom corner could still be satisfactorily simulated since the distortion was still present as shown in the refined global analysis in the last section.

To assure the validity of the modeling the boundaries of the sub-structure were placed a satisfactory distance away from the patch plate region, thus conforming to St. Venant's Principle. The boundaries or "cuts" were located at the floor beam to stringer connection, in the vertical hanger 124 in. above the lower chord and at the intersection points for the two bottom laterals halfway between Panel Points L6, L7, and L8. A total of 462 active nodal points were used to define the mesh and 69 reference nodes to support it. The web of the floor beam was modeled using 224 plate bending elements with sizes varying from 2 x 2 in. to 6.8 x 8.1 in. with the smallest elements in the patch plate region. The original condition was analyzed by simply decreasing the element thicknesses to reflect only the floor beam web without the patch plate.  

Figure 6.12 shows a plot of the generated finite element mesh. The model was "held in space" by 138 boundary elements which were located at each of the boundary nodal points and at desired nodal points. The sub-structure was loaded through the boundary elements by imposing the displacements and rotations obtained directly from the output of the refined
global analysis. Interpolation was used to generate the displacement fields for the remaining boundary nodal points.

6.3 Results of First Level Substructure Analysis

Web stresses and displacements were examined for both the original and patch-plated conditions of the floor beam. The results were compared to see how the addition of the patch plates changed the distribution and magnitude of stress in the floor beam-hanger connection.

Figure 6.13 shows a comparison of the distribution of longitudinal surface stress on the east face of the web near the vertical stiffener for both the original and patch-plated conditions. The distribution was almost identical for both conditions, being nearly constant across the depth of the web and showing a maximum live load compressive stress of only -1.25 ksi near the bottom flange for the original condition. Figures 6.14 and 6.15 show comparisons of the longitudinal stress distribution in the web along the connection angle and at the edge of the connection angle. The web surface stress for the original condition varied from zero near the top flange to a maximum tensile stress of 2.5 ksi near the bottom flange as shown in Fig. 6.15. The web stress distribution in the same cross-section for the patch-plated condition varied from zero at the top flange to 1.25 ksi near the bottom flange. A similar stress distribution was obtained along the edge of the connection angle with the stress varying from zero near the top flange to a maximum of 3 ksi near the bottom flange for the original condition and 2 ksi for the patch-plated condition.
Examination of the transverse (vertical) web surface stresses for the same cross-sections also revealed similar distributions for the two conditions. Figure 6.17 gives the stress distributions across the depth of the web near the vertical stiffener indicating zero stress in the web. Plots of the stress distribution in the web along the connection angle and in the connection angle, given in Figs. 6.18 and 6.19, revealed peak stresses of only 1.8 ksi near the bottom flange for the original condition. The addition of the patch plates however increased the stress in the web and connection near the top of the patch plates. Although the peak stress for this condition was quite low, the change did imply that the presence of the patch plates caused a redistribution of the stresses in the region. Also in comparing the transverse stress distribution for the three cross-sections, an increase in the stress magnitude near the hanger is detected.

Comparisons were made of the out-of-plane rotations along the horizontal junction of the web to vertical legs of the bottom flange angles in order to determine the severity of the distortion in the bottom corner of the floor beam. Figure 6.20 gives the plots of the rotations for the two conditions showing sudden changes near the intersection of the bottom flange angles and connection angles. This relative rotation was attributed to the attachment of the bottom laterals which produced a relative twisting of the bottom flange causing vertical bending stresses to develop in the web. The addition of the patch plates decreased the rotations by only a small amount, however, the distortion was still present.

Out-of-plane rotations along the floor beam web to connection angle junction were examined in order to assess bending stresses from distortion
in the web. Figure 6.21 gives the plots for the two conditions revealing an increase in rotation along mid-depth of the web. This increase occurred because the top and bottom flanges of the floor beam, although not attached to the hanger, still provided more restraint for rotation near the flanges than along the mid-depth of the web. The comparison of the two conditions show that the addition of the patch plates decreased the relative rotations but did not eliminate them.

In general the first level substructure analysis revealed that the magnitudes of stress in the floor beam web and hanger connection were low. The attachment of the laterals resulted in rotational distortion in the bottom corner of the floor beam which caused longitudinal and transverse bending stresses to develop. The addition of the patch plates produced only localized changes in web stress distribution but did not eliminate the distortion.

6.4 Measured Floor Beam Stresses and Behavior

Strain records from the test train runs for gages on Floor Beams 6 and 7 were examined in order to evaluate the actual behavior of the floor beams. Figure 6.22 shows traces of vertical gages 62R and 46R which were mounted at mid-depth on the east and west connection angles of Floor Beam 7. Gage 46R was located near a rivet hole and was 1.5 in. above gage 62R. The traces show tensile strains in both connection angles with the higher strains occurring in the west connection angle leg (46R). The maximum stress range for this gage was 6.2 ksi. It should be noted that the gages were not located at identical heights and the presence of the rivet hole
caused stress concentrations which elevated the stress level in the connection angle near gage 46R. Any bending of the angles in the plane of the truss was not apparent from the two strain traces.

Figure 6.23 shows traces for gages 62W and 66R which were also located on the east and west connection angles of Floor Beam 7, 10 in. above the top face of the bottom flange. Maximum stresses (strains) less than 1 ksi were recorded for the two gages. No evidence of bending of the angles was detectable; the gages were close to the bottom of the floor beam where the vertical bending stress is zero.

Strain traces for gage 53R which was mounted adjacent to crack tip on the upstream east bottom flange cope of Floor Beam 7 revealed high tensile strains during the passage of the test train. This is shown in Fig. 6.24. A peak stress of 15 ksi was recorded. The crack was propagating toward a nearby rivet hole and was not considered serious.

Gages 52R and 52W, mounted vertically and horizontally on the east web face of Floor Beam 6 at the top corner of the patch plate to connection angle weld, were not measured during the test train runs. Examination of traces for the two gages recorded under normal traffic revealed peak stresses of only -2.1 ksi and 1.4 ksi, respectively, due to the passage of the engines. The stresses remained near zero for the passage of the cars.
6.5 Correlation of Substructure Analysis Results to Measured Test Strains

The computed stresses of the substructure analysis for the patch plate condition were compared to the measured test train strains at several gage locations to evaluate the accuracy of the model. Because only one load case (Load Case 10) was used in the analysis and the exact location of the test train was not known during measurements, an estimate had to be made of the corresponding location of the measured strain value on the trace.

Figure 6.25 shows the time corresponding to load case 10. The measured trace for the gage was superimposed onto the measured and computed traces for lower chord L4-L5. The resulting measured stresses are summarized in Table 6.1 with the computed values.

Fair correlation was obtained for gages 64R and 64W on the bottom flanges of Floor Beam 7. The measured stress at the west flange tip was 0.5 ksi as compared to a computed stress of -3.65 ksi. The measured stress for gage 64W on the east flange tip was 10 ksi, and the computed stress was 6 ksi. These computed stresses were consistent with the computed stresses of the global analysis discussed in Chapters 4 and 5. They indicate that the out-of-plane flange bending was in good agreement. The normal bending stress was measured to be greater than estimated.

Comparisons of the computed stresses to the measured stresses for gages 46R and 62R on the connection angles was poor. The equivalent measured stresses for the two gages were 3.7 ksi and 2.4 ksi, whereas the computed stresses were 0.44 ksi and 1 ksi, respectively. This poor
correlation was expected, because there were actually three thin plates, consisting of the web and two connection angles, as opposed to one plate of equivalent thickness which was assumed for the analysis. Also, local conditions such as rivet holes could not be incorporated into the finite element model, and as mentioned in Section 6.4, the difference in height of the two gages produced different strain responses. Gage 46R was near a rivet hole. Measured strains for gages 66R and 62W which were also on the connection angles compare more favorably with the computed stresses, although the correlation was still rather poor. Because the magnitudes of the stresses were so low, the comparison was not considered significant.

In spite of the limitations regarding the modeling of the floor beam-hanger connection and the fact that a two level analysis was required, the correlation of the computed stresses to the measured stresses was reasonable. The comparison revealed stresses which were similar in sign and magnitude even though the location of the measured stresses corresponding to load case 10 were approximated.

Although the substructure analysis did reveal local web bending stresses at the bottom corners of the floor beam for both the original and patch plated conditions, it did not explain the causes of cracking in the connection angles and in the horizontal and vertical welds of the patch plates. The effects of the welds with respect to the cracking is discussed in Chapter 10. The stresses in the beveled web gaps of the floor beams are examined next.
6.6 Second Level Substructure Analysis of Web Gap

Although the first level substructure model showed the stress and deformation patterns in the floor beam-hanger connection, it did not incorporate the gap between the beveled legs of the bottom flange angles and connection angles, as shown in Fig. 6.26. Results from analysis of other bridge structures indicated that the cracks, which had developed in these web gaps and led to the addition of the patch plates, were due to high out-of-plane bending stresses. To verify this assumption, a second level substructure model was developed.

In order to simplify the modeling and to save time, two approximations were made:

1. As in the first level substructure model of the "original" floor beam condition, the analysis was based on the present system of stringers and bottom laterals.

2. The web gap between the two beveled angle legs was assumed to be oriented on a 45° angle with the bottom flange even though the actual gap would have required the use of triangular plate bending elements which are not defined for the SAP IV program. Thus to make modeling easier, the legs of the connection angles were assumed to be 4 in. wide instead of 5 in.

Figure 6.27 shows the mesh of the first level substructure model. The heavy lines indicate where "cuts" were made defining the size of the second level substructure model. A total of 253 active nodal points and 111 reference nodal points were used. The floor beam web was modeled
using 117 plate bending elements. As in the previous models, elements corresponding to the flange and connection angle legs had equivalent thicknesses of 1.875 in. and 1.25 in., respectively. The web gap was modeled using three rows of four elements with 0.375 in. thickness. This produced an 0.75 in. gap between the two beveled angle legs.

The bottom flange, gusset plate, and hanger were modeled in the same manner as in the previous analysis using plate bending elements and beam elements. Two beam elements were used to simulate the attachment of the bottom laterals to the gusset plate. A plot of the finite element mesh is given in Fig. 6.28.

The substructure model was supported by 222 boundary elements and was loaded by applying displacements at the boundary nodal points. The displacements were obtained from the output of the first level substructure analysis of the original floor beam-hanger connection.

The analysis resulted in plate bending stresses in the web gap which were many times greater than the nominal stress in the surrounding region and were greater than the nominal yield point of the wrought iron web (about 26 ksi). The highest stresses which were perpendicular to the gap length occurred at the lower edge of the gap. Figure 6.29 shows a sketch of the plate elements in the web gap with the transverse stresses for the east face given at the center of each element. These stresses are estimated values in accordance with all the assumptions disregarding yielding. Nevertheless, the values do show that although the stresses in the floor beam were not high, the combination of the eccentric lateral connection and the small gap could cause very high bending stresses to develop in the floor beam web.
A plot of the rotations along the bottom flange of the floor beam about its longitudinal axis was made to determine the magnitude of the distortion. Figure 6.30 shows the rotations along the bottom row of nodes of the floor beam and across the bottom of the web gap. Large changes in rotations occur at the edge of the gap, revealing the relative movement within the gap region. The magnitude of the rotation changes from 0.003 radians to 0.0011 radians, a factor of 3.

This resulted from loads in the laterals which were transmitted into the floor beams and caused the distortion to be concentrated within the gap, since its bending rigidity was much less than the bending rigidity of the beveled angles. This caused the web to "kink" and resulted in high out-of-plane bending stresses. The repeated loading on the floor beams caused fatigue cracking of the webs to occur.

A second analysis using the same basic model was performed for the patch plated condition to see how the stresses and distortions were affected. The model was modified by increasing the thicknesses of the web elements to reflect the patch plates on either side of the web. Also, the thicknesses of the web gap elements were increased from 0.375 in. to 1.56 in. to simulate the filling in of the gap with weld metal. Displacements from the first level substructure analysis of the patch plate condition were used to load the model.

Figure 6.31 shows a sketch of the elements in the web gap with the east face transverse stress at the center of each element. The large bending stresses which occurred in the original web gap were reduced
significantly making their magnitude consistent with the stresses in the surrounding elements. A plot of the rotations of the row of nodes along the bottom flange and in the gap for the two conditions is given as Fig. 6.32. The large change in rotations which occurred in the gap with the original condition are no longer present with the addition of the patch plates and filling in of the gap.

Thus, the analysis of the filled-in gap showed that by eliminating the gap, the stresses and distortions in the region were drastically reduced. In other words, had the web gap not been present, the original cracks most likely would not have developed.
7. TRAFFIC ANALYSIS AND PREDICTED STRESS CYCLES

7.1 Traffic Study

In order to assess accurately the severity of the fatigue cracking in the welded repairs, the stress history spectrum for each critical member must be determined. By reviewing the traffic that the structure has experienced and its relation to the stress levels in a particular member, an estimate can be made of the cumulative fatigue damage and the remaining life of the structure. The following explains, in detail, the procedure used to determine the load history for Bridge 651 at Hannibal, Missouri.

During the course of the traffic study it became apparent that available data was limited. This is partly due to the fact that the Wabash Railroad, the original operator of the bridge, was absorbed by the Norfolk and Western Railway in 1963. This made the process of obtaining early data on train movements and traffic density difficult.

The data made available by the Norfolk and Western Railway includes the following:

- Wabash Railroad Train Timetables from 1936 to 1963
- Annual gross tonnage figures from 1971 to 1981
- Listing of all train movements during 1982
- Detailed information on trains between October 31 and November 5, 1982

Because the available data was so meager, it was desirable to supplement it with other sources of data. It was decided to correlate the
available data with known information from the Canadian National Railroad, Kingston and Newmarket subdivisions (10,11). The Canadian National has done extensive traffic studies on many of their lines which has allowed them to develop useful averages and to indicate particular trends. Also, the Kingston and Newmarket carried traffic that was similar to the traffic crossing Bridge 651. Specifically, the traffic is comprised primarily of freight, and the lines are major east to west lines.

Given a limited amount of traffic data from which to work from, there are several important parameters which can be used to generate the necessary information. These are: the number of trains per day, locomotive weights, engine units per train, annual gross tonnage, and car-load distribution. Although not all parameters are known for each year under study, not all are needed for any one particular year. The following procedure was used to supplement the Norfolk and Western data for each parameter.

Trains per Day

Between 1936 and 1964 the number of trains per day was determined from the Wabash Railroad timetables. This is summarized in Fig. 7.1 and shows the average number of freight and passenger trains that crossed the bridge each day. Passenger service declined through the 1950's and was terminated in 1959. The use of timetables is extremely useful in the early years, because the locomotive is the primary unit that contributes to fatigue damage.
Locomotive Weights

Figure 7.2 summarizes the root-mean-square (RMS) values for the locomotive weight for each year. It is important to determine years in which steam locomotives were phased out and replaced by diesel locomotives, since the steam locomotives, enginer and tender, were significantly heavier than the diesel locomotives. Also, diesel units allowed for the combining of engines in order to increase horsepower and load-pulling capacity, increasing the number of locomotive loading cycles per train.

The RMS value for the steam locomotives was determined from a listing of retired locomotives, dated 1947, for the Wabash and Ann Arbor Railroad Companies. It was assumed that this distribution is similar to that which crossed the bridge between 1937 and 1947. All locomotives in the listing were built prior to 1937.

The frequency of use for diesel locomotives was determined from a listing of locomotive equipment, dated 1953, for the entire Wabash Railroad Company. All steam locomotives had been phased out for both freight and passenger service by this time. Again, it was assumed that the distribution of locomotive types for the entire system reflected the frequency of locomotive types on the bridge. The RMS value from this data is 120 tons per unit and was assumed to have remained constant to 1967. The 1982 traffic analysis resulted in an RMS value of 175 tons per unit. This reflects the increased use of heavier six-axle engines. It was assumed that the six-axle units were phased in starting in 1968.
and reached their current level in 1973. Also plotted in Fig. 7.2 is the data from the Canadian National Railroad which gives relatively good agreement with what has been used in this traffic study for the Norfolk and Western. It also indicates that the Canadian National lagged behind the Norfolk and Western in phasing out steam locomotives. The Kingston Subdivision used steam up until 1959.

Units per Train
Since steam locomotives were rarely paired, it was assumed that there was only one unit per train through 1947. With the introduction of diesel locomotives, engines could easily be combined which permitted increasing the number of units per train. The 1953 equipment listing gives an average value of 1.7 units per train. This takes into account the fact that some diesel locomotives (such as Class D45) actually consisted of three separate units, causing three individual load cycles in certain members. It was assumed that the yearly value increased steadily until it reached its present day value of 3.7 in 1972. The assumption shows good correlation with Canadian National data when the lag in steam phase-out is considered, as shown in Fig. 7.3.

Annual Gross Tonnage
The gross tonnage for each year is probably the most easily obtained set of information, though for this particular study only the figures for 1971 to 1981 were available. The 1982 data analysis
gave a projected value of 11.55 MGT, excluding the fact that the bridge was out of service for three months. A value of 10.0 MGT for 1962 was obtained from a System Map, dated October 9, 1963. Values were linearly interpolated between 1962 and 1971, assuming constant growth in traffic, as shown in Fig. 7.4. It was assumed that from 1955 to 1962 the annual gross tonnage remained constant at 10.0 MGT.

Car Load Distribution

No direct data was available for the car load distribution of the Norfolk and Western. Information used in the analysis came from the Kingston Subdivision of the Canadian National. This data is shown graphically in Fig. 7.5. It has been adjusted to reflect the differences in steam locomotive phase-out, assuming that this affected car load distributions. Since it is only the heavier cars that cause significant fatigue damage, only the cars whose gross weight over sixty tons was considered.

For Bridge 651, the period from 1937, when welded repairs were first used on the bridge, to the present was examined in order to assess cumulative damage to date. Traffic was extrapolated to the year 2000 in order to evaluate any further damage from continued use. For the fatigue assessment, this period has been divided into four different time periods depending upon the available data and known traffic characteristics. The four periods selected were: 1937 to 1954, 1955 to 1961, 1962 to 1982, and 1983 to 2000.
For the first period (1937-1954) an assumption has been made that only
the locomotives contribute to fatigue damage. During this time the average
locomotive weight ranged from 130 tons to 245 tons, while few car loads
exceeded 60 tons. Studies by the Canadian National concluded that only
cars over 60 tons have a significant effect on the fatigue life and all
other lighter traffic can be ignored\(^{(12)}\). Therefore, the yearly number
of cycles is the number of trains times the number of locomotive units per
train. Influence line analysis shows that for steam locomotives only one
predominate load cycle occurs in each member when excluding car loads.
Although the trailing tender causes a slight rise in the stress history
curve, its magnitude is insignificant.

The second time period examined was from 1955 to 1961. It is at the
beginning of this period that the car loads began to increase and contrib­
ute to fatigue damage. The number of cycles per year due to car loads was
determined by multiplying the annual gross tonnage by the estimated per­
centage of car loads over 60 tons divided by the average value for car
weights. The number of locomotive loadings was determined from the
available timetables. The number of trains per year was multiplied by the
average number of units per train.

For the third period, 1962 to 1982, the annual gross tonnage was the
main parameter used to determine the number of loadings. Car loading
cycles were determined by the method used for the 1955 to 1961 period:
the annual gross tonnage times the percentage of cars over 60 tons divided
by the average value for car weights. The number of locomotive units was
determined by calculating the average gross tonnage pulled by one unit.
In 1962 each unit pulled an average of 1700 tons. It was assumed that this increased linearly to the 1800 tons per unit that was used in 1982. The yearly number of unit loadings was then determined by dividing the annual gross tonnage by the average load per unit.

In order to extrapolate between 1983 and 2000, it was assumed the applicable parameters remain constant at their 1980 values. As shown in Fig. 7.4, the annual gross tonnage that moved across the bridge has steadily decreased since 1972. By using 1980 figures, the detrimental effects of the 1981-1982 recession were removed. The parameter values selected for this phase were taken as:

- Locomotive weight (RMS) = 175 tons
- Units per train = 3.7
- Annual gross tonnage = 15.6 MGT
- Car load distribution = 95 tons @ 53%

7.2 Stress Cycle Assessment

The traffic study resulted in the estimated number of locomotives and cars for each year and their respective RMS weight values (Table 7.1). Having an estimate of the type and distribution of the traffic that crossed the structure, the stress history spectrum for each critical member can be determined. Members from two spans, Span D (Spans A, B, and C) and Span F were examined in detail. These two spans contained details that were the most severe, as well as, typical for the bridge. The following explains the procedure used in the stress cycle assessment.
Influence lines for critical members were constructed from the global finite element analysis of both spans (see Figs. 7.6 and 7.7). For each span, the three most critical members were analyzed: a bottom chord, a hanger, and a diagonal. These were wrought iron members, and all of them incorporated the welded lap splice repair except the hanger in Span D.

A typical wheel spacing configuration was assumed for a steam and diesel locomotive and a typical car (Fig. 7.8). Each was given a unit weight and passed across the influence diagrams to determine the stress range. Then, using the estimated static values for the locomotive and car weights, the effective stress range could be determined for each year. Using Miner's linear fatigue damage relationship, the total fatigue damage could be determined for each member by summing the number of cycles at a particular stress range.

A comparison of the influence diagrams and strain gage traces showed that different members experienced stress cycles at different periods (cars per stress cycles). In general, the bridge members can be categorized into two different types. Type I members experience one stress cycle per car or locomotive unit. This was found to include hangers, floor beams, and stringers. These members are generally unaffected by loadings beyond adjacent panels of the truss. Type II members are those in which the stress cycle is caused by a group of cars. This was found to include diagonals, chord members, and bracing.

For Type II members the stress cycle is affected by the dependence of the influence diagram on load position, the span length, and the sequence of loaded and unloaded cars. As the length of the influence diagram
approached the span length, car loads are combined and their individual influence attenuated. This becomes more pronounced with an increase in span length. The distribution of loaded and empty cars is the principle cause of cyclic stress. Each individual car causes a relatively small cycle that is superimposed on the larger cycle. An alternating series of loaded and unloaded cars will result in an increased number of stress cycles. Though they result in higher member stresses, unit trains produce relatively few large stress cycles in Type II members, since there is no unloading of the member.

Therefore, the total stress cycles depend upon the member type. For both Span D and Span F, the hanger members were observed to experience a stress cycle for each group of wheel trucks which correspond to each car. The diagonal and bottom chord members were found to behave in a similar manner and were observed to average five cars per cycle. The diagonal members exhibited slightly more fluctuations due to the negative (compressive) portion of the influence diagram.
8. FATIGUE RESISTANCE OF WROUGHT IRON SPLICES

The fatigue characteristics of wrought iron were required, since it was the principle structural material used in Bridge 651. Though the material properties of wrought iron are well known, the fatigue behavior of welded wrought iron details was never quantified. A test program was developed in order to determine the fatigue resistance of the welded wrought iron splice repairs. This was necessary in order to establish the fatigue strength of the welded details and permit an assessment of the cumulative fatigue damage that may have occurred.

8.1 Repair Details

Common to old pin-ended truss bridges, many of the eyebar members of Norfolk and Western Bridge 651 loosened with time and required tightening. The corrective repair procedure used on this bridge was to cut the eyebar body and weld steel lap shear splices over the cut after the member was tightened. A small length was cut out of the eyebar body, and the two cut ends drawn together, so as to retension the member component. Splice plates were welded on each side of the wrought iron bar, as illustrated in Fig. 8.1. The steel lap plates were fabricated from A7 material. A continuous fillet weld was placed around the entire perimeter of each lap plate which resulted in both transverse and longitudinal welds, the latter bridging the exposed gap between the two wrought iron plate ends. Weldment was also used to fill the opening at each edge of the gap. This welded splice was used on many of the diagonals, bottom chord members, and
on six of the eight eyebar hangers in Span F, as well as eyebar members in the other seven spans. For the hangers in Span F, the original cross-sectional area of the wrought iron eyebar was 4.0 in\(^2\). The two steel lap splice plates provided a total area of 4.5 in\(^2\) which decreases the stress by 12 percent. This type of tightening procedure was a common method of repair before the fatigue strength of weldments became well known. The AREA specifications now provide procedures for eyebar tightening by heating and upsetting of the eyebar body.

8.2 Physical Properties

Wrought iron is a two-component metal consisting of high purity iron and iron silicate, a particular type of glass-like slag. Originally, the slag content of wrought iron (2.5% by weight) was considered as an undesirable impurity. But it is the slag which is now recognized as being responsible for the desirable properties of wrought iron; its resistance to corrosion and to fatigue.

Nineteenth century furnace temperatures were not high enough to keep the refined iron from solidifying and trapping some of the molten slag during the final stages of the refining operation. Bessemer's development of steel making was originally intended to produce wrought iron of higher quality and lower cost than what was currently available. Slag is distributed throughout the iron base metal generally in the form of threads of fibers which extend in the direction of rolling. Approximately 250,000 of the siliceous fibers are present in each cross-sectional square inch of good quality wrought iron. Corrosion resistance is attributable to the
purity of the iron base, freedom from segregated impurities, and most of all, to the presence of the slag fibers distributed throughout the base metal. The slag fibers are present in such great numbers that they serve in one capacity as an effective barrier to the process of corrosion, forcing it to spread over the surface of the metal, rather than to pit or penetrate (13).

Tensile tests of wrought iron bars taken from the structure gave a minimum yield stress of 26 ksi at 0.2 percent offset. The tensile strength was measured as 45 ksi. Test results are summarized in Table 8.1, and the recorded load-strain plots of the test specimens are shown in Figs. 8.2 and 8.3. These results agree with ASTM Specifications for A42-13 wrought iron plates.

8.3 Fatigue Tests

Laboratory fatigue tests of the hanger lap splice were conducted in two parts in order to accurately assess the severity of the transverse weld cracks. Initially, specimens were fabricated and tested using pieces of wrought iron bars that were salvaged from Span E. Span E was replaced when the original span was knocked into the river during a barge collision. Later, as a result of the replacement of the cracked hangers, the actual welded joints became available from Span F. These were also fatigue tested, examined, and compared to the fabricated specimens. Figure 8.4 shows a Span F welded lap splice in the alternating stress testing machine.
8.3.1 Testing of Fabricated Specimens

A total of seven tests were conducted using three different constant amplitude stress ranges and two variations of the weld orientation. The first three test specimens were run at a stress range of 12 ksi. Although there was noticeable cracking along the toe of the transverse weld, failure of these three specimens did not occur at this location. The first two specimens incorporated the center junction which was subjected to a stress range of 10 ksi. This resulted in failure of the specimen at this location after 2.1 million cycles for Specimen No. 1 and 3.6 million cycles for Specimen No. 2. The cracks initiated in the longitudinal weld that bridged the gap between the wrought iron plates and propagated into the steel plates. These failures correspond to the fatigue life for ordinary steel and plot near the Category D fatigue strength curve, as can be seen in Fig. 8.5. However, the test sample is so small that assessing the resistance using Category E is a more reasonable lower bound. Clearly, it was this location that controlled the fatigue life of the lap splice connection.

After failure of the steel plates, both Specimens No. 1 and No. 2 were regripped to permit continued testing of the transverse fillet welds which provide a cover plate-like detail. Both specimens were subjected to 20 million cycles without failure at this location, although noticeable cracking at the weld toe and in the weld occurred. Specimen No. 3, incorporating only the end detail, was cycled to six million without failure and then was destructively examined to evaluate the extent of cracking.
Since crack growth at the weld toe on the wrought iron plate did not result in failure at 12 ksi, the stress range was increased to 18 ksi for the next three specimens. At the higher stress range level, failure occurred at the toe of the transverse weld beginning at approximately 0.74 million cycles. This was still well above the fatigue strength expected at Category E detail. All fatigue test results applicable to the wrought iron members are plotted in Fig. 8.6. Those tests which did not fail are indicated with an arrow.

Since the properties of wrought iron are not isotropic, but directionally dependent, a seventh test specimen was fabricated with the steel plates welded to the edges of the wrought iron bar. This specimen was tested at a stress range of 15 ksi, and the fatigue crack propagated perpendicular to the edges of the elongated stringers and did not result in crack arrest. The test was run at a constant stress range of 15 ksi and failed after 0.46 million cycles. This failure life plots within the scatter band of a Category E detail and is shown by the cross in Fig. 8.6.

In order to evaluate the fracture toughness of the toe crack, the temperature was lowered on two specimens during fatigue testing. Specimens No. 1 and No. 4, cycled at stress ranges of 12 ksi and 18 ksi respectively, did not fail by fracture as a result of the reduction in temperature to -40° F. The test results indicate that the welded wrought iron had a relatively high resistance to crack instability at reduced temperatures. It was concluded that the toe-cracked eyebars would not fail due to brittle fracture.
8.3.2 Testing of Span F Hangers

Six of the eight hangers in Span F which contained the lap splice repair were removed from the structure. The six lap splice joints were cut from the members and shipped to the laboratory for a detailed examination. Several were fatigue tested, and the remainder cut open to reveal the extent of the cracking.

The actual bridge repair lap splices were found to have lower quality welding and workmanship when compared to the laboratory fabricated specimens. These repairs were conducted in 1937, when the art of welding had not been adequately developed and were made under field conditions. The majority of the transverse welds were found to be undersized for the given plate thickness. The weld sizes varied between 3/16 and 1/4 in. for the 7/8 in. plate. Current design specifications require a minimum weld size of 5/16 in. In addition, the welds exhibited porosity and undercutting of the wrought iron plate. Many of the weld profiles had a contact angle that greatly exceeded 45 degrees.

One full size joint was placed in the alternating stress machine and fatigue tested at a stress range of 12 ksi in the eyebar. As expected, failure occurred with a fatigue crack originating at the center gap, where the stress range was 10.7 ksi and propagated into the steel plates. Final failure occurred after 1.9 million cycles, and the results are plotted in Fig. 8.5. This obviously is a lower bound estimate of the life, as it did not consider the stress cycles experienced by the detail in the bridge. Each of the two resulting halves of the original joint were then tested separately in order to evaluate the fatigue resistance of the visible
transverse weld toe cracks. Although each piece experienced toe cracks in
the wrought iron and root cracking of the welds, the tests were carried to
10.0 million cycles without any evidence of failure or inability to con­
tinue to resist the cyclic loads. These tests are also plotted in
Fig. 8.6 with the arrows indicating they had not failed. Two additi­
onal joints were tested to determine the fatigue strength of the steel splices.
These tests are plotted in Fig. 8.5 as the solid squares. They provided
slightly less fatigue resistance than the fabricated specimens, as a
result of their prior cyclic loading in the structure.

The remaining lap splice joints were cut open to expose cracks that
might have occurred in the welds during service. For most joints, cracks
were found on only one side of the joint. This was generally observed at
the weld toe with the greater reinforcement angle. Also, two cracks were
found towards the center portion of the transverse weld and root cracks at
the outer ends of the weld. No cracking was found in the longitudinal
welds except at the center gap location. Here, the crack had propagated
through the entire weld, but in no case was the crack observed to enter the
steel lap plates. Figure 8.7 shows an exposed surface at the member gap.
The two lap plates were partially saw cut before pulling the member apart
at a reduced temperature. No significant crack extension can be seen in
the splice plates. Cracking was visible in the welds at each edge only.

8.4 Examination of Results

The mechanism of crack formation and propagation in the welded joints
is as follows. Fatigue cracks were found to begin at the toe of the
transverse weld, in the center portion of the wrought iron plate. Visible cracks were first noticed at approximately 0.5 million cycles at a stress range of 12 ksi. Initially, the toe cracks propagated perpendicular to the thickness of the wrought iron plate until they encountered a significant slag stringer. The crack may reinitiate out of the stringer at a location that is further into the joint. The weld toe cracks continue to encounter stringers, and this process continues along the weld toe to a depth of approximately 3/16 to 1/8 in. at an approximate angle of 45 degrees. At this point, the crack is arrested as the crack turns and extends vertical into the joint parallel to the stress. This angle of growth corresponds to the plane perpendicular to the stress field through the fillet weld. The crack growth results in a staircase effect, as can be seen in Figs. 8.8 and 8.9.

The importance of the stringers in the wrought iron appears to be their ability to arrest the transverse crack and deflect the growth of the crack into the joint, down under the fillet weld. A similar joint geometry in steel would result in the toe crack propagating through the thickness of the main plate. In the welded wrought iron, the crack is arrested and turned parallel to the plate and cyclic stress. There is an obvious redistribution of stress in the vicinity of the crack front. As the crack moves into the joint following the laminated stringers, the stress is redistributed to the longitudinal welds. The redistribution of the stress to the outer portions of the joint resulted in root cracking in both of the longitudinal welds, and the outer portions of the transverse welds in the bridge members as the weld sizes were small. Eventually, nearly all the
load was transferred through the longitudinal welds which resulted in continued crack extension along the weld throat.

Fatigue testing at a high stress range of 18 ksi resulted in a maximum stress of 21 ksi. This maximum stress corresponds to approximately 80 percent of the measured yield stress of the wrought iron. As the fatigue crack propagated into the joint, the cross-section of the wrought iron plate was reduced. This resulted in yielding of the net section of the wrought iron plate and resulted in continued crack extension with the eventual failure of the section. Such high stress ranges would not be expected in a wrought iron structure.

The test results from the welded wrought iron specimens and the actual hangers are summarized in Figs. 8.5 and 8.6. These tests demonstrate that the fatigue resistance at the weld toe cracks is much greater than that provided by Category E, the expected fatigue category for such welded steel details. None of the details tested at a stress range of 12 ksi failed as a result of the crack that formed at the weld terminations. These cracks were all arrested and did not impair the load-carrying capability of the joint. Failure, should it ever occur, was shown to develop at the gap region.

8.5 Summary and Conclusions

As a result of the fatigue tests of welded wrought iron with welded lap splice repairs, the following conclusions were reached:
1. Surface cracks were found to develop at the toe of the splice plates that were welded to the surface of the wrought iron eyebars at cycle lives comparable to welded steel components. These cracks were found to have no adverse effect on the fatigue resistance of the wrought iron members, as they were arrested by the wrought iron slag stringers.

2. Laboratory fatigue tests were carried out on simulated welded joints and on several of the cracked eyebar hanger splices. The simulated test joints demonstrated that the wrought iron would arrest the fatigue cracks that formed at the weld toe at stress levels expected in the structure. The weld toe cracks encountered the flattened longitudinal stringers and were arrested as the crack deflected parallel to the cyclic stress. The crack arrested details were able to sustain 20 million stress cycles at 12 ksi without any further evidence of distress. The cracked hangers were also fatigue tested and yielded comparable results.

3. The laboratory tests also demonstrated that the steel splice plates were the more critical detail. The four tests carried out provided a fatigue resistance comparable to Category D, although they are classed as Category E details. Since the stress range at the critical steel section was less than the wrought iron bar, the hangers had not experienced much growth at those sections.
4. A pilot test on the edge welded wrought iron splice demonstrated that the stringers were not effective in arresting fatigue crack growth from one edge of the plate towards the other edge. The test result was comparable to a welded steel detail.
9. **EVALUATION OF CUMULATIVE DAMAGE IN WELDED AND RIVETED WROUGHT IRON MEMBERS**

With the stress history spectrum for each critical member determined from the traffic study and the stress cycle assessment, the results were compared to the fatigue resistance of the welded wrought iron and lap splice repair detail. Comparisons are made between the predicted effective stress range, based on Miner's hypothesis, and accumulated cycles and the AREA fatigue categories that apply to this particular detail. The test results acquired in the laboratory were used to establish the fatigue resistance applicable to the joint.

The measured stress range histograms for the highest stressed bottom chord, diagonal and hanger members in Spans D and F are summarized in Figs. 9.1 to 9.6. Also shown in Figs. 9.1 to 9.6 are the effective stress range for the measured spectrum and the predicted effective stress range for the traffic using the structure during 1982. Except for diagonal member L4-U5 in Span D, the effective stress range for the year as a whole was slightly less than the effective stress range for the trains using the structure during the test period. The histograms all indicate the stress in the member applicable to the weld toe or to a rivet hole.

The following summarize the analysis made for the selected critical members of Span D and Span F.
9.1 **Span D**

For the bottom chord L4-L5 the effective stress range was estimated to be 4.1 ksi for the 0.68 million stress cycles estimated to occur between 1937 and 1982. When projected to the year 2000, the effective stress range increases to 4.2 ksi for 1.1 million cycles (see Table 9.1). As was expected from the traffic using the structure, the effective stress ranges for traffic since 1937 is less than the values for 1982 that are indicated in Figs. 9.1 to 9.6. The higher loads being carried in 1982 result in an increase in the effective stress range. This was also the reason for the increase noted between 1982 and the year 2000. The diagonal member L4-U5 was estimated to have an effective stress range of 6.0 ksi for 0.68 million stress cycles up to 1982 and 6.2 ksi for 1.1 million cycles by the year 2000. The effective stress range at the splice plate is 5.3 ksi up to 1982 and 5.5 ksi for the year 2000. Figure 9.7 shows that the estimated stress-cycle data for both the bottom chord and diagonal member plot well below the Category E curve. Hence, no significant fatigue crack growth is expected to occur in any of the diagonal and bottom chord members until well into the next century at either the weld toe or at the splice plate connections.

The critical hangers in Span D are built-up members which do not contain the welded lap splice detail. The effective stress range was estimated to be 7.6 ksi for 3.1 million cycles in 1982 and to increase to 8.1 ksi for 5.2 million cycles in the year 2000. These points are also plotted in Fig. 9.7 as the solid dots. It is apparent that the hanger members L1-U1 (L7-U7) are approaching the lower bound resistance for
Category D. This resistance curve is applicable to riveted built-up wrought iron members\(^\text{(14)}\). The riveted connections appear to be more severe than the welded attachments to the hanger surface. The projected cyclic loading to the year 2000 will likely result in fatigue cracking in these members. The effective stress range of other tension members in Spans A, B, C, and D are summarized in Table 9.2. This summary confirms that only the riveted hangers in these spans have relatively high effective stress range and are sensitive to fatigue cracking.

9.2 Span F

In general, the Span F member stresses were found to be relatively low as a result of the 1912 span shortening. Many members were designed for higher live load stress levels than what are now being realized. The effective stress range in the bottom chord L1-L2 was estimated as 1.3 ksi for 0.68 million cycles in 1982. No significant change in stress range would result for 1.1 million cycles in 2000. Since the effective stress range at the splice detail is even smaller (see Table 9.3), no crack growth should develop in the bottom chord members of Span F. All stress cycles appear to be below the fatigue limit. The diagonal member L2-U1 gave higher effective stress ranges equal to 3.3 ksi in 1982 and 3.4 ksi for the year 2000. These estimated effective stress range values are plotted in Fig. 9.8 to compare with strength of fatigue categories. They indicate that neither member will experience significant crack growth at the welded splice plates. Other chord and diagonal members in Span F have even lower effective stress range values, as can be seen in Table 9.3.
For the Span F hangers, a problem of determining the correct effective stress range corresponding to all damage cycles arose because of the slackness in some of the eyebars. The actual accumulated fatigue damage estimate for a given loaded member is bounded by the condition of both bars carrying the load equally and the condition that only one bar carries the entire load for the member. Assuming that both eyebars were loaded resulted in an effective stress range at the weld toe equal to 4.3 ksi and 3.1 million variable load cycles. The effective stress range increases to 4.5 ksi for the 5.2 million cycles projected to the year 2000. The corresponding values in the splice plate are 3.8 and 4.0 ksi respectively. With only one bar participating, the effective stress range at the weld toe increases to 9.8 ksi for 1982 and to 10.3 ksi for the year 2000. Both of these estimates are plotted in Fig. 9.8. The cracking observed in the loaded bars of hangers U1-M1 and U8-M8 suggest that a single bar was carrying the load for a significant period of time. The degree of fatigue crack growth observed at the weld toe suggests that the effective stress range was about 7 ksi. This corresponds to the average value of the two estimates. The corresponding value in the splice plate is 6.5 ksi and is compatible with the amount of fatigue crack growth observed at those locations.

9.3 Reinforcement Members

A number of lower chords and diagonals throughout the structure have been reinforced by the addition of additional members, as was illustrated in Figs. 1.7, 2.6, 3.2, and 3.3. Generally this reinforcement was accomplished by adding a U-shaped strap around each pin and then welding additional bars to it.
Many of these reinforcement members were found to be loose and carrying little if any dead and live load. In view of the low effective stress range observed and predicted in the lower chord and diagonal members, the lack of load sharing does not appear to be a serious problem.

A number of these reinforcements have also exhibited cracking when they did in fact share the load. This cracking has developed in the U-shaped strap at two locations which can be seen in Fig. 9.9. Failure has obviously occurred at the center of the U-shaped strap. An examination of the crack surface, shown in Fig. 9.10, indicates that the fatigue crack initiated at outside flame out edge of the strap directly opposite the bearing surface of the strap on the pin. The net width of the strap at that location is only equal to half the net section of the strap. This condition does not conform to the AREA provisions for pin connected plates. Assuming a simple force distribution model indicates that the stress at the edge of the plate is many times greater than the stress on the net section.

Cracks were also observed at the net section in the sharp reentrant corner that can be seen to exist at that location in Fig. 9.9. A closer view of the crack is given in Fig. 9.11. The degree of cracking at this location when compared to the failure location verifies the magnitude of the bending stress acting on the outside edge of the plate.
9.4 Summary and Conclusions

An evaluation of the load history to which the members were subjected indicated that the welded lap splice repairs on the chord and diagonal members were not susceptible to any appreciable crack growth and would not be fatigue critical. Except for the hangers, the estimated effective stress range and accumulated cycles of variable loading for selected members of Span D and Span F plotted well below the fatigue resistance provided by wrought iron welded details.

The assessment of the accumulated fatigue damage in the hanger members of Span F verified that significant crack growth would be expected because of the high stress range in a single active bar. These members were replaced, so they are no longer critical.

The estimated damage in hangers U1-L1 and U7-L7 in Spans A, B, C, and D indicates that fatigue cracking is likely to develop at the rivet holes between 1982 and 2000.
10. FATIGUE CRACKING IN THE FLOOR BEAMS

10.1 Welded Reinforcement Along Patch Plates

As was indicated in Chapter 6, (Fig. 6.29) high out-of-plane web bending stresses were estimated to occur in the web gap between the flange and end connection angles. In order to resist the distortion and reinforce the cracked web plate, the existing patch plates were added and welded to the edges of the angles, as illustrated in Figs. 2.17 to 2.27. During the field inspection of this study, cracks were observed in and along the end connection angles of the floor beam and in the welded reinforcement along the triangular patch plates.

It was visually apparent that the quality of these weld repairs was poor by the present standard. Undercut, lack-of-fusion and other defect conditions were observed. The quality of the weldments and geometry of the reinforcement suggested that the fatigue strength at the connection angle was less than Category E'.

Stress measurements on Floor Beam 7 of Span D indicated that the stress range at the weld termination on the connection angle was fairly high (see Table 6.1). The estimated plate bending stress in the web gap with reinforcement was of comparable magnitude (see Fig. 6.31). The estimated effective stress range at the weld toe was about 2.8 ksi, with a corresponding number of variable amplitude stress cycles about $3.1 \times 10^6$ since the addition of the patch plates in 1943. The fatigue strength was less than that of Category E'.
Since the connection angles and patch plates were both made of steel, no beneficial effect of crack arrest by iron silicate could be expected as was observed in the wrought iron bars. The cracking observed in the floor beam along the weld reinforcement at the triangular patch plates would continue to develop if retrofitting is not undertaken.

10.2 Retrofitting the Floor Beam Web at End Connections

The patch plates welded to the web and angles of the floor beam improved the undesirable web gap condition, as was demonstrated in Chapter 6. However, the patch plate welds have resulted in cracks along the connection and bottom flange angle welds and at the weld end on the connection angles. These crack conditions were shown to result from the weak axis bending and distortion at the floor beam end connections.

In order to correct this condition and minimize future cracking in the connection angles, an analysis was carried out using the model described in Section 6.6 on a model with bolted splice plate on each side of the floor beam web, as shown schematically in Fig. 10.1. These plates were bolted to the connection angles and the bottom flange angles as well as the welded patch plates. This effectively bridges these poor quality welds between the patch plates and the angles and reduces the cyclic stresses.

Figures 10.2 and 10.3 show the resulting horizontal and vertical surface stresses on the 3/8 in. splice plates along the edge of the connection angle. It is clear that the cyclic stress for the maximum load is well below the fatigue resistance of the bolted plate. Such
retrofitting by splice plates will reduce the cyclic stress in the floor beam as well as bridge across the existing cracked weld reinforcement along the end connection and flange angles. It should be noted that the crack growth observed in the welded connections between the patch plates and the flange and connection angles results from the large lack of fusion along those weld lines. By connecting the angles to the patch plates with the bolted splice plates, modest reductions in the stress range normal to the lack of fusion plane are brought about. Of greater importance is the ability to splice over those defect conditions and transfer the floor beam load with the bolted plates. Any subsequent crack growth in the existing welded reinforcement is not significant to the performance and resistance of the member.
11. RETROFIT AND INSPECTION RECOMMENDATIONS

11.1 Welded Lap Splices

1. The cracked hangers in Span F (M1-U1 and M8-U8) were recommended for replacement in November 1982. This was carried out in early 1983. The crack conditions in the replaced splices were examined and observed to be comparable to the laboratory simulations of the welded splice.

2. No further retrofit work needs to be carried out on any of the remaining splice-plated members in Bridge 651. The laboratory test on simulated splices and the hangers removed from Span F demonstrated that weld toe cracks in the wrought iron members were not significant. The presence or development of toe crack did not impair the fatigue resistance of the member. Cracking at the steel splice plates was found to be more serious than at the weld toe. However, the effective stress range at all of these details was sufficiently low, so that no significant fatigue damage has developed nor is it expected in the future.

3. Only a cursory annual inspection needs to be made at the weld spliced details, as any significant cracking will reveal itself first at the weld toe. Such cracks will be readily apparent from the oxides that form. Such cracking is not detrimental to the serviceability of the member and structure.
11.2 Riveted Wrought Iron Hangers

1. The riveted wrought iron hangers at L1-U1 and L7-U7 of Spans A, B, C, and D were found to have accumulated some fatigue damage. They appear to be approaching the fatigue resistance provided by Category D which is applicable to these members. No cracks were actually detected or observed in these members to date. However, by the year 2000, cracking may become apparent in one of the hanger channel sections. Since two channels are provided at each member, there is redundancy and the development of a crack will not endanger the structure.

2. It is recommended that the four hangers in each of the spans A, B, C, and D, be monitored at yearly intervals with particular attention to the rivet holes on the inside face of the member near the floor beam and at the upper gusset connection.

11.3 Bottom Laterals in Spans C, F, and G

1. The cracks which have formed in the flame-cut notches of the inverted tee members should be arrested by drilling holes at the crack tips. Splice plates can be bolted to the flanges of the notched section.

2. The lateral connection plates for the bottom laterals at panel points L0 and L8 need attention. If the crack at the reentrant corner is less than 1 in. long, a 3/4 in. hole can be drilled at the crack tip, and this condition monitored
during regular inspections. If the crack length exceeds 1 in.,
the cracked plate should be replaced with a new plate that has
a ground radius of 2 in. for the reentrant corner. This plate
can be welded to the end post and bolted to the lateral, as is
presently done.

11.4 Floor Beam-Hanger-Lateral Connections

1. No farther floor beam end connection angles need to be replaced
even if cracks are observed at the weld reinforcement. Holes
should be drilled at the crack tips.

2. Triangular splice plates should be installed over the existing
welded triangular patch plates and the flange and connection
angles, as illustrated in Fig. 10.1. High strength bolts
should be used to attach the plates to the floor beam in place
of the rivets. Additional bolts should be installed in the
splice and welded plates.

3. At the notched out end of the bottom flange angle where cracks
have developed in the outstanding angle leg, the rivet in the
flange to lateral connection plate should be replaced with a
high strength preloaded bolt. We do not think this condition
warrants replacement or more elaborate corrective action.

11.5 Handrails of Truss Members

The handrail attachments which are welded to tension hanger or
diagonals should be removed, and the weld area ground smooth. The handrail
attachment can be bolted to the member using the lacing holes.
12. SUMMARY AND CONCLUSIONS

The results and conclusions of this study are summarized below.

1. A field inspection of the spans in the bridge revealed cracks in numerous truss members containing eyebars with welded lap splices and slot welds. Cracks were also found in the patch plate welds, connection angles and filled-in beveled gaps of the floor beams. In addition, cracks were also found in the coped floor beam bottom flanges, end post lateral gussets and in notched stems of numerous bottom laterals.

2. Field measurements showed that train direction and speed had little measureable impact effects on member stresses and responses. Gages placed in the vertical gaps between the doubler plate and stringer connection angle on Floor Beam 7 of Span G revealed low longitudinal stresses which indicated the possibility of cracking to be very low. Load distribution in the gaged truss members was not equal among the bars in the counters and diagonals.

3. A three dimensional analysis of Span D provided information on forces and stresses which compared quite well to measured values.

4. The out-of-plane bending and twisting of the floor beams and bending of the hangers in the plane of the trusses was attributed to the stringers and bridge support conditions. The stringers restrained the middle portions of the floor beams.
from displacing longitudinally as much as the lower chord members. This relative movement was caused by the difference in stiffness between the trusses and floor system.

5. The attachment of the bottom laterals to the bottom flanges of the floor beams resulted in large horizontal bending stresses to develop in the bottom flanges. Computer analysis showed that framing the lateral bracing directly into the panel points significantly decreased the horizontal bending moment of the bottom flanges.

6. Disconnecting the bottom laterals from the stringers changed the stress distribution along the depths of the tee-shaped laterals causing the neutral axis to move toward the top flange of the tees. This suggested that attaching the laterals to the stringers contributed to cracking in the notched stems.

7. A finite element analysis of the floor beam hanger bottom lateral connection for the original condition revealed out-of-plane rotational distortion in the bottom corner of the floor beam which produced nominal web bending stresses of up to 3.5 ksi. A second level finite element analysis of the original web gap between the beveled legs of the bottom flange and connection angle predicted out-of-plane vertical web bending stresses which exceeded the nominal yield point of wrought iron. It was concluded that these high stresses caused cracks to develop in the original web gaps and propagate into the floor beam webs.
8. A finite element analysis of the floor beam–hanger–lateral connection including the patch plates revealed that out-of-plane rotational distortion was still present and the magnitude of the web bending stresses were low. A second level finite element analysis of the filled-in web region also indicated low stresses in the patch plate welds. The results showed that if the floor beams had been fabricated without the beveled gaps, the original cracks would probably never have developed.

9. Traffic analysis using available data of the bridge and information from the Canadian National Railroad provided effective stress ranges and corresponding number of cycles for the critical members of the spans. Hangers, floor beams and stringers experienced one cycle per car or locomotive unit, while diagonals, chord members and bracing members were found to undergo about one cycle for five cars.

10. Tensile tests of wrought iron bars gave a yield point of 26 ksi and tensile strength of 45 ksi. Fatigue testing of fabricated specimens, and actual lap splices in hangers revealed that surface cracks would develop at the toe of the splice plates but could not penetrate the wrought iron slag stringers at the level of stresses in the hangers. The steel splice plates had lower fatigue strength and were subjected to lower stresses.
11. Comparison was made between effective stress ranges — cycle members and fatigue strength of the welded and riveted wrought iron members. Except the hangers, all members have long remaining life. The hanger members of Span F have been replaced. Those hangers in Spans A, B, C and D were found likely to develop fatigue cracks between 1982 and the year 2000, but no significant fatigue damage is expected. Annual inspection is recommended.

12. The fatigue strength of the floor beam connection angle patch plates was estimated to be less than that of Category E. Cracks in this region would continue to develop if retrofitting is not undertaken. Retrofitting by bolting splice plates is recommended. Bolted splice plates over the existing welded patch plates on both sides of the floor beams will effectively bridge the existing poor quality welds.

13. Minor repairs to bottom laterals and lateral connection plates at end panel points in Spans C, F and G have been recommended.
### TABLE 3.1 SUMMARY OF GAGES ON TRUSS MEMBERS

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<th>Span</th>
<th>Gage Number</th>
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<td>F</td>
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<td>L7-U7N</td>
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TABLE 3.4 SUMMARY OF TEST TRAIN RUNS

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## Table 5.1 Longitudinal Displacements of Floor Beam Bottom Flange Nodes at the Span Centerline and at the Panel Points

<table>
<thead>
<tr>
<th>Floor Beam</th>
<th>@ Bridge Centerline (in.)</th>
<th>@ Panel Points (in.)</th>
<th>Relative Displacement (in.)</th>
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<td>0.046</td>
<td>0.022</td>
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<tr>
<td>2</td>
<td>0.033</td>
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<td>3</td>
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<td>0.146</td>
<td>0.368</td>
<td>0.222</td>
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-87-
TABLE 5.2  COMPARISON OF STRESSES AND MOMENTS FOR CASES 1, 2 AND 3

<table>
<thead>
<tr>
<th>Member</th>
<th>Case 1 Laterals Connected (ksi)</th>
<th>Case 2 Span G (ksi)</th>
<th>Case 3 LateralsDisconnected (ksi)</th>
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<td>4.68</td>
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Notes: Stresses in ksi
Moments in k-in.
TABLE 5.2 COMPARISON OF STRESSES AND MOMENTS FOR CASES 1, 2 AND 3

Continued

<table>
<thead>
<tr>
<th>Member</th>
<th>Case 1 Laterals Connected</th>
<th>Case 2 Span G</th>
<th>Case 3 Laterals Disconnected</th>
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<tr>
<td></td>
<td>1 2 3</td>
<td>1 2 3</td>
<td>1 2 3</td>
</tr>
<tr>
<td>Hangers</td>
<td>σ_p σ_bx σ_by</td>
<td>σ_p σ_bx σ_by</td>
<td>σ_p σ_bx σ_by</td>
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<tr>
<td>L1-U1</td>
<td>9.76 0.0 1.61</td>
<td>9.83 0.14 1.60</td>
<td>9.61 0.28 1.57</td>
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<tr>
<td>U4-L5</td>
<td>9.32 3.28 1.34</td>
<td>9.25 3.60 1.46</td>
<td>8.96 3.14 1.40</td>
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</table>

1Axial stress, ksi
2Bending Stress in plane of truss
3Bending stress in plane of floor beam

<table>
<thead>
<tr>
<th>Bottom Laterals</th>
<th>σ_p</th>
<th>M^4</th>
<th>σ_p</th>
<th>M^4</th>
<th>σ_p</th>
<th>M^4</th>
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<tbody>
<tr>
<td>L6N-L7S</td>
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<td>1.68</td>
<td>-36.70</td>
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<td>-57.70</td>
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<td>30.83</td>
<td>8.25</td>
<td>27.64</td>
<td>5.11</td>
<td>-88.05</td>
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4Peak Horizontal Bending Moment in floor beam bottom flanges k-in.


**TABLE 5.2 COMPARISONS OF STRESSES AND MOMENTS FOR CASES 1, 2 AND 3**

(continued)

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<thead>
<tr>
<th>Floor Beams</th>
<th>Case 1 Lateral Connected @ Stringer</th>
<th>@ Lateral</th>
<th>Case 2 Span G @ Stringer</th>
<th>@ Lateral</th>
<th>Case 3 Lateral Disconnected @ Stringer</th>
<th>@ Lateral</th>
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<td>0.78</td>
<td>0.31</td>
<td>0.96</td>
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<tr>
<td>2</td>
<td>0.96</td>
<td>2.01</td>
<td>0.86</td>
<td>0.32</td>
<td>1.44</td>
<td>3.49</td>
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<td>1.32</td>
<td>4.25</td>
<td>1.24</td>
<td>0.51</td>
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<td>4</td>
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<td>4.77</td>
<td>1.38</td>
<td>0.50</td>
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<td>5</td>
<td>1.65</td>
<td>6.00</td>
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<td>0.69</td>
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<td>9.01</td>
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<tr>
<td>6</td>
<td>2.47</td>
<td>9.55</td>
<td>2.59</td>
<td>1.04</td>
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<td>7</td>
<td>3.21</td>
<td>9.41</td>
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<td>1.36</td>
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TABLE 6.1  FIRST LEVEL SUBSTRUCTURE ANALYSIS RESULTS  
AND MEASURED STRESSES (ksi)

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<th>Measured</th>
<th>Theoretical</th>
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<td>62R</td>
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<td>66R</td>
<td>0.7</td>
<td>0.28</td>
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<td>62W</td>
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<td>64R</td>
<td>0.5</td>
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<td>64W</td>
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# TABLE 7.1 ESTIMATED TRAFFIC AND RMS WEIGHTS, 1937-2000

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<th>LOCOMOTIVES</th>
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<th>CARS</th>
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TABLE 8.1 RESULTS OF TENSILE TESTING OF WROUGHT IRON SPECIMENS

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<th>Diameter (in.)</th>
<th>Area (in²)</th>
<th>L₀ (in.)</th>
<th>Lₘ (in.)</th>
<th>&amp; Yield</th>
<th>Load Stress</th>
<th>Load Strength</th>
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TABLE 9.1 MEASURED AND ESTIMATED STRESS RANGES

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<th>Measured</th>
<th>1982 Traffic</th>
<th>$S_{re}$ (ksi) Estimated</th>
<th>Estimated Cycles $(10^6)$</th>
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<td>0.68</td>
</tr>
<tr>
<td>D</td>
<td>L4-U5</td>
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<td>5.1</td>
<td>6.0</td>
<td>6.2</td>
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<td>6.2</td>
<td>7.6</td>
<td>8.1</td>
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<td>9.8</td>
<td>10.3</td>
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*Both Eyebars Loaded*
**TABLE 9.2 EFFECTIVE STRESS RANGE IN MEMBERS**

OF SPANS A, B, C AND D

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<td>( S_{re \ @ \ \text{Splice}} )</td>
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<tr>
<td><strong>Bottom Chords</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>L0-L1 (L7-L8)</td>
<td>4.3 ksi</td>
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<tr>
<td>L1-L2 (L6-L7)</td>
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<td>*</td>
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<td>L2-L3 (L5-L6)</td>
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<td>*</td>
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<tr>
<td>L3-L4 (L4-L5)</td>
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<td>*</td>
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<tr>
<td><strong>Hangers</strong></td>
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<td></td>
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<tr>
<td>L1-U1 (L7-U7)</td>
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<td><strong>Diagonals</strong></td>
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<tr>
<td>L2-U1 (L6-U7)</td>
<td>6.2</td>
<td>*</td>
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<td>L3-U2 (L5-U6)</td>
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<td>6.8</td>
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<td>L4-U3 (L4-U5)</td>
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*Member does not contain lap splice repair detail according to repair drawing*
### TABLE 9.3 EFFECTIVE STRESS RANGE IN MEMBERS

**OF SPAN F**

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<tr>
<th></th>
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<th>S&lt;sub&gt;re&lt;/sub&gt; @ Splice</th>
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<tbody>
<tr>
<td><strong>Bottom Chords</strong></td>
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<td>L0-L1 (L8-L9)</td>
<td>1.2 ksi</td>
<td>1.0 ksi</td>
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<td>L1-L2 (L7-L8)</td>
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<td>1.1</td>
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<tr>
<td>L2-L3 (L6-L7)</td>
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<td>*</td>
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<td>L3-L4 (L5-L6)</td>
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<td>1.3</td>
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<td>L4-L5</td>
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</tr>
<tr>
<td><strong>Hangers</strong></td>
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<td>M1-U1 (M8-U8)</td>
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<td><strong>Diagonals</strong></td>
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<td>L3-U1 (L6-U8)</td>
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<td>L4-U2 (L5-U7)</td>
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<td>L5-U3 (L4-U6)</td>
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<tr>
<td>L6-U4 (L3-U5)</td>
<td>3.2</td>
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*Member does not contain lap splice repair detail*
Fig. 1.1 Elevation Sketch of Bridge
Fig. 1.2 View of Bridge Looking Northeast

Fig. 1.3 View of Bridge Looking West
Fig. 1.4 View of Typical Built-up Truss Members

Fig. 1.5 View of a Typical Diagonal Comprised of 2 Eyebars
Fig. 1.6 View of a Lower Chord Member Comprised of 4 Eyebars

Fig. 1.7 View of Floor System showing Bottom Lateral Connections and a Reinforced Lower Chord
Fig. 1.8 Sketch of Bottom Lateral Arrangement Between 2 Panel Points
Fig. 1.9 View of Hanger M8-U8 in North Truss of Span F
Showing Welded Lap Splices on Eyebars

Fig. 1.10 View of Lower Chord Eyebar with Welded Lap Splice

-102-
Fig. 1.11 Sketch of Bottom Corner of Floor Beam Depicting Crack in Beveled Web Gap
Fig. 2.1 Crack at Upper End of Outside Splice Plate in Outside Eyebar of Hanger M8-U8N in Span F.

Fig. 2.2 Crack in Weld Metal @ Center of Double Lap Splice of the Same Bar
Fig. 2.3 Crack at Top End of Weld Splice in Outside Eyebars of Hanger M8-U8S in Span F

Fig. 2.4 Crack in Lower End of Lap Splice of the Same Bar
Fig. 2.5 Small Crack at Weld Toe of Lap Spliced Diagonal L3-U4N in Span B

Fig. 2.6 View of Counter Showing Original Eyebars and Welded Steel Reinforcing Bars
Fig. 2.7 Crack in Slot Weld of Counter L1-U2S of Span G

Fig. 2.8 Close-up View of Crack in Weld of Handrail Connection on Built-up Vertical Hanger
Fig. 2.9 Sketch Showing the Notching of Lateral Tee Stems
Fig. 2.10 View of Notch in Intersecting Bottom Laterals in Span G

Fig. 2.11 Close-up View of Flame Cut Notch of Tee Stem showing Small Fatigue Crack
Fig. 2.12 Close-up View of Small Fatigue Crack in Notch

Fig. 2.13 View of Deeper Notch in Stem where Fatigue Crack Propagated into Flange
Fig. 2.14 Crack Propagating into Flange of Bottom Lateral

Fig. 2.15 End Post-Lateral Connection Plate with Fatigue Crack at Notched Corner
Fig. 2.16 Close-up View of Fatigue Crack at Notch

Fig. 2.17 View of Welded Triangular Patch Plate on the Upstream West Side of Floor Beam 3 in Span B
Fig. 2.18  Fatigue Crack which originated at Beveled Web Gap on Floor Beam 2 in Span D

Fig. 2.19  Crack forming in Horizontal Weld between Flange Angle and Patch Plate
Fig. 2.20  Crack in Vertical Weld of Floor Beam 3 in Span G
Fig. 2.21 View of Upper End of Patch Plate With Arrow Pointing to CrackOriginating in Weld and Extending to Rivet Hole (FB3 - Span D)

Fig. 2.22 Close-up View of Crack after Sandblasting and applying Dye Penetrant
Fig. 2.23 Crack in Connection Angle on Upstream East Face of Floor Beam 3 in Span C

Fig. 2.24 Close-up View of Crack Extending from Weld Termination into Rivet Hole
Fig. 2.25 View of a Replaced Connection Angle installed with High Strength Bolts and Rewelded to Patch Plate

Fig. 2.26 View of Fatigue Crack which Reinitiated at Weld Termination
Fig. 2.27 View of Coped Bottom Flange and Bevelled Gap Showing Small Crack in Weld

Fig. 2.28 View of Crack in Coped Bottom Flange
Fig. 3.1 View of Gages on Diagonal L4-U5 in Downstream Truss of Span C

Fig. 3.2 View of Gages on Counter L3-U4 in Upstream Truss of Span D
Fig. 3.3 View of Gages on Lower Chord L4-L5 in Upstream Truss of Span D

Fig. 3.4 View of Gages on Lower Chord L1-L2 in Upstream Truss of Span F
Fig. 3.5 View of Gages on Diagonal L2-U1 in Upstream Truss of Span F

Fig. 3.6 View of Gages on Upstream East Face of Floor Beam 6 in Span D
Fig. 3.7 View of Gages on Upstream West Face of Floor Beam 7, Bottom Lateral L7N-L8S and Hanger Channel Flanges in Span D

Fig. 3.8 View of Gages on Coped Lateral Gusset and Bottom Lateral L8N-L7S in Span D
Fig. 3.9 Sketch of Exact Gage Locations on East Face of Floor Beam 6 in Span D
Fig. 3.10 Sketch of Exact Gage Locations on West Face of Floor Beam 7 in Span D
Fig. 3.11 Sketch of Cage Locations in Vertical Web Gap of East Face of Floor Beam 7 in Span G
Fig. 3.12 Strain Recording Equipment
Fig. 3.13  Test Train
Fig. 3.14 Strain-Time Response of Gage 68R on Hanger M1-ULS in Span F
Fig. 4.1  Computer Generated Plot of Span D
Fig. 4.2 Wheel Spacing of Test Engines and Cars
Fig. 4.3 Comparison of Measured and Theoretical Responses for Lower Chord L4-L5N
Fig. 4.4 Comparison of Measured and Theoretical Responses for Diagonal L4-U5S
Fig. 4.5 Traces Showing Unequal Stress Distribution in Eyebars of Diagonal L4-U5S
Fig. 4.6 Strain Traces Showing Bending Gradient in Lap Splice of Diagonal L4-U5S
Fig. 4.7 Comparison of Measured and Theoretical Responses of Counter L3-U4N
Fig. 4.8 Traces Showing Unequal Distribution Among Members of Counter L3-U4N
Fig. 4.9 Comparison of Measured and Theoretical Responses of Bottom Lateral L7W-L8S
Fig. 4.10 Comparison of Measured and Theoretical Responses of Bottom Lateral L8N-L7S
Fig. 4.11 Comparison of Measured and Theoretical Responses of Bottom Lateral L6N-L7S
Fig. 4.12 Comparisons of Measured and Theoretical Responses of Bottom Lateral L7N-L6S
Fig. 4.13 Comparison of Measured and Theoretical Responses for North Channel of Hanger L7-U7N
Fig. 4.14 Comparison of Measured and Theoretical Responses for South Channel of Hanger L7-U7N
Fig. 4.15 Comparison of Measured and Theoretical Responses for Bottom Flange of Floor Beam 7
Fig. 4.16 Comparison of Measured and Theoretical Responses for Eyebar M1-815 in Span F
Fig. 5.1 Eastbound and Westbound Traces for Lower Cord L4-L5N Showing No Directional Effects
Fig. 5.2 Eastbound and Westbound Traces for Diagonal L4-U5S Showing No Directional Effects
Fig. 5.3 Eastbound and Westbound Traces for Lateral L7N-L8S Showing No Directional Effects
Fig. 5.4 Eastbound and Westbound Traces for Lateral L8N-L7S Showing No Directional Effects
Fig. 5.5 Strain-Time Traces for 15 and 30 MPH
ML-ULS - Span F (Gage 68R)
Fig. 5.6 Traces of Gages in Vertical Web Gap of Floor Beam 7 in Span G
5.7 Analytical Response of Diagonal L6-U7
Fig. 5.8 Maximum Bending Stress in Plane of Truss for Verticals L1-U1 to L7-U7
train speed = 24 km/hr

Fig. 5.9 Eastbound and Westbound Traces for Gages on Bottom Flange of Floor Beam 7
Fig. 5.10 Stress Gradients Across Bottom Flange for Several Time Frames in Fig. 5.9
Fig. 5.11 Horizontal Bending Moments in Bottom Flanges of Floor Beams 1 through 7
Fig. 5.12 Displacement Responses of Floor Beam 7 at Stringer and Hanger
Fig. 5.13 View of Bottom Lateral to Lower Chord Connection on Span G
Bottom Lateral Stress Distribution - CASE 1 MPa (KSI)

Fig. 5.14 Measured and Computed Stress Distribution in Lateral L7N-L8S - Case 1
(Laterals Attached to Stringers and Ends of Floor Beams)
Bottom Lateral Stress Distribution - CASE 3  MPa (KSI)

Fig. 5.15 Measured and Computed Stress Distribution in Lateral L7N-L8S - Case 3
(Laterals Attached to Ends of Floor Beams)
Fig. 5.16 Measured and Computed Stress Distribution in Lateral L8N-L7S - Case 1
(Laterals Attached to Stringers and Ends of Floor Beams)
Bottom Lateral Stress Distribution - CASE 3  MPa (KSI)

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<td>65.5 (9.5)</td>
</tr>
<tr>
<td>54.5 (7.9)</td>
<td>50.3 (7.3)</td>
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<tr>
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Fig. 5.17 Measured and Computed Stress Distribution in Lateral L8N-L7S - Case 3  (Laterals Attached to Ends of Floor Beams)
Fig. 6.1 Computer Plot of Refined Global Mesh for Span D
Fig. 6.2 East Face Longitudinal Web Stress Near Connection Angle
Fig. 6.3 East Face Longitudinal Web Stress Near Connection Angle
Fig. 6.4 Longitudinal Stress in East Connection Angle
Fig. 6.5 East Face Transverse Web Stress Near Stringer
Fig. 6.6 East Face Transverse Web Stress Near Connection Angle
Fig. 6.7 Transverse Stress in East Connection Angle
Fig. 6.8 Longitudinal (out-of-plane) Displacement Along Bottom Flange
Fig. 6.9 Out-of-Plane Rotation Along Bottom Flange
Fig. 6.10 Horizontal (out-of-plane) Displacement Along Connection Angle
Fig. 6.11 Out-of-Plane Rotation Along Connection Angle
Fig. 6.12 Computer Plot of First Level Substructure Model
Fig. 6.13 East Face Longitudinal Web Stress Near Vertical Stiffener
Fig. 6.14 East Face Longitudinal Web Stress Near Connection Angle
Fig. 6.15 Longitudinal Stress in East Connection Angle
Fig. 6.16 Location of Cross-Sections where stress distributions are analyzed in floor beam.
Fig. 6.17 East Face Transverse Web Stress Near Vertical Stiffener
Fig. 6.18 East Face Transverse Web Stress Near Connection Angle
Fig. 6.19 Transverse Stress in East Connection Angle
Fig. 6.20 Out-of-Plane Rotation Along Bottom Flange
Fig. 6.21 Out-of-Plane Rotation Along Connection Angle
Fig. 6.22 Strain Traces of Gages 46R and 62R for Westbound Test Train
Fig. 6.23 Strain Traces of Gages 62W and 66R for Westbound Test Train
Fig. 6.24 Strain Trace of Gage 53R for Westbound Test Train
Fig. 6.25 Traces for L4-L5 and Gage 46R Showing Location of Load Case 10
Fig. 6.26 View of Typical Bevelled Web Gap
Fig. 6.27 Mesh of First Level Substructure Model Showing Size of Second Level Web Gap Model
Fig. 6.28 Computer Plot of Second Level Substructure Model of Web Gap
Stresses in Web Gap of Original Detail (in ksi)

Fig. 6.29 Transverse Stress in East Face of Web Gap for the Original Condition
Fig. 6.30 Out-of-Plane Rotation Along Bottom Flange and in Web Gap for Original Condition
Stresses in Web Gap of Present Detail (in ksi)

Fig. 6.31 Transverse Stress in East Face of Filled-in Web Gap for Present Condition
Fig. 6.32 Out-of-Plane Rotation Along Bottom Flange and in Web Gap for Present Condition
Fig. 7.1 Summary of Train Frequency Between 1937 and 1964
Fig. 7.2 Change in Locomotive Weight Between 1937 and 1982
Fig. 7.3 Estimated Number of Locomotive Units Per Train
Fig. 7.4 Summary of Gross Tonnage Per Year
Fig. 7.5 Frequency of Car Loads Over 60 Tons
Fig. 7.6 Influence Diagrams for Critical Members
Span D (A, B, and C)
Fig. 7.7 Influence Diagrams for Critical Members - Span F
Steam Locomotive

Diesel Locomotive

Car

Fig. 7.8 Typical Wheel Spacing Configurations
Fig. 8.1 Detail of Welded Splice Plates
Fig. 8.2 Tensile Test Results - Specimen A

Yield load = 5500 lbs

Static yield load = 4800 lbs

Strain (in./in.)

Machine Load (k)
Fig. 8.3 Tensile Test Results - Specimen B
Fig. 8.4 Test Specimen Mounted in Alternating Stress Machine
Fig. 8.5 Comparison of Fatigue Results at Welded Steel Lap Plates With Fatigue Strength Curves C, D, & E
Fig. 8.6 Comparison of the Weld Toe Test Results with Categories C, D, & E Resistance Curves
Fig. 8.7 Crack Surface in Weld and End of Flame Cut Gap
Fig. 8.8 Polished and Etched Surface Showing Fatigue Crack at Weld Toe of Fabricated Specimen

Fig. 8.9 Polished and Etched Surface of Hanger Showing Weld Toe Crack
Fig. 9.1 Measured Stress Range Spectrum in Chord Member L4-L5 - Span D
Fig. 9.2 Measured Stress Range Spectrum in Diagonal Member L4-L5 - Span D
Fig. 9.3 Measured Stress Range Spectrum in Hanger L7-U7 - Span D
Sr Miner (1982 Traffic)
Sr Miner (Measured)

Fig. 9.4 Measured Stress Range Spectrum in Lower Chord L1-L2 - Span F
Fig. 9.5 Measured Stress Range Spectrum in Diagonal Member L2-U1 - Span F
Fig. 9.6 Measured Stress Range Spectrum in Load Bar of Hanger M1-U1 - Span F
Fig. 9.7 Comparison of the Effective Stress Range Estimated for 1982 and 2000 with Fatigue Resistance Curves C, D, & E - Spans A, B, C, & D
Fig. 9.8 Comparison of the Effective Stress Range Estimated for 1982 and 2000 with Fatigue Resistance Curves C, D, & E - Span F
Fig. 9.9 Cracked U-Shaped Strap of Reinforced Truss Members

Bearing Surface

Outside Surface

Fig. 9.10 Fatigue Crack Surface
Fig. 9.11 Crack at Sharp Reentrant Corner of Net Section
Fig. 10.1 Schematic Sketch of Triangular Splic Plates for Retrofitting Floor Beam - Hanger Connections
Fig. 10.2 East Face Web Stress Along Connection Angle

-221-
Fig. 10.3 East Face Web Stress Along Correction Angle
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