A SURVEY OF LOCALIZED
CRACKING IN STEEL BRIDGES

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This report summarizes the results of a survey of localized failures due
to fatigue and/or fracture in bridge members. Altogether, information on
142 bridges was obtained. The cracking observed was classified into 28
types of details.

A representative sample of fifteen of the bridges was selected for a
detailed review, and a summary was prepared describing the bridge
structure, the cracking, its cause, and the repair, retrofit, and cost.
These case studies are a part of this report.
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   1.1 Summary Tables
   1.2 Selected References

2. Eyebars and Pin Plates
   2.1 Silver Bridge (I.D. 2)
   2.2 Illinois 157 over St. Clair Avenue (I.D. 36)

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1. INTRODUCTION

As part of this study, a survey of localized failures due to fatigue and brittle fracture of domestic and foreign bridge structures was carried out. Detailed information was not readily available from most foreign sources. Some information was made available but for the most part the information was not extensive.

A mail solicitation was made of several states in order to obtain supplemental information on cases of cracking that were known to exist and where further information was needed in order to summarize the cracking.

In addition, visits were made to several states in order to obtain additional information on known cases of cracking and to acquire new information. The states visited included: California, Illinois, Iowa, Louisiana, Minnesota, Oregon, Texas, Washington and Wisconsin.

Altogether information was acquired on 142 bridge sites. An effort to group cracking problems into categories required use of more than 30 terms to represent the various types of cracking problems. Often a bridge site contained more than one kind of bridge structure and several types of cracking problems were observed. Table 1 summarizes the design details where cracks developed and for which information was acquired. It also shows the number of bridge sites where the various kinds of cracks were detected.

A representative sample of these bridges was selected for a detailed review and summaries were prepared which describe the bridge structure, the cracking and its causes, the repair or retrofit and the cost. These
summaries form the basis for this report.

Table 2 provides a list of most bridge sites, the state they are located in, and a brief description of the detail or condition that led to cracking.

Each summary has its own reference list and sequence of figures. References cited in the individual introductory sections are listed at the end of report.
<table>
<thead>
<tr>
<th>Detail</th>
<th>Initial Defect or Condition</th>
<th>Number of Bridges</th>
<th>Fatigue Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Eyebars</td>
<td>Stress corrosion</td>
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<td>Forge laps, unknown defects</td>
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<td>2. Pin Plates</td>
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<td>Other</td>
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<td>3. Coverplated Beams</td>
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<td>E</td>
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<tr>
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<td>Fabrication cracks</td>
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<td>4. Flange Gussets</td>
<td>Weld toe</td>
<td>5</td>
<td>E or E'</td>
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<tr>
<td>5. Flange or Web Groove</td>
<td>Lack of fusion</td>
<td>6</td>
<td>Large initial crack</td>
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<tr>
<td>6. Coverplate Groove</td>
<td>Lack of fusion</td>
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<td>7. Electroslag Welds</td>
<td>Various flaws</td>
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</tr>
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<td>8. Longitudinal Stiffeners</td>
<td>Lack of fusion, poor weld</td>
<td>4</td>
<td>Large initial crack</td>
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<td>9. Web Gusset</td>
<td>Intersecting welds</td>
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<td>Gap between stiffener &amp; gusset</td>
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<td>10. Flanges &amp; Brackets through Web</td>
<td>Flange tip crack</td>
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<td>11. Welded Holes</td>
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<td>12. Cantilever Brackets</td>
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<td>Riveted Connection</td>
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<td>13. Lamellar Tearing</td>
<td>Restraint</td>
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<td>14. Transverse Stiffeners</td>
<td>Shipping &amp; handling</td>
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<td>15. Floorbeam Connection Plates</td>
<td>Web gaps</td>
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<td>16. Diaphragm Connection Plates</td>
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<td>17. Diaphragm &amp; Floorbeam Connection Plates at Piers</td>
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<td>18. Tied Arch Floor Beams</td>
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<td>19. Tied Arch Floor Beam Connections</td>
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<td>20. Coped Members</td>
<td>Notch</td>
<td>13</td>
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<td>21. Welded Web Inserts</td>
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<td>22. Plug Welds</td>
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<td>Detail</td>
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<td>Number of Bridges</td>
<td>Fatigue Category</td>
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<td>23. Gusset Plates</td>
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<td>24. Box Girder Corner Welds</td>
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<td>25. Stringer-Floorbeam Brackets</td>
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<td>26. Stringer End Connections</td>
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<td>Weld termination</td>
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<td>27. Hangers (Truss &amp; Arches)</td>
<td>Vibration-Wind</td>
<td>4</td>
<td>Aeroelastic Instability</td>
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<td>28. Welded Repair</td>
<td>Lack of fusion, weld termin</td>
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TABLE 2: SUMMARY OF BRIDGES

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<th>Name</th>
<th>Location</th>
<th>Type Cracking</th>
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<tbody>
<tr>
<td>1</td>
<td>Kings Bridge</td>
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<td>West Va.</td>
<td>End coverplates</td>
</tr>
<tr>
<td>3</td>
<td>Yellow Mill Pond</td>
<td>Conn.</td>
<td>End coverplates</td>
</tr>
<tr>
<td>4</td>
<td>Quinnipiac River</td>
<td>Conn.</td>
<td>Longit. stiffener groove weld</td>
</tr>
<tr>
<td>5</td>
<td>Aquasabon River</td>
<td>Ontario</td>
<td>Web insert groove weld</td>
</tr>
<tr>
<td>6</td>
<td>Lafayette Street</td>
<td>Minn.</td>
<td>Lateral gusset-web</td>
</tr>
<tr>
<td>7</td>
<td>Dan Ryan Elevated</td>
<td>Illinois</td>
<td>Flange piercing web</td>
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<tr>
<td>8</td>
<td>I79 Back Channel</td>
<td>Pa.</td>
<td>Electroslag welds</td>
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<td>Pa.</td>
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<td>Brady Street</td>
<td>Pa.</td>
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</tr>
<tr>
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<td>Meadville</td>
<td>Pa.</td>
<td>Electroslag welds</td>
</tr>
<tr>
<td>12</td>
<td>I78</td>
<td>N.J.</td>
<td>Longit. stiffener groove welds</td>
</tr>
<tr>
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<td>Floorbeam web gaps</td>
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<td>Cantilever bracket plates</td>
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<td>Chester-Bridgeport</td>
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<td>Verticals-wind vibration</td>
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<td>Cantilever bracket plates</td>
</tr>
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<td>South Bridge</td>
<td>Pa.</td>
<td>Cantilever bracket plates</td>
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<td>Country Club</td>
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<td>Corrosion &amp; fatigue</td>
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<td>Ft. Duquesne Bent</td>
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<td>Lamellar tearing</td>
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<td>St. Anne, Ottawa River</td>
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<td>Welded patch plates</td>
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<td>22</td>
<td>I24 Bridge at Paducah</td>
<td>Ill. - Ky.</td>
<td>Thrust brackets, groove welds &amp; box corner welds</td>
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<td>23</td>
<td>Des Plaines River</td>
<td>Ill.</td>
<td>Web gaps</td>
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<td>13.9 Mile Grimsby</td>
<td>Alberta</td>
<td>Diaphragm web gaps</td>
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<tr>
<td>25</td>
<td>5.09 Mile Edmonton</td>
<td>Alberta</td>
<td>Diaphragm web gaps</td>
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<tr>
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<td>Walt Whitman</td>
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<td>Floorbeam truss-stringer brackets</td>
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<tr>
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<td>Mackinac</td>
<td>Mich.</td>
<td>Floorbeam truss-stringer brackets</td>
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<td>C.P.R. Subway</td>
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<td>Longit. stiffener groove</td>
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<tr>
<td>30</td>
<td>Founders Bridge</td>
<td>Conn.</td>
<td>Lateral connection plates</td>
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<td>31</td>
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TABLE 2: SUMMARY OF BRIDGES (CONT'D)

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<tr>
<th>Bridge I.D. No.</th>
<th>Name</th>
<th>Location</th>
<th>Type Cracking</th>
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<tbody>
<tr>
<td>32</td>
<td>Seymour at Mill Pond</td>
<td>Conn.</td>
<td>Welded stringer connections</td>
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<td>33</td>
<td>Cromwell</td>
<td>Conn.</td>
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<td>34</td>
<td>Wainwright</td>
<td>Virginia</td>
<td>Pin plates</td>
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<td>36</td>
<td>C.P. Bridge, Dutch Creek</td>
<td>Canada</td>
<td>Cracked hangers</td>
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<td>37</td>
<td>Belle Vernon</td>
<td>Pa.</td>
<td>Stringer copes</td>
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<td>Dekorra Bridge</td>
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<td>Stringer angle connection rivets</td>
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<td>Girard Point Pier Caps</td>
<td>Pa.</td>
<td>Web crack; gusset cracks; flange crack</td>
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<td>40</td>
<td>I90 over Conrail Tracks</td>
<td>Ohio</td>
<td>Transverse stiffener web gaps</td>
</tr>
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<td>41</td>
<td>I90 over Cuyahoga River Valley</td>
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<td>Coverplate end</td>
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<td>Chamberlain</td>
<td>South Dakota</td>
<td>Cross-frame diaphragm web gaps</td>
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<td>45</td>
<td>Belle Fourche</td>
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<td>46</td>
<td>Tilford</td>
<td>South Dakota</td>
<td>Coverplate groove weld splice</td>
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<td>L&amp;N Bridgeport</td>
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<td>Eyebars</td>
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<td>Silver Memorial</td>
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<td>49</td>
<td>Southwest Miramichi</td>
<td>Canada</td>
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<td>50</td>
<td>35.2 Dublin Santa Fe</td>
<td>Texas</td>
<td>Eyebar head</td>
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<td>412.1 Santa Fe</td>
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<td>Prairie du Chien</td>
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## TABLE 2: SUMMARY OF BRIDGES (CONT'D)

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<thead>
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<th>Bridge I.D. No.</th>
<th>Name</th>
<th>Location</th>
<th>Type Cracking</th>
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<td>60</td>
<td>B37-30, Big Eau Pleine</td>
<td>Wisconsin</td>
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<td>Dresbach</td>
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<td>62</td>
<td>I229 Big Sioux River</td>
<td>South Dakota</td>
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<td>63</td>
<td>Napier Ave. - St. Joseph</td>
<td>Michigan</td>
<td>Riveted diaphragms</td>
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<td>Welded insert;</td>
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<td>65</td>
<td>Fremont</td>
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<td>Old Cairo</td>
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<tr>
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<td>Ill.</td>
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<td>I70 Fayette County</td>
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<td>Little Vermilion River</td>
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<td>Gusset plate cracks</td>
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<td>Crooked River Canyon</td>
<td>Oregon</td>
<td>Stringer end conn. &amp; floorbeam bracket</td>
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<td>Milwaukee Harbor</td>
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<td>New England Mills</td>
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<td>Merced River</td>
<td>Calif.</td>
<td>Stringer brackets-web</td>
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<td>Bryte Bend</td>
<td>Calif.</td>
<td>Flange gussets</td>
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<td>US 61 (657)</td>
<td>Iowa</td>
<td>Diaphragm web gap</td>
</tr>
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<td>87</td>
<td>Decatur (3567)</td>
<td>Iowa</td>
<td>Lateral (gusset) connection plate-web</td>
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<td>Bear Creek (3958)</td>
<td>Iowa</td>
<td>Diaphragm web gap</td>
</tr>
<tr>
<td>89</td>
<td>I80 (3758)</td>
<td>Iowa</td>
<td>Welded bolt holes</td>
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TABLE 2: SUMMARY OF BRIDGES (CONT'D)

<table>
<thead>
<tr>
<th>Bridge I.D. No.</th>
<th>Name</th>
<th>Location</th>
<th>Type Cracking</th>
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<tbody>
<tr>
<td>90</td>
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<td>Groove welded blast plates</td>
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<tr>
<td>91</td>
<td>Yaquina Bay Bridge</td>
<td>Oregon</td>
<td>Floorbeam end restraint</td>
</tr>
<tr>
<td>92</td>
<td>Siuslaw River Bridge</td>
<td>Oregon</td>
<td>Cracked Stringer webs-Basquile</td>
</tr>
<tr>
<td>93</td>
<td>Coos Bay</td>
<td>Oregon</td>
<td>Floorbeam web above end conn.</td>
</tr>
<tr>
<td>94</td>
<td>Santiam River Bridge</td>
<td>Oregon</td>
<td>Coped stringers</td>
</tr>
<tr>
<td>95</td>
<td>Thruway-Buffalo</td>
<td>N.Y.</td>
<td>Longit. stiffener groove</td>
</tr>
<tr>
<td>96</td>
<td>Clarion River Bridge</td>
<td>Pa.</td>
<td>Floorbeam web gap</td>
</tr>
<tr>
<td>97</td>
<td>Canoe Creek Bridge</td>
<td>Pa.</td>
<td>Floorbeam web gap</td>
</tr>
<tr>
<td>98</td>
<td>Fayette County (359)</td>
<td>Iowa</td>
<td>Bearing stiffener web gap</td>
</tr>
<tr>
<td>99</td>
<td>Red Rock Reservoir (613A)</td>
<td>Iowa</td>
<td>Bearing stiffener web gap</td>
</tr>
<tr>
<td>100</td>
<td>Cedar River I80 (8859)</td>
<td>Iowa</td>
<td>Bearing stiffener web gap</td>
</tr>
<tr>
<td>101</td>
<td>Moshannon Creek</td>
<td>Pa.</td>
<td>Coped stringers</td>
</tr>
<tr>
<td>102</td>
<td>Ramp C Viaduct I83-I695</td>
<td>Md.</td>
<td>Box girder web gaps</td>
</tr>
<tr>
<td>103</td>
<td>I470 Ohio River</td>
<td>West Va.</td>
<td>Hanger cable fatigue</td>
</tr>
<tr>
<td>104</td>
<td>BN Sandpoint</td>
<td>Idaho</td>
<td>Weld repair-flange plate</td>
</tr>
<tr>
<td>105</td>
<td>Metro Pier Caps</td>
<td>Wash., D.C.</td>
<td>Web penetration</td>
</tr>
<tr>
<td>106</td>
<td>Shitoku-Ohashi Tied Arch</td>
<td>Japan</td>
<td>Steel pipe diagonals-wind vibration</td>
</tr>
<tr>
<td>107</td>
<td>Mukaijima-Ohashi Tied Arch</td>
<td>Japan</td>
<td>H-shaped hangers-wind vib.</td>
</tr>
<tr>
<td>108</td>
<td>Maoroshi Tied Arch</td>
<td>Japan</td>
<td>H shaped hangers-wind vib.</td>
</tr>
<tr>
<td>109</td>
<td>Dallas Elevated</td>
<td>Texas</td>
<td>Intersecting welds, end restraint</td>
</tr>
<tr>
<td>110</td>
<td>Green Bayou Bridge</td>
<td>Texas</td>
<td>Lateral brace welded to flange</td>
</tr>
<tr>
<td>111</td>
<td>Red River Bridge</td>
<td>Texas</td>
<td>Coped flange at joint</td>
</tr>
<tr>
<td>112</td>
<td>Port Allen Intracoastal</td>
<td>La.</td>
<td>Coped finger joint seat</td>
</tr>
<tr>
<td>113</td>
<td>Sunshine Bridge</td>
<td>La.</td>
<td>Coped sidewalk bracket</td>
</tr>
<tr>
<td>114</td>
<td>I90 over Train Ave.</td>
<td>Ohio</td>
<td>Transverse stiffener web gap</td>
</tr>
<tr>
<td>115</td>
<td>Third St. Via., Columbus</td>
<td>Ohio</td>
<td>Lateral connection plate</td>
</tr>
</tbody>
</table>
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Note: This list of references is cited in the introduction and in the introductory articles provided for each type of cracking discussed. Each example has its own list of references.
2. **EYEBARS AND PIN PLATES**

The failure of the Point Pleasant or Silver Bridge (I.D. 2) in 1967 as a result of the fracture of an eyebar is well known and a summary of this failure is provided hereafter.

Several other eyebars have cracked in bridge structures although none resulted in a collapse. At least two railroad bridges in the U. S. developed fatigue cracks in eyebar heads that resulted in fracture (I.D. 52, I.D. 48). One such failure that occurred in a Santa Fe Bridge in Oklahoma (I.D. 52) was found to be caused by forge laps in the eyebar head that happened to be oriented normal to the axis of the pin. This served as an initial crack that resulted in fatigue crack growth and eventual fracture.

Five Japanese railway bridges built between 1900 and 1913 have developed fatigue cracks at the eyebar head normal to the axis of the pin. A description of the Fujigawa Bridge is given in Ref. 1. About 820,000 train crossings have occurred over most of these structures.

Two bridge structures have experienced cracking in pin plates which supported suspended spans. One such crack formed on the net section at the pin hole, much like an eyebar failure (I.D. 34). However, in the second structure (I.D. 36) several pin plates failed in the gross section as a result of corrosion product that "welded" the pin plate heads to the girders. This caused bending stresses to be developed in the pin plates and the fixity at the ends resulted in cracking at the corroded area. Bending stresses were observed in the pin plates of two bridges that were
instrumented during the summer of 1981. Friction and corrosion product provided fixity at the pins and caused rigid link behavior and bending.

A summary of the structure with cracked pin plates is provided in this section on the Illinois 157 bridge (I.D. 36).
2.1 FATIGUE/FRACTURE ANALYSIS

of

POINT PLEASANT (SILVER) BRIDGE OVER OHIO RIVER

-- A Brief Summary --

I.D. No. : 2
Town/City/etc. : Point Pleasant, W.V. and Kanauga, Ohio
Railway/Highway/Avenue : US. 35
State : West Virginia
Owner : West Virginia-DOT
1. DESCRIPTION AND HISTORY OF THE BRIDGE

   - Brief Description of Structure

   The Point Pleasant Bridge which carried U.S. 35 highway over the Ohio River was located between Point Pleasant, West Virginia and Kanauga, Ohio. The bridge was also known as the "Silver Bridge" because it was one of the major structures to be painted with aluminum paint (Ref. 1). It was one of two nearly identical and unique eyebar chain suspension bridges in the U.S. The other bridge, also spanning the Ohio River was at St. Mary's, West Virginia until it was dismantled.

   This bridge was an important link between major cities on both sides of the Ohio River. It was also an important local artery for industrial plants in Point Pleasant and the Gallipolis areas of Ohio.

   The Point Pleasant Bridge was an eyebar chain suspension bridge with its axis in an east-west direction over the Ohio River. It had a 700 ft. center or main span and two 380 ft. side spans as shown in Fig. 1 (Ref. 1). In addition, there were two approach spans on each side of the bridge. They were plate girder spans 75.25 ft. and 71.50 ft. in length supported on concrete piers. The two suspension bridge towers extended 130 ft-10.25 in. above the tops of the two main piers. The total length of the bridge was 1,753 ft. (Ref. 1).

   The roadway of the suspended span as originally built in 1928, consisted of a timber deck and sidewalks. In 1941, the timber deck was replaced with a 3-inch deep steel grid floor filled with concrete. The new roadway was 21 feet wide for two lanes of traffic with a 5 ft-8 inch sidewalk. The new deck caused no significant change in the dead load of the structure.
The bridge superstructure was unique in that the stiffening trusses of both the center span and the two side spans were framed into the eyebar chain to make up the top chord of about half of the length of the stiffening trusses (Ref. 1). Most of the eyebars were between 45 and 55 feet in length of varying thickness (1.5 to 2.2 in) depending on the truss panel in which they were located. The shank width of eyebars was 12.0 in. They were made of heat treated rolled carbon steel bars with forged heads designed to break at ultimate loading in the shank. Each joint was composed of 4 eyebar heads and a connecting pin or pin rod, and was secured by double nuts on each end of the pin rod as shown in Figs. 3, 4 and 5. A view of the stiffening truss and joint details are given in Figs. 4 and 6, 7, 8.

**Brief History of Structure and Cracking**

Construction began on the Point Pleasant Bridge on May 1, 1926 and the Bridge was opened to traffic on May 19, 1928. The bridge originally was built and owned by the W. Virginia-Ohio River Bridge Corporation. On December 24, 1941, the State of W. Virginia purchased the Point Pleasant Bridge and continued to operate the bridge under the authority of the State Highway Commission as a toll bridge until January 1952, when it became toll free (Ref. 1).

For almost 40 years, it carried an increasing number of vehicles across the Ohio River (Ref. 1). On December 15, 1967 the Point Pleasant Bridge collapsed suddenly without any warning about 5:00 PM with the loss of 46 lives and 37 vehicles of all types (Ref. 1). The three suspended sections fell within 60 seconds. The collapse was immediately preceded by several loud "cracking" sounds on the Ohio side span superstructure. According to eyewitness accounts, the bridge started to fall, hesitated, and then collapsed.
completely with a "roaring" sound that lasted during the collapse (Ref. 1).
The temperature at the time of collapse was recorded as 30°F (Ref. 1).

It has been clearly established by the investigation (Ref. 1) following
the failure that the nominal static stresses in all eyebar elements at the
time of collapse did not exceed those provided for in the original design.
The main cause of collapse (Refs. 2,3,4) was traced back to the unstable
extension or brittle fracture of two corrosion cracks located at the pinhole
of one of the four eyebars (Eyebar 330) of joint C13 north (see also Figs. 1,
2 and 6). The design of the chain was unique in that only two eyebars were
used in each chain segment and the chain also formed a portion of the top
chord of the stiffening trusses. Thus, failure of any one eyebar in the
bridge would cause the complete collapse of the structure (see Fig. 9).
2. FAILURE MODES AND FAILURE ANALYSIS

Location of Fracture, Failure Modes and Stages

Model test results (Ref. 3) on an eyebar and joint assembly made up from the structural elements recovered from the wreckage similar to those of joint Cl3N indicate that the separation of joint Cl3N was the key event in the sequence of collapse of the Point Pleasant Bridge. Several possibilities for separation of Cl3N were considered and tested which imitated the cyclic and static load conditions at the time of failure. Physical evidence on the components of the joint by the induced brittle fracture through the lower limb of eyebar 330, as shown in Fig. 8, was essentially identical to that of the model prototype. Also the opening of the eye of the eyebar due to plastic strain as well as the appearance of this second fracture in the upper limb of this same eye are essentially similar. Branching of the model test opposite limb of the eyebar introduced a slightly different appearance after the midwidth of the section (Ref. 3). Also, it was observed during the model test that there was an unexpectedly long time interval between the instant of brittle fracture in the lower limb of eyebar 330 and the subsequent complete separation of the joint. This indicated that some time interval would likely be present during collapse of the bridge structure.

Other studies (Ref. 4) corroborated the mechanism of stable crack extension responsible for the crack which triggered the collapse. It was concluded that fatigue was probably not the cause of the crack extension (Ref. 4). The low live load stress range in the eyebar would not be large enough to cause appreciable fatigue crack propagation.
The two corrosion-fatigue cracks from which the fracture of the eyebar initiated are shown in Fig. 11. Figure 12 shows a sketch of the flaws and their dimensions (Ref. 2). Also shown in Fig. 12, are the results of a hardness survey through the thickness of the eyebar. Due to heat treatment of the eyebar a harder layer existed at a depth of 0.1-0.4 in. from the surface. This also decreased the toughness of the material. Since the two flaws lie in the area of highest hardness, the local fracture toughness was less than the average value shown in Fig. 13 (Ref. 2).

Examination of the flaws and the crack surface revealed considerable corrosion and corrosion product. At the location of the flaws, contaminated water could be trapped in a crevice area of high stress and high local hardness. Hence, stress corrosion appears to have been the dominant mechanism for the creation of the two initial flaws or cracks. These flaws coupled with the low toughness at the time of collapse caused the sudden brittle fracture phase in the lower limb of the eye of the eyebar and consequently the catastrophic failure of the Point Pleasant Bridge.

Cyclic Loads and Cyclic Stresses (Known or Field Recorded)

A traffic survey conducted by Ohio-DOT in 1964 indicated that 6,640 vehicles (ADT) crossed the structure as a result of the two-way traffic. There were 5,040 of the vehicles that were passenger cars, and the remaining 1,600 were trucks.

The bridge was not posted for weight limits. However, an "overload" permit was required for all vehicles over 70,000 pounds. On December 7, 1967 a tractor trailer weighing 98,000 lbs. crossed the bridge without any unusual occurrences.
A computer analysis of the bridge was conducted after the collapse, and it was found that the unit stress at collapse was less than the 1927 design unit stress for all essential structural members in the bridge (Ref. 1). Figure 15 summarizes the results for eyebars 11-13 (Ref. 4). The mechanical characteristics of the bridge steel obtained from tests after the failure are given in Fig. 16 a,b (Ref. 3).

Temperature and Environmental Effects

The atmosphere provided the necessary elements for the development of stress corrosion cracking in the heat treated 1060 eyebar steel. Thus the size of the corrosion induced cracks was increasing with passage of time.

In addition, the recorded temperature at the time of collapse was 30°F (Ref. 1). The fracture resistance was then low enough in comparison with crack size so that onset of rapid brittle fracturing developed.

The measured static fracture toughness $K_{IC}$ of the specimens from the eyebar material and the estimated dynamic toughness $K_{ID}$ are plotted in Fig. 12. Approximate values of $K_{IC}$ and $K_{ID}$ corresponding to the temperature of 30-32°F at the time of collapse are indicated in Fig. 12.

Material "Mechanical, Chemical and Fracture Characteristics"

Analysis in the Failure Regions

Several specimens were tested from the shank portion of the eyebar members of the collapsed Point Pleasant Bridge.
Chemical and metallurgical characteristics of the material were reported and evaluated in Ref. 2. The tensile and fatigue characteristics of the eyebar steel from the collapsed bridge was also investigated, and the results are reported in Ref. 4.

The tensile strength values obtained from standard and special tension specimens at room temperatures met the minimum strength requirements of the original specifications, as can be seen in Tables 1 and 2. No major variation in the tensile properties of the eyebar steel is apparent.

An analytical and experimental study of the fatigue characteristics of the small specimens fabricated from the eyebars of the collapsed bridge showed that the small stress range levels probable at eyebar Cl3N would be too low to cause the imitation and extension of the flaws and cause failure by fatigue (Refs. 4,5). The design live load stress in the shank of the eyebar was 12.5 ksi. The dead load design stress was 36 ksi. At the time of failure, the total shank stress was estimated to be 41 ksi. The stress intensity estimate of the crack tip was about 50 ksi√ft. (Ref. 4).

The measured static toughness $K_{IC}$ and the estimated dynamic toughness $K_{Id}$ of the eyebar material are shown in Fig. 12 (Ref. 5). A temperature shift of 100°F was used to estimate the dynamic fracture toughness. At the failure temperature of 30-32°F, the value of $K_{IC}$ is 41 ksi√ft. and $K_{Id}$ is 26 ksi√ft. (Ref. 5).

The initial flaws or cracks from which the fracture of the eyebar initiated are shown in Figs. 10 and 11. Figure 11 also shows results of a hardness survey through thickness of the eyebar. Due to heat treatment of the original eyebar during manufacturing, the material in the center of
the plate is harder than the material on the surface. The flaws are located in the area of maximum hardness (Ref. 5) which produces lower fracture toughness. It seems probable that the two corrosion cracks would have eventually coalesced and increased $K$ substantially. Hence fracture would likely have occurred with a higher toughness material as well if it was stress corrosion sensitive.

Visual and Fractographic Analysis of Failure Surfaces

Examination of the flaw surfaces on the Cl3N eyebar, revealed considerable quantities of corrosive products. Stress corrosion tests on the material showed that it was corrosion sensitive. The fatigue tests on specimens with machined surfaces indicated that eyebars of the structure should have adequate fatigue resistance (Ref. 4).
3. CONCLUSIONS AND RECOMMENDATIONS

Conclusions Regarding Fracture/Failure

The following conclusions can be made regarding the collapse of the Point Pleasant (or Silver) Bridge (Refs. 4,5):

a) The tensile properties of the eyebars in the structure met the minimum requirements of the original design.

b) The mechanism of crack extension of the initial flaws in Cl3N eyebar to the critical size at failure was most likely caused by stress corrosion. The eyebar material provided to be stress corrosion sensitive. The location of the initial flaws in an area of high local hardness in combination with stress corrosion sensitivity appears to have been the primary cause of failure.

Recommendations for Repair/Fracture Control

This is not applicable to the case of the Point Pleasant Bridge failure. The structure collapsed suddenly before the flaws and stress corrosion cracking was detected.

4. ACTUAL REPAIR/FRACTURE CONTROL EFFORTS

Not applicable to this case.
5. POSSIBLE ADVERSE EFFECTS OF REPAIR

Not applicable to this case.

6. ACTUAL OR ESTIMATED REPAIR/REHABILITATION COSTS

   . Cost of Failure Investigation

   This is not available. The failure of the Point Pleasant Bridge was investigated by Federal Highway Administration, Washington, D.C. (see, for instance, Reference List on this Summary).

   . Cost of Repair/Rehabilitation

   Not applicable to this case.

   . Original (or Adjusted) Cost of Structure

   This is not available.
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APPLICATION OF FRACTURE MECHANICS TO ANALYSIS OF BRIDGE FAILURES
### TABLE 1

TENSILE PROPERTIES OF THE MATERIAL AT ROOM TEMPERATURE

<table>
<thead>
<tr>
<th>SPECIMEN NUMBER</th>
<th>TENSILE STRENGTH</th>
<th>YIELD STRENGTH</th>
<th>ELONGATION IN 2 INCHES</th>
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<tbody>
<tr>
<td></td>
<td>k.s.i.</td>
<td>k.s.i.</td>
<td>percent</td>
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<td>1</td>
<td>116.1</td>
<td>76.9</td>
<td>23.62</td>
</tr>
<tr>
<td>2</td>
<td>116.1</td>
<td>79.4</td>
<td>21.50</td>
</tr>
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<td>4</td>
<td>115.1</td>
<td>79.45</td>
<td>20.90</td>
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<td>5</td>
<td>114.95</td>
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<td>113.55</td>
<td>79</td>
<td>22.00</td>
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<td>AVERAGE</td>
<td>116</td>
<td>79</td>
<td>21.58</td>
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</table>

### TABLE 2

TENSION TEST RESULTS TOGETHER WITH THE MINIMUM STRENGTH REQUIREMENTS OF ORIGINAL DESIGN SPECIFICATIONS

<table>
<thead>
<tr>
<th></th>
<th>Yield Strength k.s.i.</th>
<th>Tensile Strength k.s.i.</th>
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<td>Average Results of</td>
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<td>116</td>
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<td>Standard Specimens</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Special Specimen</td>
<td>82</td>
<td>109</td>
</tr>
<tr>
<td>Values as Required* by</td>
<td>75</td>
<td>100 to 105*</td>
</tr>
<tr>
<td>Specification</td>
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* Taken from original drawing prepared by J. E. Greiner Co.
TABLE 3  COMPUTED STRESSES IN THE PROTOTYPE EYEBAR
WHICH WERE REPRODUCED IN THE MODEL EYEBARS.

<table>
<thead>
<tr>
<th>DESIGN LOADS IN POUNDS</th>
<th>ACTUATOR LOADS IN POUNDS</th>
<th>MEASURED TOTAL LOAD IN EYEBARS 11-13 IN POUNDS</th>
<th>MEASURED STRESS IN MODEL EYEBARS 11-13 IN FPSI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load</td>
<td>67,200</td>
<td>68,500</td>
<td>36,600</td>
</tr>
<tr>
<td>Live Load</td>
<td>22,950</td>
<td>23,400</td>
<td>12,500</td>
</tr>
<tr>
<td>Temp.</td>
<td>2,380</td>
<td>2,430</td>
<td>1,300</td>
</tr>
<tr>
<td>Total</td>
<td>92,530</td>
<td>94,330</td>
<td>50,400</td>
</tr>
<tr>
<td>Yield (Theoretical)</td>
<td>137,600</td>
<td>140,400</td>
<td>75,000</td>
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</table>

NOTE:

Total area of model eyebars 11-15 were calculated as $2 \times 2.4 \times 0.39 = 1.872$ in$^2$
C13N (Far Side)
See Figure 2

Fig. 1 Elevation View of Point Pleasant Bridge
Fig. 2 Location of Joints C13, U7, and U13
Typical eyebar chain joint for portions of the eyebar chain where the chain was not framed into truss members. Note hanger strap and strap plates in the center of photograph. The strap plate was connected to the top chord of truss members vertically below the eyebar joint. This plate shows makeup of joint C13 north.

Fig. 3 Details of Joint C13
Detailed diagram of construction of joint U7 of the north truss of the Ohio side span showing the joint pin, the two sets of eyebars, gusset plates attaching members of the north truss to the eyebar joint, retaining plate and the retaining pin which was inserted through a concentric hole in the joint pin and which was installed with two nuts on either end of the retaining pin. The makeup of the retaining pin, retainer plates and the use of double nuts on each end of the retaining pin were common to all eyebar joints except at the tops of the two towers and at the connections of the eyebar chain with the chain bent posts.

Fig. 4 Details of Joint U7
Details of connection of Ohio end of lower chord of south truss of Ohio side span into south chain bent post on Ohio side of bridge. Note the square pin connected to vertical chain bent post. The lower chord of the side span truss was permitted to move longitudinally through provision of a slotted section around the square pin to equalize expansion and contraction of the bridge due to temperature changes and changes in live load. The detail shown was that used to connect the lower chords of both trusses of the two side spans into the four chain bent posts in the bridge.

Fig. 5 Details of Joint at Ohio End of the Point Pleasant Bridge
(See Also Fig. 2)
Fig. 6 Point Pleasant Bridge (after collapse)

Fig. 7 One of the Eyebar Joints (after collapse)
Fig. 8 Fractured Cl3N Eyebar

Fig. 9 Fracture Surface of Cl3N Eyebar
Fig. 10 Initial Cracks in Pinhole of Reyebar Cl3N
Fig. 11 Location of Initial Eyebbar Cracks and the Local Hardness of the Material

Fig. 12 Fracture Toughness of Eyebbar Steel
2.2 FATIGUE/FRACTURE ANALYSIS
OF
ILLINOIS RT. 157 OVER ST. CLAIR AVENUE
-- A Brief Summary --

I.D. No. : 36
Town/City/etc.: French Village
Railway/Highway/Avenue: Illinois 157 over St. Clair Avenue
County : St. Clair
State : Illinois
Owner : Illinois
1. DESCRIPTION AND HISTORY OF THE BRIDGE

Brief Description of Structure:

The Route 157 bridge located in St. Clair County, Illinois is a skewed seven span continuous structure 474 ft. - 6 in. long over St. Clair Avenue. It is composed of thirteen rolled WF beams spaced at 5 ft. - 6 in. and varying spans. Spans 2 and 6 are each 100 ft. long with 60 ft. suspended segments supported by cantilevering the adjacent spans. The suspended spans are supported by a rocker bearing on one end and by pin plate links on the other end, as illustrated in Fig. 1. Figure 2 shows a view of span 2. Spans 1 and 7 are 54 ft. - 9 in. long and spans 3, 4 and 5 are 55 ft. - 0 in. long.

The original A7 steel rolled sections were W36X230 in spans 1, 3, 5 and 7. The suspended spans were W36X150 sections as was span 4. Welded cover plates were attached to the bottom flange over the supports at piers 1, 2, 5 and 6. Two additional A36 steel beams were added in 1966 and were W33X200 sections between pier 3 and the north abutment, W36X160 sections between pier 4 and the south abutment. The structure was designed for H20-44 loading. Figure 3 shows a schematic of suspended span 2.

The structure was built in 1945. In 1966 it was widened by adding two beams to the west side of the structure. On October 23, 1978, the pin links supporting the ends of beams (No. 8, 9, 10) in the passing lane of span 2 were found to be fractured as illustrated in Fig. 4. The cracked plates were discovered by a district employee driving beneath the structure. The beams had dropped between 1/2 to 3/4 in. below the deck slab. Span 6, which is identical to span 2, was found to be structurally sound with no evidence of cracks in the hangers.
2. FAILURE MODES AND FAILURE ANALYSIS

Location of Cracks, Failure Modes and Stages

Figure 3 shows the locations of the fractured hanger plates in span 2. In addition to the hangers supporting beams 8, 9 and 10, seven additional beams had plate hangers which were either bowed or partially cracked, and these are also identified in Fig. 3.

The cause of the failures appears to be due to rust and other corrosion product which builds up between the beam web reinforcement and the hanger plates. Since the hangers were located beneath the expansion joint finger plates, water, salt and other debris came in contact with the pinned connections of the hanger plates. This resulted in a freezing of the lower pins. Hence, rigid joints developed with passage of time.

The frozen joints subjected the hanger plates to large in-plane bending stress as a result of traffic and thermal expansion. Measurements on other bridges with suspended span hangers have confirmed that the relative end rotation that is assumed to occur results in large bending stresses when frozen hanger plates exist. The repeated loading resulted in cracking and eventual failure of the 3/4 in. x 7 in. pin plates.

Cyclic Loads or Stresses (Known or Field Recorded)

No measurements or design stresses were available for this structure.

The average daily truck traffic (ADTT) for 1980 was 300. The total daily traffic for 1979 was 18,600. The average daily truck traffic results in 110,000 cycles of random loading per year. Hence, about 3.5 million cycles of repeated load were experienced by the structure.
Temperature and Environmental Effects (Known or Field Recorded)

No temperature data was available. However, it seems likely that the thermal expansion of the structure resulted in large bending stresses in the hanger plates.

The slant fractures of the hangers do not suggest any evidence of cleavage fracture (see sketch in Fig. 3).

Material "Fracture, Mechanical and Chemical Properties"
in the Failure Regions

No tests were performed on the hangers.

Visual and Fractographic Examination of Failure Surfaces

Visual inspection of the cracked hangers revealed a 45° failure surface, as shown schematically in Fig. 3. As can be seen in Fig. 4, corrosion product exists at the cracked area. No fractographic examination was carried out.

Crack Growth (Initiation and Propagation) and Probable Causes of Failure

The primary cause of failure was the development of frozen pin joints as a result of corrosion product. The hanger and girder web reinforcement became rigidly attached as a result of the water, salt and other debris to which the hangers were exposed. This prevented the links from rotating and accommodating the repeated loading from traffic and thermal expansion. The cracks developed in the gross section at the edge of the frozen joint.
The corrosion product effectively welded the hanger to the web and prevented the net section from becoming critical. The fracture surface of the hanger from Beams 8 and 9 appeared to be "very old." The hangers on Beams 5, 6, 7 and 11 were all bowed.
3. CONCLUSIONS/RECOMMENDATIONS

Conclusions Regarding Fracture/Failure

The cracked hangers in the Ill. 157 bridge appear to be repeated load induced as a result of the frozen pin connection. The suspended span end displacements subjected the hanger to high cyclic in-plane bending stresses from traffic and thermal conditions. The resulting cumulative damage caused cracks to form and propagate at the edge of the corrosion "welded" region.

The highest thermal expansion and contraction stresses were probably induced during the periods June-August and December-February. This corresponds to the highest and lowest temperature periods. Final failure may have occurred during the winter period when the material toughness would be at its lowest level.

Recommendations for Repair/Fracture Control

The northbound passing lane was closed, and Beams 8, 9 and 10 were shored, as illustrated in Fig. 5 until the broken hangers were replaced. Fourteen additional hangers were replaced during subsequent maintenance operations. The hangers on the beams which were added in 1966, were inspected and found to be satisfactory. No evidence of cracking was observed.
4. ACTUAL REPAIR/FRACTURE CONTROL EFFORTS

The cracked and defective hangers were replaced by Illinois Department of Transportation. Beams 7, 8 and 9 were jacked back into place, and new hangers and pins installed. All corrosion product was removed, and all of the pins were freed up.

5. POSSIBLE ADVERSE EFFECTS OF REPAIR

No adverse effects are expected as a result of the repair procedures which were carried out. Other cracks should be prevented from developing as long as the hangers function as intended.

6. ACTUAL OR ESTIMATED REPAIR/REHABILITATION COSTS

- Cost of Investigation

No investigation was carried out.

- Cost of Repair/Rehabilitation

The cost of repairing the cracked and defective hangers with Illinois Department of Transportation personnel was $10,000, including traffic control.

- Original (or Adjusted) Cost of the Bridge

The original cost of the structure was $120,836.96 in 1945. The cost of widening the structure in 1965 was $208,199.20.
TYPICAL SECTION

Fig. 1 Typical Hanger Section for Suspended Spans 2 and 6

Fig. 2 View Showing Suspended Span 2 over St. Clair Avenue
Fig. 3 Schematic of Suspended Span Showing Locations of Cracked Hangers
Fig. 4 View Showing One of the Cracked Hangers in Span 2.

Fig. 5 Steel Shapes Shoring up Cracked Beams in Span 2
3. COVER-PLATED BEAMS AND FLANGE GUSSETS

The Yellow Mill Pond multibeam structures (I.D. 3) at Bridgeport, Connecticut have developed extensive fatigue cracks at the ends of cover plates. A summary of this cracking is provided hereafter. These cracks resulted from the large volume of truck traffic and the unanticipated low fatigue resistance of the large sized cover-plated beam members (Cat. E').

The King's Bridge in Australia (I.D. 1) is well known for its failure as a result of fatigue cracking and fracture from very large weld toe cracks that developed during fabrication in all four girders.

A third structure experienced cracking at the cover plate end of one beam in New Jersey (I.D. 44).

In addition to welded cover plates, several other Category E or E' details have experienced fatigue cracking at weld terminations. These include gusset plates welded to the flange and the termination of longitudinal stiffener welds at transverse stiffeners or connection plates. Three structures have exhibited cracking at the end of flange gusset plates (I.D. 39, I.D. 66, and I.D. 85).

The crack that formed in the Vermillion River Bridge, originated at the end of a longitudinal fillet weld that was used to attach a 3 ft. (.9 m) long lateral connection plate to the edge of a 8 x 8 x 1 in. (20 x 20 x 3 cm) flange angle. The gusset plate lapped over the flange angle surface and was welded along both edges. An 18 in. (46 cm) wide cover plate was also welded to the flange angles.
Two structures have had cracking at the end of longitudinal stiffeners that intersect transverse welded plates (I.D. 65 and I.D. 109).
3.1 FATIGUE/FRACTURE ANALYSIS
OF
YELLOW MILL POND BRIDGE
-- A Brief Summary --

I.D. No. : 3
Town/City/etc. : City of Bridgeport
Railway/Highway/Avenue/etc. : Interstate I-95
County : Fairfield County
State/Province : Connecticut
Owner : Connecticut-DOT
1. **DESCRIPTION AND HISTORY OF THE BRIDGE**

   **Brief Description of Structure:**

   The Yellow Mill Pond Bridge carries Interstate I-95 (Connecticut Turnpike) over the Yellow Mill Channel (an extension of Bridgeport Harbor on Long Island Sound) is located in the City of Bridgeport, Connecticut.

   The simple spans of the Yellow Mill Pond complex were designed considering composite action between the rolled cover-plated steel beams and 7.25 in. (184 mm) reinforced concrete slab. In 1969 a 2 in. (51 mm) bituminous concrete overlay was placed on the concrete deck.

   The bridge complex consists of fourteen consecutive simple span cover-plated steel and concrete composite beam bridges crossing the Yellow Mill Pond Channel (fourteen bridges in each direction of traffic) (see Fig. 1). Each bridge carries three lanes of traffic. The typical position of the main beams and diaphragms of the structure are given in the plan and cross-section of span 10\(^(*)\) in Figs. 2.a and 2.b. The external facia beam (M88) of the eastbound bridge is skewed as four lanes of through traffic are being reduced to three lanes.

   In order to accommodate the Yellow Mill Pond Bridge structure to tidal shipping channel clearances, cover-plated beams were employed by the designers in lieu of deeper built-up girders. In the span 10, all beams are W36 A242 steel sections. The tension flanges of all beams except (M86) interior facia beam of the eastbound roadway are fitted with

\[(*)\] This is the typical span at which field strain measurements were recorded and stress-range histograms developed (see also Ref. 1).
multiple cover plates (primary and secondary). The compression flanges have single cover plates. All cover plates are partial length with the exception of the primary plates of the external facia tension flanges of (M88) and (M83) which extends to full length. The cover plate ends are square, and their corners are rounded to a 3 in. (76 mm) radius of curvature (for the cover plate details, see Figs. 3 and 4).

Brief History of the Structure and Cracking:

The Yellow Mill Pond Bridge was constructed in 1956-1957. It was opened to traffic in January 1958. In October-November 1970, the steel superstructure of the Yellow Mill Pond Bridge were inspected following the cleaning and repainting done by the contractor. On November 2, 1970, during a routine inspection of the repainting workmanship, it was discovered that a crack had developed in the eastbound roadway in span 11. The crack started at the west end of the cover-plated beam and had propagated through the flange up to 16 in. (400 mm) into the web of one of the main girders (girder No. 4, span 11). Figure 5 shows a photograph of the beam as it appeared shortly after the fracture was discovered. The crack originated at the toe of the transverse fillet weld connecting the cover plate to the tension flange of the beam (see also Figs. 6.a and 6.b).

Inspection of fifteen similar locations on different girders on the same bridge revealed the possibility of incipient cracks of varying magnitude along eight of the cover-plate end welds. The two beams adjacent to the fractured beam had fatigue cracks which extended halfway through the tension flange at the end of cover plates. These were Beams 3 and 5.
in Span 11 of the eastbound roadway. The cracks in Beams 3 and 5 were inspected by ultrasonic testing and a crack depth of 0.625 in. (16 mm) was measured.

In December 1970, following the detailed inspection, a section of fractured girder was removed, and all three damaged girders were repaired with bolted web and flange splices. In November 1973, the east end of Beams 2 and 3 of the eastbound roadway of Span 10 were inspected again. Fatigue cracks were observed in both girders at the toe of the primary cover plate transverse fillet welds. A crack depth of 0.275 in. (10 mm) was measured in Beam 2, and its presence was verified by the magnetic crack definer.

In June 1976, the cover plate details in the eastbound and westbound lanes of Span 10 were inspected for fatigue cracking by visual, dye penetrant and ultrasonic inspection techniques. Twenty-two out of forty details were found to have cracks. The smallest visually observable crack was 0.25 in. (6 mm) long. Ultrasonic inspection provided confirmation of cracks about 0.5 in. (13 mm) long at the west end of the eastbound lane in Beams 3 and 7 of Span 10. The results of the inspection in terms of the crack depth and locations are given in Refs. 1 and 5.
2. FAILURE MODES AND FAILURE ANALYSIS

   Failure Modes, Stages and Location of Failure:

   In November 1976, during a brief inspection of Span 13 with field glasses from the ground level, four large cracks were observed. These cracks were approximately 6-10 in. (150-250 mm) long and about 0.5 in. (13 mm) deep. They had broken the paint film at the weld toe (see Fig. 6a).

   In September 1977, span 10 was inspected again. The secondary details at Beam 5 of the westbound bridge, and the secondary details of Beams 5 and 6 of the eastbound bridge at the east end were inspected for the first time. No cracks were observed. Small cracks were found at the west end of the secondary details of Beams 4, 5 and 6 of the eastbound bridge. (These details were previously inspected in 1976, but no cracks were found at that time). In addition, several cracks were found in details in Span 12. The cracks were up to 15 in. (381 mm) long.

   In November 1979, several details in Span 10 were inspected again by visual and ultrasonic means. The ultrasonic inspection verified the crack found during the 1977 inspection. The secondary detail on Beam 4 of the eastbound bridge near the west end was cracked. The retrofitted details were also inspected, and only one crack was detected, which was growing from the root of the primary detail on the west end of Beam 2 of the eastbound structure. The crack was 1.25 in. (32 mm) on the surface, and ultrasonic inspection indicates 6 in. (152 mm) length at the root. Small cracks were found for the first time at the west end of the secondary detail on Beam 2 of the eastbound bridge.

   The history of inspection of span 10 and the location of the cracks detected or found each time are summarized in Table 1. Figure 2a shows the location of these details.
• Cyclic Loads or Stresses (Known or Field Recorded)

The eastbound and westbound bridges of Span 10 were selected for two major stress history studies. Both of these studies were conducted by the State of Connecticut (CONN-DOT) and the Federal Highway Administration (FHWA). The first study was conducted in July 1971 (2), and the second between April 1973 and April 1974 (3).

In the July 1971 study two electrical resistance strain gages were placed on the interior beams. One gage was placed midspan and the other 4 in. (102 mm) from the end of the primary cover plate (the gages were either on the bottom flange or on primary cover plates). A typical stress histogram for the gage on a girder under the eastbound lanes is shown in Fig. 7. A typical stress histogram is given in Fig. 8 for the westbound lanes. The truck distribution and its position in the lanes were also concurrently recorded and their weights were sampled (2).

It can be seen from stress-range histograms of the eastbound bridges that 94-99% of the stress ranges occurring at the cover plate ends fell within 0.60 - 1.95 ksi (4.137 - 13.445 MPa) levels. Very few events were found to have cyclic stresses in excess of 2.40 ksi (16.548 MPa). A single event exceeded 3.3 ksi (22.753 MPa). At the midspan gages, 97 - 99% of all recorded stress ranges fell within the limits 0.60 - 2.85 ksi (4.137 - 19.650 MPa). Thirty-five events exceeded 3.0 ksi (20.684 MPa). The overwhelming majority of midspan stress ranges were below 3.6 ksi (24.821 MPa). A large event equal to 7.2 ksi (49.643 MPa) occurred in only one case. The gross truck weights were fairly evenly distributed between 10,000 and 70,000 lb. with the maximum weights recorded between
90,000 and 100,000 lb. The distribution of truck traffic indicated that approximately 55 percent of the trucks were in the outer lane and 45 percent in the middle lane. Less than 1 percent were in the inner lane. The composition of the average daily truck traffic (ADTT) at the Yellow Mill Pond Bridge was about the same as the gross vehicle weight distribution developed from the 1970 FHWA nationwide Loadometer survey. The one-way average daily traffic (ADT) and the average daily truck traffic (ADTT) on span 10 is plotted in Fig. 9 for 1958 through 1975. From January 1958 to June 1976 approximately 259 million vehicles have crossed Span 10. Approximately 13.5% of this total traffic flow is truck traffic. Therefore, approximately 35 million trucks crossed the eastbound and westbound bridges of Span No. 10 during the interval between 1958 and 1976.

The output of a gage on the diaphragm produced some of the most surprising results within 63 hours, the bottom flange at midspan of the diaphragm was stressed to 3.0 ksi or more 2400 times. This included 27 stress cycles greater than 5.4 ksi. The only explanation for this phenomenon must be related to the rotation of the main girders. Rotation of the internal beams are restrained by the diaphragm at their points of intersection with the girders unless some lateral movement of the latter is permitted. This might explain the high incidence of bolt failures in these connections.

During the second study, from April 1973 to April 1974, the stress history of the eastbound bridge of span 10 was monitored by mechanical strain recorders that were attached at midspan to Beams 3 and 4.
A comparison of the results with strains recorded at the midspan of Beams 3 and 4 during July 1971 indicates that slightly higher stress range events occurred in 1973 as compared to the 1971 study.

Also, a limited strain history record was obtained for the eastbound bridge of span 10 in June 1976.

The Miner and root-mean-square (RMS) effective stress ranges for the July 1971 study do not differ as much as implied by the June 1976 study.

Temperature and Environmental Effects (Known or Field Recorded):

The material toughness for the Yellow Mill Pond Bridge girders satisfied the requirements for AASHTO temperature Zone 1. The transition temperature at 15 ft-lb. (20 joules) for the beam flange was 55° F (13° C) as shown in Fig. 10. The transition temperature diagram is given in Fig. 11 for the web. The web of the cracked beam showed evidence of some cleavage fracture. This rapid crack growth may have occurred in September 1970 when a large overload is believed to have crossed the bridge.

In spite of the lower temperatures [i.e. less than 0° F (-18° C)] the adjacent girders with sizeable fatigue cracks withstood these conditions.
Material "Chemical, Mechanical and Fracture Properties"

Analysis in Failure Regions:

In December 1970 after a detailed inspection, following the discovery of cracks, a portion of the flange and web adjacent to the crack was removed for testing.

The chemical analysis of the material from the web and flange of the W36X300 rolled steel beam indicated that excessive manganese was present. The chemistry check based on ASTM A.242 specifications yielded 1.69% manganese which is above the limit value 1.45%. All other chemical elements appear to be satisfactory and within tolerance limits (4).

Mechanical tests of the flange and web material provided a 57.8 ksi (398.519 MPa) yield point for the flange and a 95.1 ksi (655.695 MPa) tensile strength. The web material had a yield point of 57.2 ksi (394.383 MPa) and a tensile strength of 87.5 ksi (603.295 MPa). Both flange and web exhibited satisfactory elongation characteristics (i.e. 28%).

The fracture characteristics of the web and flange material was assessed using Charpy V-notch impact tests. These tests revealed an interesting difference in transition temperature of the flange and web material. The flange material provided a +55° F (13° C) transition temperature at 15 ft-lbs. (20.337 joules), whereas the web provided -10° F (-23° C) transition temperature. This is most likely due to the excess manganese and increased thickness of the flange material. Transition curves for both flange and web material are given in Figs. 10 and 11. Compact tension tests were also carried out, and these provided 1 sec. fracture toughness values of about 80 ksi $\sqrt{\text{in.}}$ at -10° F.
Visual and Fractographic Examination of Failure Surfaces:

The crack surfaces in the web and flange were subjected to visual examination in order to evaluate crack growth regions. Three distinct areas or regions were noted. The fracture surface examinations indicated that fatigue crack growth started at the weld toe and grew completely through the flange and into the web about 2 in. (50.8 mm). The crack path changed when it encountered a lamination condition near the center of the flange. The lamination retarded the advancement of the path.

Near the edges of the beam flange, the crack path moved into a second crack growth region away from the weld toe. At this stage, tensile fracture partially occurred as evidenced by some apparent necking. The fracture surface in the web characteristically indicates that rapid crack growth developed after a 2 in. (50.8 mm) fatigue crack had penetrated into the web. The crack extended about 7 in. (177.8 mm) up the web and was finally arrested near middepth. The fracture surface suggested a "brittle fracture" mode in the web. There was also some evidence of shear lips at the surface of the web plate. It is highly probable that the rapid crack extension occurred in September 1970 when a large overload crossed the bridge.

A third region of apparent slow crack growth which likely occurred after the large overload was also observed.

The fracture surface was coated with oxide and paint. This suggested that the crack existed for several months prior to its discovery on November 2, 1970.
3. CONCLUSIONS AND RECOMMENDATIONS

Conclusions Regarding Fracture/Failure:

The field observations on cracking of the Yellow Mill Pond Bridge yielded the following conclusions:

a. Significant fatigue crack growth can occur at welded cover plate ends when a few stress cycles in the variable stress spectrum exceed the constant cycle fatigue limit and a large number of stress cycles accumulate ($\geq 10^7$).

b. It is possible that other comparable cover-plated beam bridges (with the flange thickness $t_f > 0.8$ in. (20 mm) will require retrofitting in the future if subjected to high volume truck traffic.

c. The observed field behavior at the Yellow Mill Pond Bridge is compatible and corroborated by the results of laboratory tests on full scale cover-plated beams.
Recommendations for Repair/Fracture Control

Peening and gas tungsten arc remelting procedures were recommended and used to retrofit the cover-plated beams in Span 10 of the Yellow Mill Pond Bridge. Twenty-five of the cover plate details in Span 10 were treated. Of these, fourteen were peened and eleven were gas tungsten arc (GTA) remelted. Figure 12 shows a transverse fillet weld after the gas tungsten retrofit at each original cover plate weld. All retrofitting was carried out under normal traffic with the dead load in place. A peened weld toe at Yellow Mill Pond is shown in Fig. 13. The depth of indentation due to peening was approximately 0.03 in. (0.8 mm)

All cracks that exceeded 1-1/2 in. length along the weld toe were spliced with bolted butt splices. Holes were placed in the girder web directly above the crack. Figure 14 shows typical bolted joints used to splice across the crack locations.

4. POSSIBLE ADVERSE EFFECTS OF REPAIR

No adverse effects of repair and retrofit are expected in the case of Yellow Mill Pond Bridge. The end welds of the cover plates should be checked periodically in order to insure the integrity of the cover-plated girders.
5. ACTUAL OR ESTIMATED REPAIR/REHABILITATION COSTS

   Cost of Investigation:
   This is not available as a separate item.

   Cost of Repair Rehabilitation:
   The cost of the retrofit for the Yellow Mill Pond Bridge is given by CONN-DOT as $28,825.00 for the original bolted splices.

   Original Cost of the Bridge:
   The original cost was $4,944,333.00 in 1958.

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<td>X</td>
<td>CV</td>
<td>X</td>
<td>NC</td>
<td>CV</td>
<td>NA</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

# - Beam No.  
E - East end of beam  
W - West end of beam  
P - Primary cover plate detail  
S - Secondary cover plate detail  
NA - Not applicable  
R - Root Crack  
X - No inspection  
NC - No indication of cracking  
C - Visual indication of cracking  
CV - Crack verified by ultrasonic inspection
Fig. 1 Plan and Elevation of the Yellow Mill Pond Bridges
Fig. 2a Plans of Inspected Details in Span No. 10, Yellow Mill Pond
Intermediate diaphragms shown

End diaphragms shown at pier

Fig. 2b Typical Expressway Cross-Section (Eastbound)
Fig. 3 Details of Beam and Strain Gage Location
Fig. 4 Cover Plate Detail at Yellow Mill Pond
Fig. 5 Cracked Girder 4 in Span 11
Eastbound of Yellow Mill Pond
Fig. 6a Typical Large Crack at Weld Toe at End of Cover Plate

Fig. 6b Smaller Crack at Weld Toe of Cover Plate
Fig. 6.c Crack Shape for Beams 3 and 7, 1976 (Eastbound Span 10)
Fig. 7 Stress Range Histogram, 1971 Eastbound Span 10

Fig. 8 Stress Range Histogram, 1971 Westbound Span 10
Fig. 9 One-Way ADT and ADTT on Span 10, 1958-1975
Fig. 10 Charpy V-Notch Test Data for Beam Flange

Fig. 11 Charpy V-Notch Test Data for Beam Flange
Fig. 12 Gas Tungsten Arc Remelted Weld Toe

Fig. 13 Peened Weld Toe
Fig. 14 Retrofit Details of Bolted Splices
4. WEB CONNECTION PLATES

Cracks that have developed in the web at lateral connection plates have generally occurred as a result of intersecting welds. One of the first bridge structures to exhibit this cracking was the Lafayette Street Bridge over the Mississippi River at St. Paul, Minnesota. A summary of the crack development is provided hereafter (I.D. 6). The primary problem was the large defect in the weld attaching the lateral connection plate to the transverse stiffener. Since this weld was perpendicular to the cyclic stresses, and the weld intersected with the vertical welds attaching the stiffener to the web and the longitudinal welds of the connection plate, a path was provided into the girder web.

Similar cracks formed in at least five other bridge structures with similar intersecting welds (I.D. 39, 51, 61, 87). In two of these cases the cracks were discovered before the girder flange was cracked in two.

Cracking was also observed in two other bridges where the lateral connection plates were installed on each side of the transverse stiffeners (I.D. 33 and 63). This often provided an intersecting weld condition between the transverse stiffener welds and the longitudinal welds attaching the connection plates to the girder web. Since no connection was provided between the lateral connection plate and the transverse stiffeners, out-of-plane movement occurred in the web gap or at the intersecting welds. The resulting high cyclic stress conditions in the web caused vertical cracks to form in the web. This mode of cracking is shown in Fig. 58 of Ref. 2.
4.1 FATIGUE/FRACTURE ANALYSIS

OF

LAFAYETTE STREET BRIDGE

-- A Brief Summary --

I. D. No. : 6
Town/City/etc. : St. Paul
Railway/Highway/Avenue/etc. : Lafayette Street
State/Province : Minnesota
Owner : Minnesota DOT
1. DESCRIPTION AND HISTORY OF THE BRIDGE

Brief Description of Structure:

The Lafayette Street Bridge spans the Mississippi River in St. Paul, Minnesota. The main channel crossing consists of two parallel structures composed of two main girders extending three spans over the Mississippi River with the central main span (Span 10) of 362 ft. (110 m) and side spans of 270 ft. (82 m) and 250 ft.-6 in. (76 m) (see Fig. 1) corresponding to Spans 9 and 11, respectively. These continuous main girders extend beyond piers 8 and 11 as 40 ft. (12 m) cantilevers. The transverse cross-section consists of two main girders connected by transverse floor beams and by transverse bracing, as shown in Fig. 2. The transverse floor beams support two stringers.

The web and flanges of the main girders were fabricated from ASTM A441 steel.

Brief History of Structure and Cracking:

The Lafayette Street Bridge was opened to traffic on November 13, 1968. The southbound lanes were closed between May 20, 1974 and October 25, 1974 for repairs to the deck and overlays.

On May 7, 1975 a crack was discovered in the east main girder of the southbound structure in Span 10 (see Fig. 2). Figures 3, 4 and 5 show the crack developed in the main girder web and the lateral bracing gusset plates.
2. FAILURE MODES AND FAILURE ANALYSIS

- Location of Fracture Failure Mode(s) and Stages:

The crack occurred in the "east" girder of the southbound lane in Span 10, 118 ft. - 8 in. (36 m) from pier 10. The web crack had propagated to within 7.5 in. (19 cm) of the top flange when it was discovered on May 7, 1975. The entire bottom flange was fractured (see Figs. 3 and 7).

A detailed study\(^1,2\) and visual examination of fractured portions of the girder web, flange and the gusset plate of transverse bracing indicated that fatigue crack growth originated in the weld between the gusset plate and the transverse stiffener as a consequence of a large lack of fusion discontinuity in this location (see Figs. 3, 5, 6 and 8). The fracture surface of the web shows that a brittle or cleavage fracture occurred after the fatigue crack propagated into the web through the gusset-stiffener weld. As can be seen in Fig. 7, the cleavage fracture of web continued and also extended down into the bottom flange, and consequently the entire bottom flange was broken (see also Fig. 8).

- Cyclic Loads and Stresses (Known or Field Recorded):

The estimated average daily truck traffic (ADTT) crossing the bridge was 1500 vehicles during the period November 1968 - May 1975. Thus, approximately 3,300,000 trucks crossed the bridge prior to the time the crack was discovered.

Stress range histograms for the main girders and other structural members are not available. However, the root-mean square stress range was...
approximated from the gross vehicle weight distribution in Ref. 1. A standard HS20 truck is used to generate a moment range in the main girder which varied from +3.814 ft-kips (+5.172 kNm) to -1.622 ft-kips (-2.199 KNm). This results in a stress range of 4.68 ksi (32 MPa) in the girder flange and 4.13 ksi (28 MPa) at the gusset-web connection. A few strain measurements were acquired on the main girders near the fracture during passage of an HS 20 type truck and resulted in a stress range of about 2 ksi (14 MPa).

Temperature and Environmental Effects (Known or Field Recorded):

During the winter prior to May 7, 1975 the temperature at St. Paul, Minnesota was recorded at -8° F (-22° C) on March 13, 1975 and on February 9, 1975 the temperature in St. Paul reached -22° F (-30° C).

Material "Fracture, Mechanical and Chemical Properties" in the Failure Regions:

Four pieces (i.e. from I to IV, as shown in Fig. 3) of material from the flange, web and gusset plate were removed and tested (see Refs. 1,2). Charpy V-notch tests, tensile tests and chemical analysis were performed. The results are given in Fig. 9 and Tables 1 and 2. In addition, a few compact tension tests (KQ) were conducted on the web plate.

The web plate was observed to satisfy the fracture toughness requirements of the 1974 AASHTO Specifications for Temperature Zone 2 [15 ft-lbs. (20 Nm) was reached @ 15° F (-9° C) for the web material]. The flange and the gusset plates, however, met the requirements of Temperature Zone 1.
The flange and web plate show correspondence with the requirements of A441 and the gusset plate with A36 steel.

Visual and Fractographic Examination of Failure Surfaces:

The fracture surfaces of part of the web, flange, and gusset plate are given in Fig. 5, 6 and 7. From these photographs, it can be seen that the transverse and longitudinal single bevel groove welds that connect the gusset plate to the transverse stiffener and web did not penetrate to the backup bar on the root of the weld preparation. This resulted in a significant lack of fusion.

Further visual examination of the fracture surface indicated that fatigue crack growth initiated in the weld between the gusset plate and the transverse stiffener as a consequence of the fusion discontinuity mentioned above. The fracture surface of the weld indicates that mainly a cleavage or brittle fracture occurred in the web after the crack had propagated into the web through the gusset-stiffener weld. This cleavage or brittle fracture extended down the web and into the flange (Refs. 1,2).

Fractographic examination of failure surfaces at several locations by the transmission electron microscope corroborated the conclusions of the visual examinations (Ref. 1,2).

Crack Growth (initiation and propagation) and Probable Causes of Fracture/Failure:

The primary cause of failure (Ref. 1,2) is the large lack of fusion discontinuity in the weld between the gusset plate and the transverse stiffener near the backup bars. This resulted in a fatigue crack initiation
in the web at the point of the gusset-stiffener-web intersection which later created essentially brittle fracture in part of the web and the whole cross section of bottom flange of the "east" girder. Probably, the brittle fracture of the web occurred during either one of the recorded cold days on March 13, 1975 and February 9, 1975. Thus, both fatigue and brittle fracture cracks were observed. The crack surfaces showed several stages of crack growth (see also Ref. 1,2).

Final fracture of the web occurred as further fatigue crack growth and yielding of the gusset plate occurred which permitted the crack to open. This eventually led to the final girder fracture.

3. CONCLUSIONS/RECOMMENDATIONS

- Conclusions Regarding Fracture/Failure:

The fracture of the "east" main girder of the southbound lane of the Lafayette Street Bridge was due to the formation of a fatigue crack in the lateral bracing gusset plate to transverse stiffener weld. This fatigue crack originated from a significant lack of fusion region near the back-up bars. At the failure gusset, this groove weld intersected the transverse stiffener-web weld and thus provided a direct path for the fatigue crack which penetrated the web. Subsequently this fatigue crack precipitated a brittle fracture of part of web and all of the tension flange (see Ref. 1,2).

- Procedure used for Repair/Fracture Control:

If other fatigue cracks are removed or properly arrested, the material used in this structure will provide adequate resistance to brittle fracture. It is very probable that other discontinuities due to lack of fusion exist in
other similar transverse stiffener-gusset plate welds in the two bridges. Therefore, all gussets located in the regions of cyclic stress range and tensile stress should be properly treated to prevent (and to detect) any fatigue crack growth into the girder web\(^{(1,2)}\).

The scheme used to retrofit the structure is given in Fig. 10. Two vertical holes 1.25 in. (32 mm) were cut into the gusset plate and ground smooth at the corners of the gusset-transverse stiffener-web interface. Additionally, two smaller vertical holes were drilled through the gusset plate and ground smooth at the tips of groove welds which join the gusset plate to the transverse stiffener. In the first set of holes [1.25 in. (32 mm)], the web surface and the gusset should be carefully inspected by a dye penetrant after the material is removed. If no crack indication is observed, then the detail can be regarded as adequate. Otherwise, additional holes were to be cut into the girder web with a 1.25 in. (32 mm) hole saw on the two diagonal lines from opposite sides of the web surface, as shown in Fig. 10. After these plugs are removed, the holes should be ground smooth and the exposed web surface checked with dye penetrant to insure that the web crack does not extend beyond the holes. The holes should be ground smooth before the area is painted.

A suggested redesign for the type of the "gusset-transverse stiffener-web" connection is given for future use in Refs. 1 and 2.

4. \textbf{POSSIBLE ADVERSE EFFECTS OF REPAIR}

No adverse behavior of importance is expected. Some cracking may develop in the isolated transverse welds between the holes in the gusset
After the repairs there was a slight sag observed in the south facia girder at the location of the original fracture (Ref. 3). According to the Minnesota-DOT the repair did not have any significant effect on the load-carrying capacity of the structure (Ref. 3).

5. **ACTUAL OR ESTIMATED REPAIR/REHABILITATION COSTS**

   **Cost of Investigation of Fracture:**

   The cost of investigation of the brittle fracture developed in the girders of the Lafayette Street Bridge is estimated by Minnesota-DOT as $14,620.00 (Ref. 3).

   **Cost of Repair/Rehabilitation:**

   The total cost of repair/rehabilitation, excluding the costs for investigation, is given as $161,120.17 (Ref. 3).

   **Original or Adjusted Cost of the Bridge:**

   The original cost of the bridge was approximately $4,750,000.00 according to the Minnesota-DOT (Ref. 3).

6. **REFERENCES**

   (1) Fisher, J. W., Pense, A. W. and Roberts, R.  
    "Evaluation of Fracture of Lafayette Street Bridge," Proceedings, ASCE,  

   (2) Fisher, J. W., Pense, A. W. and Roberts, R.  
    "Investigation and Analysis of the Fractured Girder in Bridge No. 9800,  
    T.H. No. 56 Over Mississippi River in St. Paul, Minn.," Minnesota  
    Department of Highways (Materials, Research and Standards), October  
    1975.

   (3) Personal Communication from K. V. Benthin, Bridge Engineer,  
    Minnesota-DOT to J. W. Fisher, Lehigh University, dated March 11,  
    1980.
### TABLE 1.—Physical Properties

<table>
<thead>
<tr>
<th>Property (1)</th>
<th>Flange (A441)</th>
<th>Web (A441)</th>
<th>Gusset (A36)</th>
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<tbody>
<tr>
<td>Thickness, in inches (millimeters)</td>
<td>2 1/2 (64)</td>
<td>1/2 (13)</td>
<td>1/2 (13)</td>
</tr>
<tr>
<td>Yield stress, in kips per square inch (meganewtons per square meter)</td>
<td>46.0 (317)</td>
<td>53.7 (370)</td>
<td>37.9 (261)</td>
</tr>
<tr>
<td>Tensile strength, in kips per square inch (meganewtons per square meter)</td>
<td>76.2 (525)</td>
<td>81.8 (564)</td>
<td>67.0 (462)</td>
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<tr>
<td>Elongation, as a percentage</td>
<td>32.8</td>
<td>30.0</td>
<td>36.4</td>
</tr>
<tr>
<td>Reduction in area, as a percentage</td>
<td>65.8</td>
<td>68.0</td>
<td>67.1</td>
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</table>

### TABLE 2.—$K_{IC}$ Fracture Tests

<table>
<thead>
<tr>
<th>Specimen number (1)</th>
<th>Test temperature, in degrees Fahrenheit (Celsius) (2)</th>
<th>Crack depth, $a$, in inches (millimeters) (3)</th>
<th>$a/w$ (4)</th>
<th>Failure load $P_0$, in pounds (newtons) (5)</th>
<th>$K_{IC}$ in kips per square inch $\sqrt{\text{inch}}$ (meganewtons per square meter per meter$^{3/2}$) (6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$0^\circ$ $(-18^\circ)$</td>
<td>1.20 (30.5)</td>
<td>0.480</td>
<td>5,875 (26,133)</td>
<td>68.0 (74.8)</td>
</tr>
<tr>
<td>2</td>
<td>$-25^\circ$ $(-32^\circ)$</td>
<td>1.20 (30.5)</td>
<td>0.480</td>
<td>6,850 (30,470)</td>
<td>79.3 (87.2)</td>
</tr>
<tr>
<td>3</td>
<td>$-120^\circ$ $(-194^\circ)$</td>
<td>1.23 (31.2)</td>
<td>0.491</td>
<td>4,300 (19,127)</td>
<td>51.3 (56.4)</td>
</tr>
<tr>
<td>4</td>
<td>$-150^\circ$ $(-232^\circ)$</td>
<td>1.24 (31.5)</td>
<td>0.496</td>
<td>4,850 (21,574)</td>
<td>58.6 (64.5)</td>
</tr>
</tbody>
</table>
Fig. 1 The Lafayette Street Bridge (three main spans and neighboring spans)
Fig. 2 Schematic of Span and Cross-Section

Fig. 3 Schematic of Girder Showing Crack and Sections Removed for Examination
Fig. 4 Cracked Girder of Lafayette Street Bridge

Fig. 5 Fracture Surface of Gusset-web Showing Lack of Fusion - Piece II
Fig. 6 Fracture Surface of Gusset to Stiffener Weld and Adjacent Web-Piece

Fig. 7 Fracture Surface of Web-Flange Boundary
Fig. 8 Schematic of Stages of Crack Growth in Gusset, Web, and Flange
Fig. 9 Charpy V-Notch Test Results
Fig. 10 Preferred Procedure for Corrective Action

Cut 1/4 in Holes through Web when any discontinuity is detected in Web
5. TRANSVERSE GROOVE WELDS

Many groove welds placed into bridge structures were not well executed and inspected during the initial applications to welded structures in the late 1940's and 1950's. This lack of quality control resulted in large lack of fusion defects, slag, and other discontinuities which have led to fatigue crack growth and fracture.

At least four different types of groove weld details have experienced crack propagation. One such condition was found to occur in the Aquasabon River Bridge in Ontario (I.D. 5), where a rolled section was haunched by welding in an insert plate. This resulted in a short length of vertical groove weld perpendicular to the stress field. An embedded discontinuity and the poor quality of the detail resulted in crack propagation.

Cracks have been found in the flange and web splices at groove welds in at least four bridges (I.D. 23, 49, 65, 82). The flange groove weld cracks detected in the Quinebaug Bridge appear comparable to those observed in the Aquasabon River Bridge. Fatigue crack growth developed from large unfused region of the weldment.

In two of the structures, cracks were found in the groove welded splices of A514 steel tension members (I.D. 23, 49). In both cases, part of the cracking was traced to weld repairs. Other cracks were found to be related to cold cracking which was apparently not detected at the time of fabrication. These structures were fabricated during the period 1969 - 1973. Only the cracks found in the I24 Bridge (I.D. 23) were found to have experienced fatigue crack growth. No evidence of fatigue crack
extension was detected in the Silver Memorial Bridge (I.D. 49) nor the Fremont Bridge (I.D. 65).

Comparable cracks have been observed in the weld of groove welded cover plates. These welds were used to splice two different thickness plates which were then attached by fillet welds to either rolled sections or to riveted built-up girders. Four different bridges are known to have developed cracks at such details (I.D. 47, 69, 70, 90). A review and summary will be given of the cracking that developed in the Ill.51 Bridge at Peru (I.D. 70).

A third kind of groove welded detail that has experienced cracking occurs at splices in continuous longitudinal stiffeners. These attachments were considered secondary components, and no weld quality control was imposed on the groove weld splice. The first such structure to experience cracking at this detail was the Quinnipiac River Bridge near New Haven, Connecticut (I.D. 4). A summary and review of this structure is provided hereafter. At least three other bridges have developed cracks at such splices (I.D. 12, 30, 95). One of these structures developed a crack during a cold winter prior to being placed into service. In every instance of cracking, no weld quality control was imposed on the longitudinal stiffener splices. Some of these were in tension zones. As a result, cyclic stresses resulted in crack propagation from large defects, and these eventually resulted in fracture.

The fourth groove weld detail that has resulted in cracking is the electroslag weld. The fracture of an I79 bridge flange at Neville Island in 1977(3) has resulted in detection of significant flaws in at least five
other bridge structures. Except for the I79 structure, all discontinuities in the electroslag welded bridges were detected and retrofitted before significant crack propagation developed. The electroslag welds were found to have a lower level of fracture toughness than implied from qualification tests. Furthermore, detection of the fabrication discontinuities was found to be difficult and not reliable. As a result of the I79 failure and these other experiences, electroslag welds are not permitted in tension components of bridge structures\(^{(4)}\).
5.1 FATIGUE/FRACTURE ANALYSIS

OF

AQUASABON RIVER BRIDGE

-- A Brief Summary --

I.D. No. : 5
Town/City/etc. :
Railway/Highway/Avenue: Highway 17 (Trans-Canada Highway)
State/Province : Ontario, Canada
Owner : Ontario Department of Highways
1. DESCRIPTION AND HISTORY OF THE BRIDGE

   Brief Description of Structure

The Aquasabon River Bridge is located on the north shore of Lake Superior on Highway 17 (Trans-Canada Highway) 130 miles (210 km) east of Thunder Bay, Ontario. The structure was completed in 1948. The three-span continuous beam structure has a composite steel beam-reinforced concrete slab. It was designed by the Ontario Department of Highways to accommodate an H20 truck load.

The 200 ft. (61 m) structure has three spans of 52 ft. 5 in. (16 m) 80 ft. (24.40 m) and 51 ft. 2 in. (15.60 m) as shown on Fig. 1. The superstructure consists of four longitudinal 33 in. (84 cm) WF girders connected to 15 in. (384 mm) transverse floor beam stringers. The steel structure supports a 7 in. (178 mm) reinforced concrete deck. The main girders are haunched at both piers and abutments. These haunches were fabricated by cutting the bottom flange from the web fillet and welding a parabolic insert plate into the web which resulted in a 51.25 in. (1.30 m) deep section at the piers and abutments (see Fig. 2).

The main girders were field spliced at two points in the center span 22 ft. (67 m) from each pier as shown in Fig. 2. The splice points were placed at the points of dead load contraflexure. The riveted splice consisted of 5/8 in. (16 mm) flange plates on interior and 1/2 in. (12.7 mm)
plates on the exterior girders. All had two 3/8 in. (10 mm) web plates for shear splice.

The reinforced concrete floor slab was connected to the girders by channel-type shear connectors of 11 in. x 3 in. (275 mm x 75 mm) welded at 1 ft. 6 in. (0.457 m) intervals as shown in Fig. 2.

**Brief History of Structure and Cracking:**

The Aquasabon River Bridge was completed in 1948. In 1963, cracks were discovered at the vertical butt weld detail in three of the six haunch inserts of the north interior main girder as shown in Fig. 2b. (Ref. 1). One of these cracks extended 44 in. (1.12 m) into the girder web along a diagonal line starting from the vertical butt weld detail.

In 1973, the structure was subjected to rigorous testing to determine its safe load-carrying capacity. Prior to the 1973 tests, dye-penetrant and magnetic-particle inspection techniques were used on the repaired and other 21 remaining original welds. As a result, four other weld cracks were discovered. The location of individual cracks are marked on Fig. 1. One of these cracks is shown in Fig. 3.
2. FAILURE MODES AND FAILURE ANALYSIS

Location of Fracture, Failure Modes and Stages:

The fatigue cracks that developed were in the main WF girders and are identified in Figs. 1 and 3 (Ref. 1). These cracks propagated from large initial weld imperfections or inclusions in the short transverse groove welds at the ends of the parabolic haunch inserts in the main girders (see Figs. 3, 4, 6 and 7). Of 24 welded details, seven failed. By 1963, three of the six haunch inserts in the north interior girder had developed cracks. These were repaired by welding cover plates over the cracks. By 1973 four more welded details developed cracks. One of the cracks penetrated 7 in. (178 mm) up the web beyond the 2.5 in. (64 mm) transverse weld and had cracked about 65% of the bottom flange (Ref. 3). With large imperfections residing near the bottom of the web, crack growth developed in the web and then into the flange as a part circular crack, as shown in Fig. 4. The enlargement of this crack extended it into the flange and up the web. After the crack penetrated through the flange, most of the fatigue resistance was exhausted. All cracks were discovered before the flanges fractured, because the details were located near the contraflexure points where the dead load stress was small. The stages of crack growth are indicated in Fig. 4.

Cyclic Loads and Stresses (Known or Field Recorded)

A rigorous field testing of the Aquasabon River Bridge was conducted in 1973. Two fully loaded vehicles each with the gross vehicle weight GVW = 197,000 lb. (89,360 kg) distributed on five axles carrying the minimum payload of 153,000 lb. (69,400 kg) were used. The payload can be placed
on the semitrailer to produce any tandem axle weight up to 110,000 lb. (49,900 kg). These loads were applied in four increments, while stresses in critical areas were monitored. Pier settlements were also measured.

Dynamic strains were recorded with the test vehicle loaded to a gross weight of 91,000 lb. (41,280 kg). The vehicle crossed the bridge at varying speeds between 30 to 40 mph (48 to 80 km/h). Based on the dynamic strain measurements and traffic conditions, a representative stress range histogram \( S_r \) was determined (see Fig. 5).

The Stress Range Histogram was used to estimate the effective root-mean-square stress range \( S_{\text{RMS}} \). If all stress range conditions above 0.85 ksi (5.9 MPa) are considered, this results in \( S_{\text{RMS}} = 1.92 \) ksi (13.2 MPa). The effective stress range using Miner's rule was equal to \( S_{\text{rMiner}} = 1.85 \) ksi (12.76 MPa).

A sample of vehicles crossing the bridge indicated that each vehicle produced from six to seven stress cycles.

**Temperature and Environmental Effects (Known or Field Recorded)**

It appears that temperature and environmental effects were not significant factors in the development of cracks in the short transverse groove welds of the Aquasabon River Bridge. The examination of the crack surface and the weld detail indicated that the cracks were stable fatigue cracks.

This is reasonable considering the low dead load at the inflection points. As a result, the maximum stress was small, and this permitted the development of large fatigue cracks without brittle fracture.
Material "Chemical, Mechanical and Fracture Characteristics"

Analysis on the Failure Regions:

Web cutouts, taken from the weld failure zones as shown in Fig. 6 and 7, were analyzed to determine their chemical composition. The material used in the bridge is A7 steel base material. The chemistry checks provided 0.24% C, 0.74% Mn, 0.09% Si, 0.01% P and 0.03% S. The fracture toughness characteristics of the base material was not determined.

Visual and Fractographic Analysis of Failure Surfaces:

Visual examination of failure surfaces indicated that stable fatigue crack growth developed from imperfections in the transverse groove weld (see Fig. 6). The surfaces examined were coated with oxides suggesting that the crack surfaces were exposed for a considerable period of time. No fractographic examination was carried out with the electron microscope.

Crack Growth (Initiation and Propagation) and Probable Causes of Failure:

An examination of fracture surfaces indicated that the cracks in the Aquasabon River Bridge were stable fatigue cracks. There was no evidence of brittle fracture (Ref. 2). The fatigue cracks originated at large weld imperfections in the short transverse groove welds at the end of the haunch inserts in the main girders as shown in Figs. 2, 3, 4 and 6.

The crack growth (Ref. 2) developed in two stages as shown schematically in Fig. 4. The first stage corresponds to the growth from the large inclusions in the transverse groove welds. The fatigue crack continued
to grow and propagated into the flange and up the web. During the second stage, the crack grew as penny-shaped circular crack through the flange. The crack eventually developed into a through crack in the flange.

For the crack shown in Fig. 6 it was estimated that the effective stress range was $S_{rRMS} = 1.92$ ksi (13.2 MPa) which occurred about 1,340,000 cycles each year. Approximately 3.5 years would be required before the crack penetrated into the bottom flange during the first stage of growth. An addition twenty years would be required before the crack propagated through the thickness of the flange. The results of the analysis are in good agreement with the actual field behavior of these details. The first cracked details were detected in 1963 after fifteen years of service and in all probability had a more severe initial discontinuity than the detail analyzed (Ref. 1).
3. CONCLUSIONS/RECOMMENDATIONS

. Conclusions Regarding Fracture/Failure:

The cracking that developed resulted from large imperfections that were fabricated in the short transverse groove welds. These welds were insufficient length to produce sound welds.

The welded connection which caused the problem was difficult to fabricate. In 1948, equipment for nondestructive inspection was not available. Thus, the weld quality was poor and permitted fatigue crack propagation to develop under the normal stress spectrum that the bridge was subjected to.

This structure demonstrates the necessity to provide sound groove welds perpendicular to the cyclic stress. It also suggests that short transverse groove welds are difficult to produce with sound welds and should be avoided.

. Recommendations Regarding Repair/Fracture Control:

Fatigue cracks in weld details originating at extremely large initial weld imperfections and inclusions were retrofitted as explained in the following section (see also Ref. 1).
4. ACTUAL REPAIR/FRACTURE CONTROL EFFORTS

The fatigue cracks discovered in 1963 in the transverse (or vertical) weld detail in the three of the six haunch inserts of the north interior girder were repaired (Ref. 1). The repairs were made by welding cover plates or insert plates (see Fig. 3). Prior to the load tests in 1973 a through inspection of the repairs and remaining transverse weld details revealed four other weld cracks. The transverse weld area was cut out in a circular shape, and an insert was welded in its place. Where the crack penetrated the bottom flange, it was gouged out and filled with weld material at a slow rate of deposit, by using low hydrogen-coated electrodes. All repaired surfaces were subsequently ground smooth and flush to eliminate any stress concentrations.

5. POSSIBLE ADVERSE EFFECTS OF REPAIR

The probable adverse effects of the original repairs might be the possibility of new imperfections or inclusions fabricated into the repair welds. Installation of bolted flange splices were later applied, as cracks were also observed in the groove welds that connected the flange to the web plate insert. Figure 8 shows the flange splice detail.

This subsequent retrofit should offset and prevent any significant distress from developing in the structure.
6. **Actual or Estimated Repair/Rehabilitation Costs**

   . **Cost of Investigation:**

   According to the Ontario Department of Highways' estimate, the inspection and investigation of the cracking in the Aquasabon River Bridge is $8,200.00 in 1979.

   . **Cost of Repair:**

   The cost of repair and retrofit of the bridge in 1979 was given as $92,141.00.

   . **Original Cost or Adjusted Cost of Structure:**

   This is given as $74,616.00 in 1947 by the Ontario Department of Highways.

7. **References**

   (1) King, J. P. C., Csagoly, P. F. and Fisher, J. W.  
   *FIELD TESTING OF AQUASABON RIVER BRIDGE IN ONTARIO*,  
   Transportation Research Record No. 579, 1976, pp. 48-60.  
   (Also presented at the Transportation Research Board Annual Meeting, January 1975, Washington, D.C.)

   (2) Fisher, J. W.  

   (3) Csagoly, P. F.  
Table 1  Expected Maximum Live Load Stresses  
(Loading = two 177,000 lb. vehicles)

<table>
<thead>
<tr>
<th>Location - Y ref. East Brg.</th>
<th>Expected Stresses, ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Live</td>
</tr>
<tr>
<td>24' - Maximum End Span</td>
<td>-20.1</td>
</tr>
<tr>
<td>44' - 2nd. Haunch Weld Detail</td>
<td>-15.8</td>
</tr>
<tr>
<td>60' - East Pier</td>
<td>-17.4</td>
</tr>
<tr>
<td>80' - Splice Zone</td>
<td>-13.9</td>
</tr>
<tr>
<td>100' - Center Span</td>
<td>-20.6</td>
</tr>
</tbody>
</table>

- ive = Tensile Stress  
- ive = Compressive Stress

Table 2  Summary of Estimated Life with Crack Growth Stages  
(Aquasabon Bridge)

<table>
<thead>
<tr>
<th>Stage 1: Crack Growth Through Web</th>
<th>Initial Crack Size, a1, in.</th>
<th>Stress Cycles ( S_{RMS} = 1.92 ) ksi</th>
<th>Years to Achieve</th>
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</thead>
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<tr>
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<td>0.25</td>
<td>25,360,000</td>
<td>18.9</td>
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<tr>
<td></td>
<td>0.30</td>
<td>8,175,000</td>
<td>6.1</td>
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<td></td>
<td>0.32</td>
<td>4,500,000</td>
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</table>

<table>
<thead>
<tr>
<th>Stage 2: Crack Growth Through Flange</th>
<th>Initial Crack Radius, in.</th>
<th>Stress Cycles</th>
<th>Years to Achieve</th>
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<tr>
<td></td>
<td>1.40</td>
<td>31,700,000</td>
<td>23.6</td>
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<tr>
<td></td>
<td>1.45</td>
<td>26,200,000</td>
<td>19.6</td>
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<tr>
<td></td>
<td>1.50</td>
<td>21,400,000</td>
<td>16.0</td>
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</tbody>
</table>
Fig. 1 Plan and Elevation of Aquasabon Bridge (Crack Locations are indicated with mark X)
SLAB CONNECTION LUGS ON TOP FLANGES
WITHIN LIMITS AS SHOWN

Structural Steel — medium grade.
Rivets 7/8" Ø.
All sliding bunnings ground smooth one face.
Girders placed convex side up to counter deflection.
Do not paint tops of all girders and beams.
All splices to develop 50% of strength of heavier section.
Concrete contractor to leave 4" x 4" x 12" open holes
for anchors.

Fig. 2a Main Girder Haunch Detail, Field Shear Splice and Connector Detail
-- Aquasabon Bridge
Fig. 2b Location of Weld Failure in Main Girder
(see Fig. 3 for a detailed photograph of circled area)

Fig. 3 Typical Crack at Vertical Butt Weld Detail
(Terminal Point of Haunch Insert Detail)
Fig. 4a Details of Weld Inclusion

Fig. 4b Assumed Stages of Crack Growth
(Outer Edge of Weld Inclusion is shown as dark line)
Fig. 5 Stress Range Histogram (Smoothed)
-- Aquasabon Bridge
Fig. 6 Crack Surface at Weld Detail (Inclusion area is the dark area in the photograph)

Fig. 7 Circular Cut-Out Section at Haunch Tip Weld Detail
Fig. 8 Retrofit Detail (Top and bottom reinforcing plates along the haunch and the hole drilled at the terminal point)
5.2 FATIGUE/FRACTURE ANALYSIS

OF

QUINNIPIAC RIVER BRIDGE

-- A Brief Summary --

I.D. No. : 4

Town/City/etc. : North Haven

Railway/Highway/Avenue : Interstate Route 91

State/Province : Connecticut

Owner : Connecticut DOT
1. DESCRIPTION AND HISTORY OF THE BRIDGE

   . Brief Description of Structure

   The Quinnipiac River Bridge on Interstate 91 near New Haven, Connecticut (see Fig. 1 a,b) is a four span structure over the Quinnipiac River. Span 1 is of composite construction with WF beams and welded girders. Spans 2, 3 and 4 are noncomposite welded girders of cantilever type with a suspended center span. The suspended part of the structure in Span 3 is 165 ft. (50.3 m) long between the hinges which connect it to the anchor spans (Spans 2 and 4, see Fig. 1). The span lengths are 62 ft. -12 in., 112 ft., 220 ft. and 111 ft. -1 in. for the Spans 1, 2, 3 and 4, respectively. The entire structure is on a skew, as shown in Fig. 1. The cross-sections of Spans 2, 3 and 4 are composed of nine parallel welded girders supporting each roadway in between metal team-type guard rails separating the traffic. The main girders have transverse X-type bracing and longitudinal stiffeners.

   The roadway is a 7-3/4 in. thick reinforced concrete deck. This concrete deck forms a composite system with WF beams in Span 1.

   . Brief History of Structure and Cracking

   The structure was opened to traffic in 1964. In November 1973 a large crack was discovered in the south facia girder of the suspended span in the center portion of the bridge. The bridge had experienced approximately nine years of service life at the time the crack was discovered. The exact location of the crack in the south facia girder of the suspended span is shown in Fig. 1.

   -113-
2. FAILURE MODES AND FAILURE ANALYSIS

Location of Crack, Failure Modes and Stages

As shown in Fig. 1b and Fig. 2, the crack developed in the south facia girder web. The crack propagated approximately to the mid-depth of the girder, as shown in Fig. 2b, and had penetrated the bottom flange of the girder at the time it was discovered. The location of the crack is shown on the schematic profile of the bridge in Fig. 1. It is approximately 59 ft. - 6 in. from the left support of the noncomposite suspended span.

A detailed study (Ref. 1) of the fracture surfaces indicated that the fracture had initiated at the unfused butt weld in the longitudinal stiffener. The portions of the fracture surfaces in the vicinity of the butt weld were severely corroded from exposure to the environmental effects. The crack surface indicated that some crack extension probably developed from the unfused section of the butt weld across the thickness of the stiffener during transport and erection.

Examination of the crack surface in the web adjacent to the longitudinal stiffener showed that a "brittle fracture" had occurred following the penetration of the web thickness by fatigue crack growth. Study of the fracture surfaces shown in Figs. 4, 5 and 6 revealed that the cleavage fracture propagated throughout the depth of the fractured web and penetrated some distance into the flange before it had arrested. Several stages of crack growth and the failure modes starting from the unfused butt weld on the longitudinal stiffener-girder web interface are indicated in a schematic manner in Fig. 9.
. Cyclic Loads or Stresses (Known or Field Recorded)

The stress range histograms for the girders were obtained in 1981. At the critical section of girder, three load cases exist: (1) stress $\sigma_{dl}$ due to the "live load," (2) stress $\sigma_{d1}$ due to "dead load," and (3) residual stress $\sigma_{rs}$ due to welding.

The equivalent stress range for live load are approximated from Miner's rule using gross vehicle distribution given in Ref. 2. This is approximately $S_{r,Miner} \approx 1.95$ ksi $(13.4$ MPa$)$ in the flange where the design stress range $S_{r,Design} = 4.35$ ksi $(30.0$ MPa$)$. The corresponding stresses at the longitudinal stiffener are $S_{r,Miner} \approx 1.17$ ksi $(8.1$ MPa$)$. The maximum live load stress at the longitudinal stiffener is estimated to be $2$ ksi $(13.8$ MPa$)$. (See Ref. 1 for details.)

The average daily truck traffic (ADTT = 4300) crossing the bridge was estimated to produce 1,600,000 random cycles per year corresponding to the $S_{r,Miner} \approx 1.17$ ksi $(8.1$ MPa$)$.

The stress $\sigma_{d1}$ due to dead weight was 4.95 (34.1 MPa) at the critical cross-section (see Ref. 1 for details).

. Temperature and Environmental Effects (Known or Field Recorded)

As shown in Fig. 9, Stage III of crack growth was the brittle fracture of the web probably during a time of low temperatures. Comparison of the fracture surfaces from the three point bend tests indicated that the web had fractured when the temperature was between $10^\circ F$ ($-18^\circ C$) and $-10^\circ F$ ($-25^\circ C$). Brittle fracture initiated in a zone of high residual tensile
stresses. Once the crack became unstable, it propagated through the web and was eventually arrested in the girder flange. It is highly probable that the brittle fracture occurred during the period of December 1972 - March 1973, when the material toughness of the web would be decreased by low temperatures to a minimum level (Ref. 4).

Material "Fracture, Mechanical and Chemical Properties" in the Failure Regions

Several pieces of the web and flange material at the cracked area (see also Fig. 4) were removed. Standard ASTM Type A Charpy V-Notch (CVN) tests, and tensile tests were conducted on the web and flange material (Ref. 1).

The tensile tests provided a yield strength of 36.8 ksi (254 MPa) and an ultimate strength of 60.9 ksi (420 MPa). The results of the CVN tests are summarized in Fig. 10. Both flange and web material satisfied the toughness requirements for Group 2 of the 1974 Interim AASHTO specifications. The average CVN impact value for the web was 20 ft-lb. (27 J) at 40° F (4° C). The flange provided 37 ft-lb. (47 J at 40° F (4° C).

Several three-point bend specimens were also fabricated from the web material. A notch with a 45° chevron front was machined at the center of the specimen to initiate fatigue crack growth.

The specimens were tested at temperatures of -40° F (-40° C), -20° F (-20° C), and a 0° F (-18° C). From a comparison of the fracture surface of the web crack with the fractures of the $K_c$ test specimens an estimate of the temperature at which brittle fracture occurred was made. On this
basis, it is believed that the fracture occurred at temperatures between -10°F (-25°C) and 10°F (-14°C) (see also Fig. 11). It is apparent from Fig. 10, that CVN energy absorption and consequently the dynamic toughness of the girder material decreased significantly at 0°F (-18°C). The estimated fracture toughness of the web steel from the J-Integral Analysis are plotted in Fig. 11. These tests indicated that the fracture toughness of the web material was between 100 ksi and 150 ksi at the time of the fracture.

Visual and Fractographic Examination of Failure Surfaces

The fracture surfaces of flange-web longitudinal stiffener-web intersection (at which the crack originated), and the ends of the longitudinal stiffener are given in Figs. 4, 5 and 6 respectively.

Visual examination of the fracture surfaces (see Figs. 4, 5 and 6) indicated that the failure had initiated at the web-stiffener intersection due to an unfused butt weld in the longitudinal stiffener. The web fracture's surface indicated that brittle (cleavage) fracture occurred following the penetration of the web thickness by a fatigue crack.

Replicas of the web-longitudinal stiffener intersection (Fig. 7) and near the flange-web intersection (Fig. 8) were made and examined with the transmission electron microscope. The examination of web-stiffener intersection confirmed that fatigue crack growth had occurred (see Fig. 7) in that region. The brittle (cleavage) fracture extended into the flange (see Fig. 8).
Crack Growth (Initiation and Propagation) and Probable Causes of Failure

The primary cause of failure was the inadequate butt weld made across the width of the longitudinal stiffener. It is probable that some crack extension from the unfused section occurred during transport and erection of the girder. Several stages of crack growth based on the visual and fractographic examination of the fracture surface of the web are shown in Fig. 9 (see also Ref. 1).

Stage III in Fig. 9 was the brittle fracture of the web after the initial fatigue crack became unstable and propagated rapidly down the web and arrested in the flange. This stage likely occurred during low temperature in the period of December 1972 - March 1973.
3. **CONCLUSIONS/RECOMMENDATIONS**

. **Conclusions Regarding Fracture/Failure**

During its nine years of service life, the fractured girder is estimated to have sustained 14.5 million cycles of random truck loading. The final brittle fracture and failure of the girder resulted from an initial crack which started from an unfused butt weld in the longitudinal stiffener-girder web interface which enlarge in fatigue. The final failure occurred at the end of several stages and modes of cracking as shown in Fig. 9, probably during the period of December 1972 - March 1973, when the material toughness of the web would be decreased considerably by low temperatures.

. **Recommendations for Repair/Fracture Control:**

Reference 1 analyzes in detail the cracking and subsequent brittle fracture and failure of Quinnipiac River Bridge. However, no recommendations were presented in Ref. 1 as to the repair and rehabilitation of the structure.

The information file on the bridge indicates that the fractured south facia girder was repaired by Conn-DOT with spliced bolted plates following removal and grinding smooth of the poor quality butt weld in the longitudinal stiffener-web interface which initiated the Stage I in Fig. 9.

Figure 12 shows the bolted splice detail used in repairing the cracked girder. In addition, several other inadequate groove welds were detected, and holes were drilled in the web in order to isolate the crack which was propagating into the girder web.
4. ACTUAL REPAIR/FRACTURE CONTROL EFFECTS

The fractured south facia girder of the Quinnipiac River Bridge was repaired by the Connecticut-DOT personnel, as schematically shown in Fig. 12. After necessary grinding of flame cut surfaces in the web and flange, bolted splice plates were installed to restore the integrity of the structure (3). Holes were drilled in the web to isolate a second crack near midspan.

5. POSSIBLE ADVERSE EFFECTS OF REPAIR

Not expected in this case (3) provided all the inadequate groove welds similar to the one which caused the fracture of the south facia girder were taken care of as recommended in the previous section.

6. ACTUAL OR ESTIMATED REPAIR/REHABILITATION COSTS

   . Cost of Investigation

   This was not available (probably not documented as a separate item by the Connecticut-DOT).

   . Cost of Repair/Rehabilitation

   The repair cost of the fractured girder according to a Connecticut-DOT estimate was $7,960.00.

   . Original (or Adjusted) Cost of the Bridge

   The original cost of the structure was given as $2,236,912.00 by the Connecticut-DOT.
7. REFERENCES


Fig. 1 Side View and Typical Cross Section of Quinnipiac River Bridge
Fig. 2a Profile of Quinnipiac River Bridge

Fig. 2b Crack in Web of Fascia Girder
Fig. 3 Schematic of Girder showing Sections Removed for Examination at Crack
Fig. 4 Fracture Surface at Flange-Web Junction
Fig. 5 Fracture Surface of Web Near Longitudinal Stiffener

Fig. 6 Ends of Longitudinal Stiffener at Crack
Fig. 7  Crack Growth Striations 49125X

Fig. 8  Cleavage in Flange Near Bottom Surface - 4300X

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Stage I
Initial Crack (Unfused Weld)
Fatigue Crack Growth Through Fused Part

Stage II
Fatigue Crack Growth Through Web

Stage III
Brittle Fracture in Web Arrested in Flange

Stage IV
Fatigue Crack Growth in Flange

Fig. 9 Schematic of Crack Growth Stages
Fig. 10 Charpy Results for Web and Flange Adjacent to Crack
Fig. 11 Fracture Toughness of Web estimated from Charpy V-Notch Tests and from Three-Point Bend Specimens
Fig. 12 Bolted Splice Plates in Web and Bottom Flange
5.3 FATIGUE/FRACTURE ANALYSIS
OF U.S. 51 OVER
ILLINOIS RIVER AT PERU

I.D. No. : 70
Town/City/etc. : Peru
Railway/Highway/Avenue : U.S. 51
County : LaSalle
State/Province : Illinois
Owner : Illinois

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1. DESCRIPTION AND HISTORY OF BRIDGE

   Brief Description of Structures

   The U.S. 51 Bridge over the Illinois River at Peru is a 2292 ft. (765 m) thirteen span structure with the three main truss spans consisting of a cantilever truss 1077 ft. (360 m) long. The suspended center span of the truss is 477 ft. (160 m), and the two side spans are 265 ft. (90 m). The approach spans consist of two 75 ft. (25 m) single span units, one two span and two three span units, all with spans of ± 137.5 ft. (± 46 m). Except for Spans 1, 2 and 3, each span is constructed with four riveted plate girders spaced 10 ft. - 3 in. (310 cm) apart. Span 1 has six girders, and Spans 2 and 3 have five girders. Figure 1 shows the plan and elevation of a typical 137.5 ft. (46 m) span with five girders. Welded cover plates were attached to the riveted plate girders at the piers and at midspans, as illustrated in Fig. 2.

   The girder webs were fabricated from A7 steel. The flange angles and cover plates were furnished A373 steel.

   The bridge is one of the few early girder bridges that combined riveted girders and welded cover plates.

   Brief History of Structure and Cracking

   The structure was built in 1958. On July 22, 1980, painters reported a cracked cover plate at a transition weld, as shown in Fig. 3. As of January 30, 1981 district maintenance personnel had located thirteen transverse cracks in the bottom cover plate groove welds, including two
locations in the negative moment region adjacent to Pier 2. Cover plates were cracked in two at two locations, one including the outside leg of a flange angle, as shown in Fig. 4. Eight girder cover plates were cracked between 2 and 4 in. (5 and 10 cm) from each edge of the cover plate. At three locations, grinding revealed full width subsurface cracking, as illustrated in Fig. 5. Four out of six girders were cracked near the quarter point of the north approach span.
2. FAILURE MODES AND FAILURE ANALYSIS

   Location of Fracture, Failure Modes and Stages

   Of the thirteen transverse cracks detected in the cover plate groove welds, two were in the compression flange of the negative moment regions adjacent to pier 2. The cover plates were cracked completely at two locations. The cracks all appeared to be fatigue cracks, which had propagated from large internal flaws in the cover plate groove welds.

   Ultrasonic testing of suspected weld defects in Spans 11, 12 and 13 (corresponding to the riveted plate girders in Spans 1, 2 and 3, respectively) was also carried out. These tests revealed a 3/4 in. x 7/16 in. (2 cm x 1 cm) crack in the outstanding leg of an 8 in. x 6 in. x 7/8 in. (20 cm x 15 cm x 2.25 cm) bottom flange angle near pier 12. All cover plate butt welds were ground smooth and tested with liquid penetrant in order to locate subsurface cracks.

   Cyclic Loads and Stresses (Known or Field Recorded)

   The 1979 estimated annual daily traffic crossing the bridge was 10,700 vehicles per day. The 1980 estimated annual daily truck traffic was 1950 multi-unit vehicles per day, including occasional 120,000 lb. (54,400 kg) permit loads.

   The design stress range in Span 1 was found to equal 10 ksi (68.95 MPa) at the groove weld near the 0.26 point from the north approach for HS20 loading. The stress ranges in Span 2 were found to equal 8.5 (58.6) and 11.4 ksi (78.6 MPa) at the groove welds in the positive moment region near
the 0.23 and 0.55 points where cracks had been detected. The stress range adjacent to pier 2 in the compression flange was 4.4 ksi (30 MPa).

The net section properties were used to estimate stresses in the tension flanges of the positive moment areas and the gross section was used for the compression flange in the negative moment regions.

- Temperature and Environmental Effects (Known or Field Recorded)

There was nothing available.

- Material "Fracture, Mechanical and Chemical Properties"
  in the Failure Regions

There was nothing available.

- Visual and Fractographic Examination of Failure Surfaces

There was nothing available.

- Crack Growth (Initiation and Propagation)
  and Probable Cause of Failure

The primary cause of failure was the poor quality of the groove welds at the thickness transitions of the cover plates. This was due to incorrect procedures used during fabrication and resulted in poor weld quality. Large cracks appear to be fabricated into the members as a result.
3. CONCLUSIONS/RECOMMENDATIONS

. Conclusions Regarding Fracture/Failure

The groove weld transitions at all cover plate splices were ground to expose subsurface cracks.

Butt welds that had surface cracks less than 1/16 in. deep were ground smooth, and the crack removed. The joints were also liquid penetrant tested to insure that no evidence of cracking remained.

. Recommendations for Repair/Fracture Control

All groove welds which were found or suspected to be defective were retrofitted with bolted splice plates. A typical splice is shown in Fig. 6. The repair work was carried out by Illinois DOT day labor crews. Those groove welds that were found to be acceptable by nondestructive inspection were ground smooth and left as constructed.

. Actual Repair/Fracture Control Efforts

All defective butt welds were ground down a maximum of 1/4 in. (6 mm). Contact surfaces between the splice plates and existing cover plates were ground smooth to ensure proper fitting.

Bent cover plates were used to splice across the defective butt welds in Spans 2 and 13.

Additional bolts were added to compensate for the forces induced by the sloped bearing face. Figure 6 shows the general splice detail used throughout the structure.
. Possible Adverse Effects of Repair

No adverse effects are expected as a result of the repair procedures that were carried out.

. Actual or Estimated Repair/Rehabilitation Costs

Cost of Investigation

This was not available.

Cost of Repair/Rehabilitation

The repair cost for retrofitting the cracked cover plates was $32,035.64 (day labor costs 1980).

Original (or Adjusted) Cost of the Bridge

The original cost of the structure in 1958 was $2,539,737.88.
Fig. 1 Plan and Elevation of Spans 1 and 2
**MATCH LINE**

<table>
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<tr>
<th>5 MATCH LINE</th>
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<td>8-12&quot;</td>
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</tbody>
</table>

**AXIS PITCH**

- 5-12" to 6 ARG. PER #2
- 6-12" to 6 ARG. PER #2

**SLOPE**

- 2-0" to 1-6"

**DETAIL OF COVER R. BUTTWELD**

*SCALE: 6" = 1 FT*

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Fig. 2  Typical Welded Cover Plate Detail
Fig. 3  Cracked Groove Weld in Cover Plate

Fig. 4  Crack Extending into Flange Angle
Fig. 5  Ground Groove Weld Showing Crack
Fig. 6 Typical Repair Splice and Removal of Fillet Weld
6. WEB PENETRATIONS

The cracking of the rigid frame bents of the Dan Ryan elevated transit structure in Chicago in 1978, resulted in an examination of other structures with comparable details. In the Dan Ryan structures, longitudinal girders framing into the steel box bents had their bottom compression flanges pass through slots in the box girder web. A detailed description of this cracking is given in the summary that follows (I.D. 7).

In the Dan Ryan structures, the flange plates that passed through the web slots were groove welded to the web. However, large lack of fusion defects were found to exist in the web at the tips of the flange plates.

Two other bridge structures have experienced cracking at similar lack of fusion conditions (I.D. 40 and I.D. 105). In both instances the details were retrofitted before significant damage developed by isolating the detail with holes similar to the condition described for the Dan Ryan. In the Girard Point pier caps (I.D. 40) the flange plates were connected to the web slots by exterior fillet welds only. In the Metro pier cap (I.D. 105) vertical reaction brackets were passed through the web. These brackets were welded to the web using backup bars and the resulting crack-like condition and lack of fusion adjacent to the flange resulted in significant crack growth.
6.1 FATIGUE/FRACTURE ANALYSIS
of
DAN RYAN RAPID TRANSIT STEEL BOX BENTS
-- A Brief Summary --

| I. D. No. | : 7 |
| Town/City/etc. | : Chicago |
| Railway/Highway/Avenue | : Dan Ryan Rapid Transit |
| County | : Cook |
| State/Province | : Illinois |
| Owner | : Chicago Transit Authority |
1. DESCRIPTION AND HISTORY OF THE STRUCTURE

   Brief Description of the Structure

   The Chicago Transit Authority's (CTA) Lake-Dan Ryan Rapid Transit Line provides direct service between the city center and the south and west sectors of the city. The tracks ascend from the median onto a viaduct which carries them through a curve of 400 ft. or 120 m radius to a smaller junction curve which joins them with the old North-South Elevated structure at 17th and State Streets. The side view and underview of the structure are shown in Figs. 1a and b.

   General features of the viaduct at the location of cracked bends Nos. 24, 25, 26 are shown in Figs. 2a and 2b. The superstructure consists of four continuous plate girders which carry a cast-in-place concrete deck and two tracks. The rails of the two tracks are on wooden ties resting on ballast. The bents supporting the four continuous I-beam plate girders have a box-shaped cross-section consisting of two column legs and a horizontal box member as shown in Fig. 2b. The stringer bottom flanges pass through flame-cut slots near the bottom of the side plates or webs of the box girders. The stringer flange was inserted through these slots and then connected to the supporting box girder web by groove welds all around the flange surface.

   The stringers at the location of failure have a track curvature of 400 ft. or 120 m in the horizontal plane, and they intersect the boxes at different angles (approximately 45°). With the exception of bent No. 35, the bents were fabricated with portions of the superstructure girders or stringers welded integrally within the boxes. The girder stubs project 3 to 4 feet from the sides of the boxes and are connected with the continuous girders by a pinned link or bolted connection (see also Fig. 1a).
. Brief History of the Structure and Cracking

The entire structure was designed in 1967 in accordance with the criteria and procedures established by the American Railway Engineering Association (AREA), the American Welding Society Specifications, and the CTA design policy for elevated railways. The construction of the Dan Ryan Line started in April 1968 and was complete and opened to traffic in September 1969. On an average day, 467 trains (in both directions) pass over this viaduct. The trains vary in length from eight cars during the rush hours to two cars during the nighttime \((1,2,3)\).

On January 4, 1978, a crack was discovered in one of the steel box girders in bents (or frames) near 18th and Clark Streets. Subsequent inspection of the structure verified the existence of this crack in bent No. 24 and also revealed that bents Nos. 25 and 26 were cracked as well. A typical crack is shown in Figs. 3a and 3b. The most recent inspection prior to the discovery of the cracks was conducted in July 1976, and no significant cracks were found at that time.

Initial examination of fractures in bents Nos. 24, 25, and 26 indicated that they originated from fatigue cracks at or near the welded junctions where the bottom flanges of stringers intersect the steel box bent side plates (see Figs. 3 and 4a, b, and c).

Prior to the discovery of the fractures, temperatures in the Chicago area were extremely low on several occasions. There were five occasions when the temperatures dropped below 0° F, and the coldest temperature recorded at Midway Airport was -7° F in December 1977.
2. FAILURE MODES AND FAILURE ANALYSIS

Location of Crack(s), Failure Modes and Stages

The field examination and measurements indicated the location shape and pattern of the crack, as shown in Figs. 4a, b, c. The common feature of the cracks in bents Nos. 24, 25 and 26 was their close proximity to the vertical edge of the bottom flange of the stringers (1,2,3). The cracks completely separated the box girder bottom flange plate and both side plates or webs in all three bents Nos. 24, 25 and 26. Openings at the bottom flange of about 3/4 in. (20 mm) were measured. They were arrested at the edge of, or with only very little penetration into the top flange plate of the box girder of the bents. In bent No. 24, the crack extended into the web of the plate girder (see Fig. 4a).

The initial field examination of fractures in bents Nos. 24, 25 and 26 indicated that they initiated at the welded junction of a plate girder or stringer bottom flange tip to the supporting box girder side plate. Chevron markings were observed on the fracture surfaces pointing toward the welded junction. The fracture surfaces on bents Nos. 25 and 26 appeared to be lightly rusted, while the fracture surfaces on bent No. 24 was more heavily rusted.

The trajectory of the crack in the box girder of each bent was influenced by the dominance of tension, shear, and flexural stresses (see, for instance, Figs. 4a, 4b and 4c).

The examination of the fracture surfaces in each bent showed that a brittle fracture occurred after fatigue cracks developed from the unfused welds at the edges of the bottom flange of stringers (or I-beams) intersecting horizontal box girder of bents (3). In the fractured bents, the intersections were connected by partial penetration welds on the outside...
and little or no weld inside. Typical stages of crack growth in one of the box girders are presented in Fig. 5.

Cyclic Loads or Stresses (Known or Field-Recorded)

Following the discovery of cracking in the bents of the Dan Ryan Rapid Transit, a reanalysis of the bents based on the finite element method was performed. The stress analysis indicated that the computed stresses were generally less than the values allowed by the governing specifications of the American Railway Engineering Association. The maximum design shear stress in the web plate in the east side of bent No. 24 near stringer girder No. G127 was 17.27 ksi (119 MPa) which exceeded the value of 12.5 ksi (86 MPa) currently allowed by these specifications(1,2).

The normal design cyclic stress range during passage of a train was estimated to be about 3.2 ksi (22 MPa) at the edge of the stringer flange. With variable amounts of impact and multiple presence of two trains on occasion, it was estimated that an effective stress range of 1.4 ksi (10 MPa) was reasonable for the trains that had crossed the structure. About 7 million variable stress cycles had occurred before rapid crack extension.

Temperature and Environmental Effects (Known or Field-Recorded)

The temperatures in the Chicago area were known to be extremely low prior to the discovery of the fractures. There were five occasions when the temperatures dropped below 0° F (18° C), and the coldest temperature was recorded at the Midway Airport on December 1977 of -7° F (22 C). Low temperatures were again experienced in Chicago on January 3, 1978 at the time the crack in the box girders of the viaduct was discovered(1,2).
Material Fracture, Mechanical and Chemical Properties

on the Failure Regions

Several sample pieces were recovered from the fractured box girders (1, 2). From this material specimens were fabricated. Tensile tests, chemical analysis, Charpy V-notch tests (CVN), Compact Tension Fracture tests (CT) and metallographic and fractographic examinations were conducted on these samples (1, 2, 3).

The tensile tests provided a yield strength (0.2% offset) of 33.7 ksi (235 MPa) and an ultimate strength 69.6 ksi (480 MPa). Machining and testing was done according to ASTM Designation E8.

The chemical analysis performed on the specimens was done according to ASTM Designation E350.

Both tensile and chemical tests indicated that the material used in the structure conformed to the ASTM requirements for A36 steel (1, 2, 3).

Charpy V-notch tests (CVN) were performed on the specimens in accordance with ASTM Designation E23. The results are presented in Fig. 6. The results indicated that all plates satisfied the impact requirements of the AASHTO specifications [15 ft-lb. (20 joules) at 40°F (4°C)] for a minimum service temperature of -29°F (-34°C) for Zone 2. The transition temperature behavior for the material in the range between +40°F (4°C) and +70°F (+21°C), as shown in Fig. 6 (2).

Compact tension fracture (CT) specimens were tested at a one second loading rate between -30°F (-34°C) and +30°F (-1°C) in general accord with ASTM Designation E399. The results did not fulfill the requirements of ASTM E399, and therefore the stress intensity factor \( K_{IC} \) had to be obtained by a J-Integral approximation.
The results are summarized in Fig. 7. It is apparent from Fig. 7 that a significant increase in toughness developed about 10° F (-12° C)\(^{(1)}\).

Visual and Fractographic Examination of the Failure Surface

Visual examination of fracture surfaces indicate that the stringer bottom flange tip was partially welded to the box girder side plate. Paint was also noted in gas holes and other unfused portions of the fracture surfaces in bent Nos. 24 and 26. Slag, blowholes and paint on unfused regions were observed in the welded junctions\(^{(1,2)}\).

A detailed microscopic examination was carried out on the fracture surfaces of bent No. 26 (see Fig. 5) and to a limited extent on bent Nos. 24 and 25. The examinations were made by both the transmission electron microscope and the scanning electron microscope. The microscopic examination showed that the major portion of the fracture surface below and above the welded junction is coarse grained and contains chevron markings, typical of brittle fracture\(^{(1,2,3)}\).

The examination of a piece removed from bent No. 24 indicated that crack growth initiated near the inside flame cut surface and also at the exterior weld surface and extended toward the outside. Similar observations were made for bent No. 25.

The microscopic examination established that fatigue cracking (see Fig. 8) in the welded junctions had preceded the subsequent brittle fractures. The estimated length of flaws in bent Nos. 24, 25 and 26
(vertical height of the flame-cut region) plus additional height of fatigue cracking was 3 in. (8 cm), 3 in. (8 cm) and 2 in. (5 cm), respectively. The metallographic examination did not reveal any defects in the steel. The appearance of the grain structure near the surface was considered to confirm both fatigue and fracture modes of crack propagation.

. Crack Growth (Initiation and Propagation) and Probable Causes of Fracture Failure

The field examinations of the fractures in bent Nos. 24, 25 and 26 indicated that the fatigue cracks originated at the partially welded junction of an I-beam plate girder (or stringer) bottom flange tip to the side plate of the box bent\(^2,3\). Numerous weld deficiencies were apparent such as slag, blowholes, etc. Subsequent microscopic analysis of the failure surfaces confirmed the above observation (see Fig. 8).

The laboratory testing verified that the combination of stress concentration at the point of origin, fatigue cracking and cold temperatures was sufficient to cause the brittle fracture of box girder bents.

It can be concluded that the severe stress concentration and the lack of fusion in the welds at the junction of the stringer bottom flange and side plate of the box girder (or bent) is the primary cause of the cracks which occurred in the bents of the Dan Ryan Rapid Transit structure\(^2,3,4\).
3. CONCLUSIONS/RECOMMENDATIONS

   Conclusions Regarding Fracture/Failure

A detailed examination of the fracture surfaces from the cracked steel pier bents of Dan Ryan Elevated structure has shown that these fractures were all caused by fatigue cracks\(^{(2, 3)}\). These cracks originated from poor quality welds at the edges of stringer bottom flanges where they intersect box bents (see Fig. 5b).

During fabrication slots for the stringers flanges were flame cut into box girders. Subsequently stringers were inserted through these slots and connected to the box bent girder web by welding. This kind of connection created the high stress concentration at the weld adjacent to the stringer flange. Thus, the joint was very sensitive to fatigue crack propagation at very low levels of stress range\(^{(1, 2, 3)}\).

The fatigue cracks developed at welds adjacent to the stringer flange edge in box bent girders reached a critical condition at the low temperatures that occurred in Chicago in late December, 1977 and early January, 1978. Thus, fatigue cracks, poor quality welds, high stress concentrations and low temperatures occurring simultaneously caused the brittle fractures of box girders which were discovered in early January, 1978\(^{(1, 2)}\).
Recommendations for Repair/Fracture Control

A detailed repair and fracture control plan for the bents of the Dan Ryan Rapid Transit is given in Refs. 2 and 4. Specifically for locations where flanges pass through the web, any retrofit must reduce the severe stress concentrations at the intersection of the stringer-bottom flange and box girder webs (or side plates) and also reduce high residual stresses from weld shrinkage.

For this purpose, "dumbbell" type retrofit holes and saw cuts were installed on side plates of the box bents near the tips of all stringer bottom flanges as shown in Fig. 9. In addition, 1 in. diameter holes at the tip of the cracks in the fractured bents were used to arrest the cracks\(^{(2,4)}\). The cracked side plates and bottom plate of the horizontal box member of each bent are to be repaired with bolted splice plates. Also, the depth of the horizontal box member of each bent is to be increased by addition of a "U" shaped section which will be bolted to the underside of the existing box as shown in Fig. 10.

Prior to retrofitting the structures and after one week of lack of service, temporary support cribbing was erected under each cracked bent to provide support to the cracked members. These cribs have remained in place pending the final repair\(^{(2)}\).
4. **ACTUAL REPAIR/FRACTURE CONTROL EFFORTS**

Immediately following the discovery of cracks in box girder side plates (or webs) on January 3, 1981, Department of Public Works of the City of Chicago started the construction of temporary support structures under the box girders at Bent Nos. 21 through 26\(^2\).

Each temporary steel support structure consisted of jacking beams placed on both sides of the box girder and supporting the existing longitudinal plate girders. Jacking beams were supported by a steel falsework tower with bracing resting on timber mudsill and crushed stone\(^2\). The temporary support structures were to carry the weight of the tracks and trains until the permanent repair and retrofit recommendations\(^2\) were implemented.

Holes were placed at the flange tip of all details as shown in Fig. 9.

5. **POSSIBLE ADVERSE EFFECTS OF REPAIR**

In the case of Dan Ryan Rapid Transit Structure no adverse effects of repair are expected. However, the field inspection of the structure at critical details in intervals to preclude the possibility of fatigue crack growth should be continued.
6. ACTUAL OR ESTIMATED REPAIR/REHABILITATION COSTS

- Cost of Investigation

This is not available.

- Cost of Repair/Rehabilitation

The cost of repair/rehabilitation is not available.

- Original or Adjusted Cost of the Structure

The original cost of the structure is not available.

7. REFERENCES


Fig. 1.a General View of Dan Ryan Rapid Transit Structure

Fig. 1.b View of Cracked Bents Looking West from Clark Street (Arrows Are Pointing to Locations of Fracture)
Fig. 2.a Schematic Plan and Locations of Cracked Bents (Bent Nos. 24, 25, 26)
Fig. 2.b Bent No. 26 and Crack Locations
Fig. 3 Fracture of the Box Girder (Bent No. 26)
Fig. 9 - Crack profile on Bent No. 24

Fig. 4.a Crack Profile on Bent No. 24
Fig. 4.b Crack Profile on Bent No. 25
ELEVATION OF WEST SIDE PLATE

ELEVATION OF EAST SIDE PLATE

CRACK EXTENDS 5 1/4" 2 1/4" 3" INTO TOP FLANGE

5/32

1/16

1/32

7/8

10 1/2"

15"

1/32

3/16

MEASURED WIDTH OF CRACK

EAST SIDE

SECTION D-D

Fig. 4.c Crack Profile on Bent No. 2b
Fig. 5 Schematic of Fracture Surface at Bent No. 26
Fig. 6 Comparison of CVN Test Results
Fig. 7 $K_{IC}$ vs. Test Temperature
Fig. 8 Fatigue Crack Growth Striations in Region 3 on Box Girder Web Surface at Bent No. 2b (X17000)
(a) Schematic Showing Retrofit

Fig. 9 Typical Retrofit Detail at Bent No. 26 (same as No. 21, 22, 24, 25)

(b) Photograph of Retrofitted Box Side Plate (or Web)
Fig. 10 Recommended Repair and Structural Modifications
(Bent Nos. 24, 25, and 26)
Several bridges have developed cracks as a result of misplaced holes that were subsequently filled with weld. This has resulted in lack of fusion and other cracklike defects at each welded hole that eventually experienced significant crack growth and fracture.

At least three multibeam bridges developed major cracks in one or more girders as a result of welding-up misplaced holes. The Ill. 57 overpass at Farina (I.D. 54) is reviewed and summarized hereafter. Welded-up misplaced holes in rolled sections resulted in the cracking of one of the beams. Similar conditions were found in two structures in Iowa (I.D. 57 and I.D. 89).

In every instance cracks were observed to originate at several holes. These often linked and resulted in fracture of the section. Only the Illinois structure experienced a crack that turned and traveled down the length of the beam.

A similar condition was found to exist in a cover-plated plug welded chord of the Burned River Truss Bridge in Ontario (I.D. 55). The cover plate was attached to a rolled section with plug-welded holes. This created a lack of fusion and defect condition that was similar to the weld filled holes. The crack eventually caused the chord to fracture.
7.1 FATIGUE/FRACTURE ANALYSIS

OF

COUNTY HIGHWAY 28 BRIDGE OVER I-57

-- A Brief Summary --

I.D. No. : 54
Town/City/etc. : near Farina, Illinois
Railway/Highway/Avenue : County Highway 28 over I-57
County : Fayette County, Illinois
State : Illinois
Owner : Illinois-DOT
1. DESCRIPTION AND HISTORY OF THE BRIDGE

. Brief Description of Structure:

The county Highway 28 bridge over Interstate I-57 is located north of Farina, Illinois in Fayette County. The structure is a skewed four-span composite-reinforced-concrete-slab steel-beam bridge. It was completed and opened to traffic in 1968. The bridge primarily carries local traffic over Interstate I-57.

The general plan and elevation of the structure is given in Fig. 1. A typical cross-section is shown in Fig. 2. The bridge has a 32 ft. wide 7 in. thick reinforced concrete deck supported by five continuous W36X150 steel beams of ASTM A36-67 steel. It has two 95 ft. -5 in. main spans and two 55 ft. - 8-1/2 in. side spans. The concrete slab is connected to the main girders with shear studs with different spacings depending on the location along the span.

The entire structure was constructed in accordance with the "AASHO Standard Specifications for Road and Bridge Construction," January 2, 1958; the supplemental specifications, dated January 3, 1966; and the current specifications of Illinois-DOT.

. Brief History of Cracking:

On March 8, 1977 a large crack in the south side of the first interior girder (Girder No. 2) of the second span was discovered by a construction technician while driving under the bridge (1). The fracture in the stringer originated at one of the four horizontal rivet holes which were mispunched and later improperly repaired and filled with weldments.

During the winter of 1976-77 very low temperatures of -20° F (-29° C) were experienced in the area where the bridge was located (1).
2. FAILURE MODES AND FAILURE ANALYSIS

Location of Fracture, Failure Modes and Stages:

The location of the fracture in the south interior beam is shown in Fig. 3, and a detailed schematic drawing is given in Fig. 4 (1).

There were two separate fractures which started from two holes filled with weldment immediately under the clip angle supporting the diaphragm (1,2). Both cracks were initiated from weld filled holes with large slag inclusions. Both fractures propagated down through the flange of interior beam 2B2 at a common termination point. These cracks also extended up and turned longitudinally along the web at about middepth. One crack extended about 1 ft. (0.305 m) while the other curved upward and extended horizontally approximately 15 ft. (4.57 m), reaching the next diaphragm as illustrated in Fig. 4. Photographs showing the cracks and the plug holes are given in Figs. 3 and 5.

Several pieces, including the crack surface, were removed by flame cutting at locations shown in Fig. 7.

The results of mechanical and fracture tests of the material and visual examination of the fracture surfaces shown in Fig. 5 and 6 indicated that the failure was primarily a brittle fracture along the entire length of the cracks.

There was very little ductility in the fracture. The brittle fracture probably occurred during the extremely low temperatures recorded in the area during the winter of 1976-77. Close examination of the plug welds show the existence of the herring bone patterns near the slag and voids in
welds (1,2). Thus, the fracture likely initiated from fatigue sharpened cracks originating from slag and voids in the plug welds. During low temperatures, the brittle fracture resistance was decreased. The small crack from plug weld 4 (probably occurred at about the same time as the cracks from plug weld 3) propagated, as shown in Fig. 4.b, in brittle failure mode down toward the bottom flange and intersected the crack from plug weld 3.

Cyclic Loads and Stresses (Known or Field-Recorded):

The normal traffic using county Highway 28 bridge over I-57 is composed mainly of automobile and light farm equipment (1). There are no truck traffic counts available. Illinois-DOT estimated the ADT to be 400 vehicles in 1976 (3). The computed design stress in the bottom flange adjacent to the origins of fracture is 15.2 ksi (104.8 MPa) based on the HS 20 truck loading (3).

A series of field live-load tests with loaded trucks were conducted on the bridge following the repair of the fractured beam. During these tests a dump truck was loaded with gravel to a gross vehicle weight of 44,700 lb. (98,500 kg). This truck was driven across the bridge in order to determine the live load response of the structure.

A series of strain gages were installed on the replacement beam 2B2 at several locations including the fracture initiation point (for strain gage locations, see the schematic figure included in Table 1). This gage was used to estimate the maximum live-load stresses (1). Results of these field tests are summarized in Table 1. The measured stress ranges in Beam 2B2 were all quite low with the maximum stress range being 3.46 ksi
(23.9 MPa). The ASTM A36-37 steel used in the beam has a yield point of 43.5 ksi (300 MPa). Therefore, live-load tensile stresses observed in the beam even with the addition of the dead-load stresses are well below the yield point of the steel. Welding residual stresses from the weld filled holes would introduce stresses near the yield point at each hole.

**Temperature and Environmental Effects (Known or Field Recorded):**

The time the fractures of the south side interior girder in the bridge over I-57 were discovered, temperatures in the locality during the 1976-77 winter were as low as -20° F (-29° C). Aside from the low service temperature, no other significant environmental effects were apparent.

**Material "Fracture, Mechanical and Chemical Properties" Analysis in Failure Regions:**

The lower portion of the beam 2B2 in the shape of a tee-section was removed from the bridge by flame cutting between two bolted splices. Specimens fabricated from this portion of the beam 2B2 were used for tensile tests, Charpy V-Notch tests, fracture toughness tests and for the chemical analysis (1).

The results of chemical analysis indicated that the chemical composition of the steel was well within the chemical requirements specified for A36 steel (1).
Tensile tests were also conducted on specimens fabricated from beam 2B2. The test results indicated that the web material had an average yield point of 43.5 ksi (300.0 MPa) and an ultimate strength of 71.3 ksi (491.6 MPa). Similar results were obtained for the flange. The tensile properties of the steel satisfied the specification requirements for ASTM A709 Grade -36 steel (A36 steel).

Charpy V-Notch tests were carried out in accordance with ASTM standard A370. Four sets of standard size specimens were cut from the web and flange of the fractured girder.

The test results are summarized in Fig. 8. Both longitudinal and transverse specimens were tested from the beam web. All of the specimens tested at 40° F (4° C) exceeded the 15 ft-lb. (20 J) requirement of the AASHTO specifications for A36 steel:

- Web specimens "WL" with their lengths parallel to the direction of rolling of the beam (the notch being perpendicular to the direction of rolling)
- Web specimens "WT" with their lengths transverse to the direction of rolling (the notch being parallel to the direction of rolling).

Two sets of flange specimens were cut with their lengths parallel to the direction or rolling of the beam and their notches parallel to the surface of the flange, and

- Flange specimens "FT" was located at the top of the flange,
- Flange specimens "FM" was located at the middepth of the flange
In addition to the Charpy V-Notch tests, fracture toughness tests were conducted on bend specimens in order to establish the toughness of the steel. Since A-36 steel was used in the beam, valid $K_{IC}$ tests were not possible. Static fracture toughness values, $K_Q$, for the steel were estimated from the maximum load. No load-deflection data was acquired, hence a more accurate determination using the J-integral was not possible. These tests provided $K_Q$ values of about 49 ksi $\sqrt{\text{in.}}$ (53.9 MPa $\sqrt{\text{m}}$) at $-20^\circ$ F ($-29^\circ$ C). A single test at $-40^\circ$ F ($-40^\circ$ C) yielded 50 ksi $\sqrt{\text{in.}}$ (55 MPa $\sqrt{\text{m}}$). These results were compatible with the estimated $K_c$ values provided by the Barsom-Rolfe correlation equation and the intermediate rate temperature shift.

In addition to the Charpy V-Notch and $K_Q$ tests, a series of tensile tests were conducted on rectangular specimens containing plug welds (1). The test specimens were approximately 3 in. (76.2 mm) wide. The specimens were machined such that the plug weld was centered on the width of the specimen. Three tests were carried out at $-20^\circ$ F ($-29^\circ$ C) and one at $-40^\circ$ F ($-40^\circ$ C). The results are summarized in Table 2.

All of these tests failed at nominal stresses on the gross section that were well in excess of the yield. Obviously, none of the plug welds in these specimens had initial flaws that were as large as those at the failure section. Failure occurred at stresses about midway between the yield point and tensile strength of the base plate.
Visual and Fractographic Analysis of Failure Surface:

Visual observations indicated that the plug welds initiated the fracture of Beam 2B2.

Photographs of one of the fracture surfaces are shown in Figs. 5 and 6. These indicated that a small shear lip is visible along the outer edges over most of the length. Figure 3a demonstrates a similar condition. No spalled paint exists at the plug weld and for a distance of about 1 in. (25 mm) each side of the plug welds. Therefore, the mill scale is spalled off and confirms the plastic behavior associated with the shear lips. The herringbone surface patterns and the fine texture of the surface near the plugs in Fig. 5 also show the brittle fracture characteristic which caused the sudden unstable crack propagation at a low nominal stress level (1). The slag and the voids which initiated the unstable crack growth are also visible in Fig. 5. The herringbone patterns in Fig. 6 point towards the plug welds.
3. CONCLUSIONS AND RECOMMENDATIONS

Conclusions Regarding Fracture/Failure:

The investigation of the fracture of a rolled beam in Fayette County Highway 28 Bridge indicated the following:

- The maximum live load stresses in the cracked beam was measured to be 3.46 ksi (23.86 MPa) which was far below the AASHTO allowable stresses for such a member (1).

- The chemical and mechanical properties of the steel were well within ASTM-A36 specifications (1).

- The steel met the Charpy V-Notch requirements for service temperatures down to -30°F (-34°C) which was lower than the temperatures experienced by the fractured beam (1).

- Fatigue cracks and subsequent brittle fracture resulted from the presence of plug welded holes. These welded holes resulted in large cracklike discontinuities that were susceptible to crack propagation. The fracture surfaces shown in Figs. 5 and 6 show the cracks that formed at the welds. Radiographs of other weld filled holes showed plug welds with slag inclusions and voids.
Recommendations Regarding Repair/Fracture Control:

It can be expected that cracking will develop in other mispunched holes that are filled with weldment. The problem can be avoided by leaving the holes and installing high strength bolts rather than welding.

Existing weld filled holes should be removed by drilling or coring in order to remove the discontinuities and prevent any further undesirable cracking.

4. ACTUAL REPAIR/FRACTURE CONTROL EFFORTS

The repair of county highway 28 bridge over I-57 was carried out as follows:

The fractured section of Beam 2B2 was removed by flame cutting the lower portion of the beam in the form of a tee-section about 10 in. (254 mm) below the top flange for the entire length between splices.

A new tee-section that matched the section removed was field-welded horizontally to the remaining section of the web and then bolted to the adjacent beam segments using the existing splice plates. The repaired portion of the beam is shown in Fig. 7. The tee-section removed from the structure was used to fabricate test specimens in order to establish the chemical, physical; and fracture toughness characteristics of the steel (1,2). Fractographic studies were also carried out on the original crack surfaces on these specimens.
5. **POSSIBLE ADVERSE EFFECTS OF REPAIR**

No adverse behavior is anticipated from the repaired member. Non-destructive tests were carried out on the longitudinal field weld. Furthermore, it is located near the neutral axis, so no appreciable cyclic stress is likely to ever occur at that level.

6. **ACTUAL OR ESTIMATED REPAIR/REHABILITATION COSTS**

**Cost of Investigation:**

No direct investigative costs were accounted for. All costs were included in the cost of retrofit/repair. The University of Illinois studies were carried out as part of their ongoing work.

**Cost of Repair:**

Available records (3) indicate that the cost of the repair was $13,150.00 in 1977.

**Original or Adjusted Cost of the Structure:**

The county highway 28 bridge crossing over Interstate of I-57 was awarded at a cost of $164,607.85 in 1958 (3).
7. REFERENCES

(1) Handel, W. and Munse, W. H.
INVESTIGATION OF INTERSTATE I-57 BRIDGE BEAM BRITTLE FRACTURE,
University of Illinois, Structural Research, Series No. 477,
March 1980.

(2) Illinois Department of Transportation

(3) Illinois Department of Transportation
Personal communication from J. B. Nolan to J. W. Fisher,
TABLE 1: MEASURED STRAINS AND COMPUTED STRESSES
IN BEAM 2B2 UNDER TEST TRUCK

<table>
<thead>
<tr>
<th>Truck Run</th>
<th>Gage Location</th>
<th>Tension Stress ksi (MPa)</th>
<th>Compression Stress ksi (MPa)</th>
<th>Stress Range ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Crawl --</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Centered on</td>
<td>44</td>
<td>2.56 (17.65)</td>
<td>0.34 (2.34)</td>
<td>2.90 (19.99)</td>
</tr>
<tr>
<td>Beam line 2,</td>
<td>34</td>
<td>2.21 (15.24)</td>
<td>0.28 (1.93)</td>
<td>2.69 (17.77)</td>
</tr>
<tr>
<td>Southbound</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. Crawl --</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Right wheels</td>
<td>44</td>
<td>2.28 (15.72)</td>
<td>0.34 (2.34)</td>
<td>2.62 (18.06)</td>
</tr>
<tr>
<td>over Beam Line</td>
<td>34</td>
<td>1.98 (13.65)</td>
<td>0.28 (1.93)</td>
<td>2.26 (15.58)</td>
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<tr>
<td>2, Southbound</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3. 30 mph -</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Centered on</td>
<td>44</td>
<td>2.84 (19.58)</td>
<td>0.62 (4.28)</td>
<td>3.46 (23.86)</td>
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<td>Beam line 2,</td>
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<td>0.56 (3.86)</td>
<td>3.03 (20.89)</td>
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<tr>
<td>Southbound</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
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</table>
### Table 2: Tensile Test of Specimens with Plug Welds

<table>
<thead>
<tr>
<th>Spec. No.</th>
<th>Gross Area in.² (mm²)</th>
<th>Specimen Test Temp. °F (°C)</th>
<th>Tensile Strength kips (kN)</th>
<th>Stress on Gross Area ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10a</td>
<td>1.842 (1188)</td>
<td>-20 (-29)</td>
<td>100.97 (449.1)</td>
<td>54.82 (378.0)</td>
</tr>
<tr>
<td>10b</td>
<td>1.822 (1175)</td>
<td>-20 (-29)</td>
<td>105.13 (467.6)</td>
<td>57.70 (397.8)</td>
</tr>
<tr>
<td>10c</td>
<td>1.815 (1171)</td>
<td>-20 (-29)</td>
<td>114.9 (511.1)</td>
<td>63.31 (436.5)</td>
</tr>
<tr>
<td>10d</td>
<td>1.807 (1166)</td>
<td>-40 (-40)</td>
<td>99.6 (443.0)</td>
<td>55.12 (380.0)</td>
</tr>
</tbody>
</table>
Fig. 1 Plan and Elevation of the County Highway 28 Bridge
(a) Framing Plan and Location of Beam 2B2

(b) Cross-Section

Fig. 2 Framing Plan and Cross-Section
Fig. 3 Schematic and Photograph of Crack at the Welded Bolt Holes 3 and 4
(a) Beam 2B2 and Crack

(b) Crack Initiation and Propagation from Plug welds

Fig. 4 Location of Fracture and Crack Initiation and Propagation
Fig. 5 Fracture Surfaces
(see also Fig. 3.a for the direction in which photographs were taken)
Fig. 6 Fracture Surface
(see also Fig. 3.a for the direction in which photographs are taken)
Fig. 7 Retrofit Detail (Replacement to Beam 2B2)
Fig. 8 Charpy V-Notch Impact Tests and Transition Temperature
8. CANTILEVER FLOOR BEAM BRACKETS

One of the earlier types of cracking that was found to develop in highway bridges was fatigue cracks that were discovered in the tie plates connecting transverse floor beams and cantilever brackets across the main girders.

The first such cracking was found in the Allegheny River Bridge (I.D. 16) and was observed to originate from the rivet holes used to attach the plate to the main longitudinal girders. Shortly thereafter, cracks were observed in the Lehigh River and Canal Bridges (I.D. 14). These particular cracks were found to originate from tack welds used to connect the tie plate to the bracket prior to shop installation of the rivets.

A summary and review of the Lehigh Canal Bridge is provided hereafter (I.D. 14). Detailed measurements and an assessment of the crack development was made on the Lehigh Canal Bridge. This study demonstrated that unforeseen in-plane displacements of the tie plate, as a result of rotation of the longitudinal girders at a cross-section was the reason such cracks developed.

One other bridge experienced distress at the girder-bracket joint. The South Bridge (I.D. 17) was found to have a number of rivet heads cracked off. The in-plane movement of the connection plate distorted the rivets and caused prying of the rivet heads which resulted in separation.
8.1 FATIGUE/FRACTURE ANALYSIS
of
LEHIGH CANAL BRIDGE
-- A Brief Summary --

I. D. No. : 14
Town/City : Allentown, PA
Railway/Highway/Avenue : U.S. 22 in Pennsylvania
County : Lehigh
State/Province : Pennsylvania
Owner : Penn-DOT
1. DESCRIPTION AND HISTORY OF THE BRIDGE

   • Brief Description of the Structure:

   The Lehigh Canal and River Bridges consist of two adjacent twin structures which carry the eastbound and westbound lanes of U.S. Route 22 over the Lehigh Canal and River near Bethlehem, Pennsylvania. Each bridge is a two girder continuous structure extending over three spans with small haunches at the interior piers. The schematic plan and elevation is shown in Figs. 1.a and 1.b. A side view photograph of the bridge is given in Fig. 1.c. The typical cross-section and tie plate details are shown in Figs. 2.a and b, respectively.

   • Brief History of Structure and Cracking:

   The twin bridges (one for the westbound lane and one for the eastbound lane) were constructed in 1951-1953 and were opened to traffic in November 1953.

   During the spring of 1972, inspection by PennDOT personnel revealed several fatigue cracks in the tie plates connecting the transverse floor beams and cantilever brackets across the main girders (Refs. 1,2) as shown in Fig. A. A more detailed description and the location of these fatigue cracks in the tie plates is given in Refs. 1 and 3. The cracks started at the edges of the tie plates at a tack weld which connected the tie plates to the outrigger brackets during fabrication.

   The cracks were discovered after the twin bridges were subjected to nineteen years of traffic. U.S. Route 22 on which the bridge is located carries substantial amounts of heavy truck traffic.
2. FAILURE MODES AND FAILURE ANALYSIS

   . Location of Crack, Failure Modes and Stages:

   The approximate location and length of the fatigue cracks and stresses in the tie plates of one side span are shown in Figs. 3.a and b. Throughout the length of the Lehigh Canal and River Bridges, most of the tie plate cracks were at or near the outside edge of longitudinal or main girders (Ref. 1). Several of the plates had cracked across their entire widths. All observed cracks appeared to be through the thicknesses of the tie plates as shown in Figs. 3 and 4. All fatigue cracks initiated from the edge of tie plates where a tack weld was used to connect the plates to the outrigger or cantilever bracket during fabrication (Refs. 1,2). See, for instance, Fig. 4.

   A detailed study of the actual traffic conditions and the stress history measurements (Refs. 1, 2, 3) indicated that the fatigue cracks developed and propagated slowly from the ends of the tack welds, due to out-of-plane displacements at the top flange of the main girders. These displacements are shown schematically in Fig. 5. Corresponding strain measurements are illustrated in Fig. 6.

   . Cyclic Loads and Stresses:

   The stress range histograms for two tie plates and one of the main girders are given in Fig. 7. The Gross Vehicle Weight distribution of trucks crossing the structure in 1972 is given in Fig. 8.

   The total number of trucks that have traveled over the bridge during the nineteen year period between 1953 and 1972 was estimated from PennDOT traffic count data. The ADTT is plotted in Fig. 9. This ranged from
2,038 trucks per day (in 1953) to 3,709 (in 1972). The total volume from 1953 to 1972 was estimated to be 18.9 million trucks (Refs. 2, 3).

The measured stress range occurrences at the gages on the tie plates were used to determine an effective stress range using Miner's Rule and the RMS* methods. These were then adjusted to the plate edges where the crack growth originated.

The $S_{\text{rRMS}}$ and $S_{\text{rMiner}}$ stress range values at the plate edge and the fatigue cycles corresponding to the truck traffic of 18.9 million are plotted in Fig. 10. The data from the Lehigh Canal Bridge is seen to lie somewhat above the lower confidence limit for AASHTO Category D as shown in Fig. 10 (Refs. 2, 3). The tie plates that plotted above this limit would be expected to exhibit visible fatigue cracking.

The stress range values plotted in Fig. 10 demonstrate that Miner's Rule and the RMS method both provide a reasonable correlation between observed tie plate behavior and laboratory results.

The Live Load stresses in the main longitudinal girders were similar to those observed in longitudinal members of other bridges (the Maximum Live Load stress range was 55 MPa or 8 ksi). The live load stresses in the tie plates connecting the outrigger brackets and floor beams were much higher than those observed in the longitudinal member [this is mainly due to the out-of-plane displacements of the main girders (Refs. 1, 2)].

Temperature and Environmental Effects (Known or Field Recorded):

In this particular case there were no known temperature or other environmental effects that played a role in the cracking of the tie plates.

*Root-Mean-Square
Since the tie plate cracks were fatigue cracks resulting from cyclic loads and no sudden unstable crack propagation was observed, the tie plate fracture properties were not obtained.

Visual and Fractographic Analysis of Failure Surfaces:

The only fractographic studies of failure surfaces were visual and low magnification examination of the crack surfaces at the bridge site.

3. CONCLUSIONS AND RECOMMENDATIONS

The fatigue cracks that formed in the tie plates originated at tack weld ends where high stress ranges were introduced as a result of out-of-plane displacements. The cantilever load for which the bracket and tie plate were designed did not result in a significant stress from traffic. The connection of the plate to the longitudinal girders resulted in a secondary unaccounted for cyclic stress that resulted in fatigue cracks. Stress measurements indicated that the most severely stressed plates were near the reactions and that those particular plates had likely cracked after a few years of service.

Cracking of the tie plates did not seriously impair the performance of the structure as the web connection of the bracket was able to resist the applied loads.

It was recommended that the structure be retrofitted by removing the existing cracked and uncracked tie plates and replacing them with new plates that were not attached to the longitudinal girders.

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4. ACTUAL REPAIR AND FRACTURE CONTROL EFFORTS

The tie plates on the Lehigh Canal and River Bridges were replaced with new tie plates during the bridge deck replacement that was undertaken during 1977 and 1978. This involved removal of all existing cracked and uncracked plates and replacing them with the plate shown in Fig. 11.

5. POSSIBLE ADVERSE EFFECTS OF REPAIR

In the new tie plate configuration as shown in Fig. 11, the bolts connecting the tie plates to the main girders are eliminated. Thus, the out-of-plane displacements of main girders under Live Loads will not influence the tie plate behavior. Therefore, no adverse effects due to replacement of tie plates are expected.

6. ACTUAL OR ESTIMATED REPAIR/REHABILITATION COSTS

- Cost of Investigation:

The total cost of investigations of causes of cracking producing both the laboratory and field investigations is not available. However, the field investigation only was estimated to cost $4,000.00 by PennDOT.

- Cost of Repair/Rehabilitation:

It was estimated that $29,787.00 was required to make the necessary retrofitting and corrective action during the time period in which the reinforced bridge deck was also completely replaced.

- Original or Adjusted Cost of Structure:

The initial cost of the structure is not available, as it was built as part of a large section of Route 22 and only the total cost of the construction is available.
7. REFERENCES

1. Fisher, J. W., Yen, B. T. and Daniels, J. H.  
"Fatigue Damage in the Lehigh Canal Bridge from Displacement Induced Secondary Stresses"  
Fritz Engineering Laboratory Report No. 386.5, Lehigh University, Bethlehem, PA 18015, 1976.

2. Fisher, J. W.  
"Fatigue Cracking in Bridges from Out-of-Plane Displacements"  

3. Fisher, J. W. and Daniels, J. H.  
"Field Evaluation of Tie-Plate Geometry"  
Fritz Engineering Laboratory Report No. 386.4, Lehigh University, Bethlehem, PA 18015, November 1974.
Fig. 1.a Plan and Elevation of Lehigh Canal and River Bridge
Diagonal Braces

Floor Beam

Stringers

Long. Girders

5@5'-6" = 27'-6"
(5 @ 1676.4
= 8382)

8 Panels @ 18' = 144'-0"
(5486) (43891)

PLAN VIEW
(Deck Not Shown)

Measured ksi 21.3 13 14 11.4 8.4 19.3 22.3 26.5
Sr-HS20 Test Truck Mpa 147 90 98 79 58 133 154 183

ELEVATION VIEW

Abutment C

50'-0"
(15240)

8'-0½"
(2451)

Pier 1A

Fig. 1.b Plan and Elevation of the Side Span
Fig. 1.c Lehigh Canal and River Bridge (side view)
Fig. 2.a Cross-Section of Lehigh Canal and River Bridge

Fig. 2.b Tie Plate Detail at Floor Beam Bracket Connection to Girder
Fig. 3.a  Schematic Showing Typical Cracks in the Tie Plates of the West End of the Eastbound Span
Fig. 3.b Schematic Showing Stresses in the Tie Plates of the West End of the Eastbound Span
Fig. 4 Crack in Tie Plate Originating at Tack Weld
Fig. 5 Longitudinal Displacement at Top Flange of Girder and Horizontal Bending of Tie Plate
Fig. 6 Comparison of Measured Strain History in Tie Plates with Influence Lines for Girder Slope
Tie Plate C6N
Gage 9
78.65% : 0.9-3.0 ksi
6.2-20.7 MPa

Girder EN
Gage 15
34.37% : 1.5-1.95 ksi
10.3-13.4 MPa

Fig. 7 Stress Histograms
Fig. 8 Gross Vehicle Weight Distribution
Fig. 9 Estimated ADTT at the Lehigh Canal and River Bridge Site

Fig. 10 Comparison of Estimated Equivalent Stress Range with Laboratory Test Data
Refit Detail for Lehigh Canal and River Bridge
(Note the change in Tie Plate Detail and Configuration)
8.2 FATIGUE/FRACTURE ANALYSIS

OF

ALLEGHENY RIVER BRIDGE

-- A Brief Summary --

I.D. No. : 16

Town/City : near Pittsburgh, Pennsylvania

Railway/Highway/Avenue: Pennsylvania Turnpike

County

State : Pennsylvania

Owner : Pennsylvania Turnpike Commission
1. DESCRIPTION AND HISTORY OF THE BRIDGE

Brief Description of Structure:

The Allegheny River Bridge is a four-span continuous beam-girder bridge and a five-span truss bridge over the Allegheny River outside Pittsburgh, Pennsylvania on the Pennsylvania Turnpike. The bridge deck carries both eastbound and westbound traffic. The top view and side view of the bridge is shown in Figs. 1.

The plan and elevation of the end span and the second span of the four-span continuous girder bridge is shown in Fig. 2. A typical cross-section and the original tie-plate details for the floor beam bracket connection are shown in Fig. 3. The end span is 104 ft.-4 in" (31.822 m), and the second span is 130 ft.-5 in. (39.777 m) long. The end span contains a hinge 78 ft.-3 in. (23.866 m) from the west abutment, as shown in Fig. 2. The longitudinal girders are 7 ft.-0 in. (2.148 m) deep with a haunch at the piers of 9 ft.-1.5 in. (2.783 m).

The reinforced concrete slab in the end span is supported by twelve longitudinal W12X62 stringers resting on the floor beams and outrigger brackets. The floor beams are composed of built-up I-beams (web: 66 in. x 3/8 in. (168 cm x .10 cm), flanges: 6 in. x 6 in. x 0.5 in. (15 cm. x 15 cm. x 1 cm) angles with outrigger brackets (see Fig. 3). The second span has a similar load-carrying system.

The original tie plate detail [14 in. x 0.5 in. x 4 ft. - 8 in. (35.5 cm x 1.27 cm x 142 cm)] which connected the floor beam top flange to the outrigger bracket flange is shown in Fig. 3.b.
Brief History of Structure and Cracking:

The Allegheny River Bridge was constructed in 1952. During the fall of 1971, preliminary inspections by the Pennsylvania Turnpike Commission personnel revealed several fatigue cracks in the tie plates (1). In January 1972 all tie plates were again checked for fatigue cracks. The approximate locations and length of cracks in the end and second spans is given in Fig. 4. It was also observed that some of the rivets connecting the tie plates to the first inboard or to the first outboard stringer failed. These rivets are indicated by darkened circles in Fig. 4 (1). A typical cracked tie-plate is shown in Fig. 5.

In the spring of 1972, the original cracked tie plates were repaired with groove welds and reinforcement tie plates were added as well.
2. FAILURE MODES AND FAILURE ANALYSIS

   Location of Fracture, Failure Modes and Stages:

   The location and approximate lengths of the fatigue cracks in the tie plates in the end and second spans are shown in Figs. 4.a and 5. Most of the cracks were at or near the piers and abutments, i.e. near the supports. All cracks originated from rivet holes in the region where the tie plates were connected to the main longitudinal girders (see Fig. 4.a).

   A detailed study (1) of the traffic conditions and stress history measurements indicated that the fatigue cracks initiated and developed from the tie plate rivet holes due to horizontal in-plane bending of the tie plates. This horizontal bending was induced by the longitudinal displacement of top flange of the main girder under traffic loads. Similar cracks due to out-of-plane displacements were also observed in the Lehigh Canal and River Bridge on U.S. 22, near Allentown, Pennsylvania (2).

   As shown in Figs. 4.a and 5, some of the tie plate rivets (indicated by the filled circles in Fig. 4.a) became ineffective or failed completely.

   Cyclic Loads and Stresses (Known or Field-Recorded):

   A strain gage recording of the strains in the tie plates of the Allegheny River Bridge was undertaken from November 10, 1972 to November 17, 1972 (1). The position of strain gages are shown in Fig. 4.b. From these recordings several stress range histograms were developed. A typical stress range histogram for the tie plate (gage 18) is shown in Figs. 6.a and b (1). All tie plates provided stress ranges in the order of 12 to 18 ksi (83 to 124 MPa).
Stress distributions in the tie plates indicate that the cracked plates were subjected to high bending stresses in a horizontal plane, as shown in Fig. 7. The measured stresses suggested that the horizontal bending was induced by a relative movement between the girder and the ends of the tie plate. The stress range distribution caused by a truck at each of the gaged tie plates is shown in Fig. 8. Bending stresses in tie plates were higher near the abutment and pier. This agreed well with the cracked tie plate pattern shown in Figs. 4.a and 5.

Very low live load stresses were observed in the main girders and the floor beams. The main girder stresses as a result of an HS20-44 truck in one lane of the bridge provided a maximum live load stress of 2.4 ksi (16.54 MPa) and a stress range of 4.65 ksi (32.04 MPa). Both of these measurements were well below the calculated design live load stress 9.6 ksi (66.19 MPa) (1).

Traffic counts were also taken during the in-service testing in 1972. The results are given in the histogram of traffic flow in Fig. 9 and also in Fig. 10. The survey was comparable to the 1970 FHWA Nationwide survey (1). The total volume of truck traffic that crossed the bridge during the twenty years of service between 1952 and 1972 was 20.7 million vehicles. This corresponds to at least 20.7 million cycles of loading for each tie plate. The number of cycles for the truncated (SRM) root-mean-square stress ranges when stresses below the crack threshold are neglected are tabulated in Table 1. The estimated number of RMS stress range cycles for several tie plates are also plotted in Fig. 11 (1).
When all stress cycles were considered, the effective stress ranges decreased and are also plotted in Fig. 10 at $20.7 \times 10^6$ cycles.

The (RMS) and Miner effective stress range procedure was used to correlate the tie plate behavior with available test data. Table 1 and Fig. 11 both indicate that reasonable correlation was obtained between the observed behavior and the measured stress range spectra. Both the RMS and Miner effective stress ranges and the cumulative stress cycles suggested that fatigue distress should develop in the tie plates.

**Temperature and Environmental Effects (Known or Field-Recorded):**

In the case of the Allegheny River Bridge, there were no suspected or recorded temperature or other environmental effects that influenced cracking of the tie plates.

**Material "Fracture, Mechanical and Chemical Properties"**

**Analysis in Failure Regions:**

The tie plate cracks and failed rivets were all fatigue induced due to horizontal bending of tie plates. No sudden, unstable crack propagation was observed. The fracture toughness of the tie plate material was not determined.
Visual and Fractographic Examination of Failure Surfaces:

The failure surfaces were only examined visually. The crack surface showed clear evidence of fatigue crack growth.

Crack Growth (Initiation and Propagation) and Probable Causes of Fracture/Failure:

The fatigue cracks initiated and grew from the rivet holes in tie plates. Some of the rivets in the tie plates were cracked as a result of horizontal bending in the tie plates. The stress distribution pattern in tie plates indicated that horizontal bending of tie plates was induced by the relative longitudinal displacement of top flange of the main girder and the floor beam under cyclic live load as shown in Fig. 7 (1). The locations of cracked tie-plates are indicated in Fig. 4.a, and a typical tie plate with cracks initiated from rivet holes is presented in Fig. 5.
3. CONCLUSIONS AND RECOMMENDATIONS

Conclusions Regarding Fracture/Failure:

All fatigue cracks found in the tie plates of the Allegheny River Bridge originated from rivet holes; most of the cracks were located near the piers or supports and were formed where tie plates, connecting the brackets and floor beams, were riveted to the main longitudinal girders. The fatigue cracks developed due to the out-of-plane displacements which introduced high horizontal bending stresses in the tie plates (1). The connection of the plate to longitudinal main girders resulted in secondary cyclic stresses causing the fatigue cracks in the rivet holes.

Stress relief in tie plates was accomplished naturally under service conditions through the shearing of some of the rivets as well as by the fatigue fracture of the tie plates (1,2). Fatigue cracks in tie plates have occurred in other bridges under similar circumstances*.

4. ACTUAL REPAIR/FRACTURE CONTROL EFFORTS

During the spring of 1972, reinforcement tie plates [17 in. x 0.5 in. x 4 ft. - 7.94 in. (431.8 mm x 12.7 mm x 1420.9 mm)] were added to the structure. The original cracked tie plates were first groove weld repaired at the crack locations, and the reinforcement plates were placed over them. The bolt holes in the reinforcement plates were matched to those of original tie plates for ease of installment. The reinforcement plates were high strength bolted to the original tie plates, girders, floor beams, outrigger brackets, and the neighboring stringers as shown in Fig. 12. Also, the lock nuts are tack welded.

* see FATIGUE/FRACTURE OF LEHIGH CANAL AND RIVER BRIDGE -- A Brief Summary --, Lehigh University, Fritz Engineering Laboratory Report 448, 1981. -221-.
5. POSSIBLE ADVERSE EFFECTS OF REPAIR

From the approximate analytical and field measurements given in Ref. 1, it can be concluded that increasing the width of the tie plates will cause the horizontal bending moment in the tie plate to increase due to the elongating and shortening of the girder. Also, depositing tack welds at the lock nuts decreased the fatigue resistance.

Since the cause of longitudinal displacement at the tie plates has not been eliminated, horizontal bending stresses will continue to be developed at the edge of reinforcement plates. Tack welds are expected to initiate fatigue cracks. Similar cracks developed from tack welds and were observed in the Lehigh Canal and River Bridges (2,3).

6. ACTUAL OR ESTIMATED REPAIR/REHABILITATION COSTS

At the time this "Brief Summary" was being prepared no estimates regarding the cost of investigation of cracking, costs of retrofit and the original cost of the bridge were available.
7. REFERENCES

(1) Marchica, N. V.

(2) Fisher, J. W.

(3) Daniels, J. H. and Fisher, J. W.
FIELD EVALUATION OF TIE-PLATE GEOMETRY, Lehigh University, Fritz Engineering Laboratory Report No. 386.4, November 1974.
## TABLE 1  RMS AND MINER STRESS RANGE AND CYCLE DATA

FOR TIE PLATES

<table>
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<tr>
<th>Plate</th>
<th>Gage</th>
<th>Adjusted SrRMS at Rivet Hole ksi (MPa)</th>
<th>Adjusted SrMINER at Rivet Hole ksi (MPa)</th>
<th>Cumulative Frequency (% above Threshold)</th>
<th>Number of Stress Cycles x 10^6</th>
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<tr>
<td>ABUT</td>
<td>3</td>
<td>6.4 (44.1)</td>
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<td>ABUT LT</td>
<td>11</td>
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<td>8.4 (57.9)</td>
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Fig. 1 Side View of Allegheny River Bridge
Fig. 2 Plan and Elevation of Allegheny River Bridge - Test Spans
Fig. 3.a Bridge Cross Section

Fig. 3.b Original Tie-Plate Detail
Fig. 4.a Cracked Tie Plates and Loose Rivets in Test Spans

Fig. 4.b Strain Location and Identification
Fig. 5  A Typical Cracked Tie-Plate of Allegheny River Bridge
Fig. 6.a Stress Histogram for Gage 18 (Tie Plate RT-5)

GAGE 18
0.9-3.9 ksi: 77.99%
3.9-14.4 ksi: 22.01%

Fig. 6.b Stress Histogram for Gage 15 (Tie-Plate RT-3A)

GAGE 15
0.9-3.3 ksi: 74.37%
3.3-11.7 ksi: 25.62%
Fig. 7 Stress Distribution in Tie-Plate R-5

Fig. 8 Instantaneous Stress Pattern in Gaged Tie Plates
Fig. 9 Histogram of Traffic Flow of Allegheny River Bridge During Test Period

(Eastbound Lanes) (Westbound Lanes)
Fig. 10 Results of Loadometer Survey

(Loadometer Survey 1972 PennDOT, 20 Stations)

(Loadometer Survey - 1970 FHWA, Nationwide)
Fig. 11 Tie-Plate Response and Laboratory Fatigue, Test Results
Fig. 12 Reinforcement and Original Tie-Plates
(Bottom View)
Cracks were found during construction at the ends of transverse stiffeners on several plate girder bridges. A detailed summary of two Cuyahoga County bridges (I.D. 41 and I.D. 42) is given hereafter.

The results of the investigation of this cracking showed that the primary cause was out-of-plane movement of the short web gap at the end of the stiffeners and the flange.

Similar cracking at the ends of transverse stiffeners was detected in other bridges (I.D. 114). In this case the cracks were found to have formed in the fabricating shop, while the girders were handled (I.D. 114).

Cracking of a similar nature was found in web gaps of intersecting vertical and transverse stiffeners. These cracks occurred as the large web plate was moved and turned in the shop (I.D. 53).

In all of these cases handling resulted in high cyclic stresses in the girder webs at the short gap regions. These cyclic stresses either occurred as the girders were handled in the shop or developed as the girders were shipped. The swaying motion of the girders supported on rail cars created high repeated stresses at the weld terminations in the web gaps.
9.1 FATIGUE/FRACTURE ANALYSIS

of

CUYAHOGA COUNTY BRIDGES (CUY-90-1365L)

-- A Brief Summary --

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1. DESCRIPTION AND HISTORY OF THE BRIDGE

Brief Description of the Structure

The Cuyahoga County Bridge (CUY-90-1365) carries Interstate I-90 traffic over the Penn Central Railroad yard tracks in Cleveland, Ohio (Fig. 1a). The superstructure in each traffic direction is composed of nine continuous, built-up steel girders with an 8-1/2 in. (215 mm) reinforced concrete slab extending over several spans. The eastbound and the westbound traffic is separated by a concrete guard wall placed along the centerline of the roadway.

The structure has three main spans with lengths of 135 ft. (41 m), 140 ft. (43 m) and 93 ft. (28 m). The two side spans are 88 ft. (27 m) and 62 ft. (19 m) long. The plan of the structure and the elevation is given in Figs. 1a and 1b. The main girder arrangements, the transverse and lateral bracing, and the cross-sections are given in Figs. 2, 3 and 4.

The structure was designed in accord with the 1965 AASHO Specifications including Ohio-DOT Supplements. The design loading was the HS 20-44 truck and the Interstate Alternate Loading. The mainline ADT in 1970 was 50,738. The structural steel is ASTM A36.

Brief History of Structure and Cracking

On May 1-3, 1972, during the erection of steel girders on the eastbound portion of the Cuyahoga County Bridge (CUY-90-1365) an inspection by Ohio-DOT personnel revealed cracks on some of the erected girder webs on the eastbound bridge near the abutments. All of the steel superstructure for the eastbound structure and the first span of the westbound bridge were erected. Erection of the remaining steel was stopped so that a
thorough inspection and rechecking of all girders by MPI (Magnetic Particle Inspection) and by dye penetrant could be made. The cracks were of two kinds as shown in Figs. 7 and 8.

A sample plug was removed from Girder 138G1 at stiffener 13. This sample plug contained both types of cracks. Figure 9 shows one side of the web prior to the removal of the sample core. The crack which formed in the girder web at the end of the weld toe can be seen in Fig. 10. Figure 11 shows a cut through the core sample with a polished and etched surface of the half section. Both types of cracks are visible in the photograph. The cracks tend to turn and move up the web as they increase in size.

The field examination of other girders with cracked details exhibited similar characteristics.

All evidence from the field inspection and subsequent laboratory studies on the sample core removed from Girder 138G1 indicated that the cause of cracking was cyclic loading. The cracks were fatigue cracks. No brittle behavior or cleavage fracture was observed. Relatively large out-of-plane cyclic web bending stresses were introduced into the girders during rail transportation on a flat car (see Fig. 12) from Columbus, Ohio, to the bridge site. The cyclic displacement resulted in high stress range excursions in the web gap of stiffeners adjacent or at the support points.

Cyclic Loads and Stresses (Known or Field-Recorded)

The cracked girders were shipped after fabrication from Columbus, Ohio, to the Cuyahoga County Bridge site by rail and truck transportation. The girders shipped by rail were arranged together as shown schematically
in Fig. 12. Most girders were supported at two locations using wooden blocks and tie down rods.

The cracks were only observed in the girders which were shipped to the construction site by rail. There were no detectable cracks on the girders shipped by truck. No defects were found when the web gap at the stiffener opening was near the top flange. The cracks on the rail transported girders all occurred at stiffener locations approximately 25-30 ft. (8-9 m) from the center of the girder. This placed the cracks at stiffeners adjacent to the wooden supports on which the girders rested during transport on the gondola car.

The crack locations are indicated in Figs. 5 and 6 (3).

Several core type specimens were removed from cracked regions. Rectangular specimens were removed from the web near the abutments in order to determine the fracture properties of the steel. These specimens were shipped to Lehigh University for testing and evaluation (4).

2 FAILURE MODES AND FAILURE ANALYSIS

Location of Fracture, Failure Modes and Stages

The cracks were only observed in positive moment regions at the end of stiffeners cut short of the bottom flange. When cracks occurred in stiffener details cut short of the bottom flange, they were always observed to be near the girder support points during the rail transportation. The stiffeners were attached to both sides of the web which resulted in cracks on both sides. Two types of cracks were found.
1. Cracks at the end of the stiffener weld wrapping around the weld toe and extending parallel to the longitudinal axis of the girder as shown in Fig. 7.

2. Cracks at the end of the stiffener in the weld which sometimes extended into the web as illustrated in Fig. 8.

Since only vertical tie down rods were used to support the girders, they offered little restraint to lateral movement of the girder sections. Consequently, local twisting of the girder flanges at the girder supports would result in relatively high out-of-plane web bending strains in the web gap at the cut-short stiffeners near the support points (4). This cyclic out-of-plane movement of the web and the resulting cyclic stresses caused the development of fatigue cracks at the stiffener ends near the wooden blocks. The out-of-plane movement of the web is shown schematically in Fig. 13.

Temperature and Environmental Effects (Known or Field-Recorded)

In the case of the Cuyahoga County Bridges, the temperature and the environmental effect did not contribute to the cracking of the girder webs at the end of transverse stiffeners.

Material "Mechanical, Chemical and Fracture Properties" Analysis in the Failure Region

The material used in the girder webs of the Cuyahoga County Bridges is the ASTM A36 steel. Mill reports indicated that the steel met the mechanical requirements of the ASTM standards.
The fracture toughness of a typical girder web was determined by making transverse Charpy V-notch (CVN) tests. The small section removed from Girder 172G1 was used to fabricate CVN specimens which are summarized in Fig. 14. At 40°F (4°C), the average absorbed energy was 16.8 ft-lb. (23 Nm). The results indicated a good toughness in the girder web material (4).

Fatigue crack formation in the girder webs without evidence of rapid fracture was compatible with the measured toughness of the girder web steel (4).

Visual and Fractographic Examination of Failure Surfaces

The fracture surface topography from the sample plug shown in Fig. 10 was examined under the electron microscope. Figure 15 shows the fatigue striations that were detected in the web at the weld toe of the far side stiffener (4). The fatigue crack had propagated 0.2 in. (.5 mm) into the web at the weld toe. The striation spacing varied between 5-15 micro-inches (0.1-0.4 micro-meters).

The fatigue crack surface in the weld on the near side stiffener had numerous oxides and other impurities due to the atmospheric influences which prevented detection of fatigue striations (4).

The striation spacing observed in the base metal near the surface of the web indicates relatively high web bending stresses of variable amplitude. After the cracks propagated a few hundredths of an inch, the crack surface exhibited the characteristics of high stress fatigue. The striation spacing suggested that between 1000-10,000 cycles would be needed to propagate the cracks detected at Stiffener 13 on Girder 138G1 (4).
3. CONCLUSIONS AND RECOMMENDATIONS

Conclusions Regarding Fracture/Failure

The cracks found at the ends of transverse stiffeners cut short of the tension flange on continuous plate girders for the Cuyahoga County Bridges (CUY-90-1365-L&R) were fatigue cracks. No rapid fractures were observed. All cracks were caused by large cyclic web bending stresses in the short length of the girder web, the stiffener end, and the flange-to-web fillet weld.

The cracks were only found in the positive moment regions of the girders. All cracking was confined to the stiffeners located near the wooden blocks used to support the girders during rail transit to the bridge site. Relatively small out-of-plane movement of girders during transit introduced large bending strains at the end of the cut-short stiffeners in the girder web near the wooden supports, resulting in the fatigue cracks.

The cracks were all parallel to the bending stresses that would occur in the webs.

Recommendations for Repair/Fracture Control

The following repair procedures were used in order to prevent future crack growth during the service life of the Cuyahoga County Bridges (4):

1. Cracks in Stiffener Welds Only

   Cracks that propagated into the weld metal near the stiffener end were repaired by grinding out the crack and the weld metal around it. The ground out region was checked by nondestructive testing in order to insure the complete removal of crack tips.
2. Cracks Propagated into the Web Plate

These cracks either initiated from the weld toe termination at the end of the stiffener or propagated through the weld into the web. They were retrofitted by drilling 7/16 in. (11 mm) holes at the end of each crack and grinding the cylindrical surface of the hole smooth as shown schematically in Fig. 16.

The sample core holes that were cut into the girders were ground and any crack tip that extended beyond the core hole was removed as well.

4. ACTUAL REPAIR/FRACTURE CONTROL EFFORTS

The cracks were removed in the field according to the recommendations given above. The holes at the ends of the cracks were filled with high strength bolts which were tightened to a high preload.
5. **POSSIBLE ADVERSE EFFECTS OF REPAIR**

None of the cracks observed were detrimental to the safety of the bridge during construction. The cracks were all parallel to the stresses caused by dead and live loads. Repairs were made during construction and should insure that further crack growth does not develop during the service life of the structure. No adverse effects are expected from repairs.

6. **ACTUAL OR ESTIMATED REPAIR/REHABILITATION COSTS**

The bid price of the Cuyahoga County Bridges was $2,203,883.10. The bid price for the structural steel, excluding field painting was $924,056.

The investigation of the fracture including the repair of the cracked girder webs was $12,335.67 in 1973. No costs are available on the repairs.
REFERENCES

1. Ohio-DOT Internal Memo dated May 7, 1973 from J. D. Jones Shop Supervisory Eng. to R. B. Pfeifer Engineer of Bridges.

2. Ohio-DOT Inter-Office Communication dated May 11, 1973 from R. Van Horn, Structural Steel Engineer to R. B. Pfeifer, Engineer of Bridges.

3. Fort Pitt Bridge Co. Canonsburg, Pa Memo dated 6/13/73 from R. Osmond to W. H. Wallhauser (on Field Inspection of Open Ended Stiffeners)

Fig. 1a General Plan and Side View of Cuyahoga County Bridge (CUY-90-1365 L)
Fig. 1b  Elevation of Cuyahoga County Bridge (CUY-90-1365 L)
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Fig. 3 Typical Girders and Stiffeners of Cuyahoga County Bridge (CUY-90-1365 L)
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(Small circles indicate cracked Stiffeners)
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(Small Circles Indicate Cracked Stiffeners)
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Fig. 8 Schematic of Crack Growth at End of Stiffener into Weld (and Web)
Fig. 9 Photograph of Stiffener 13 on Girder 138 Gl
Prior to Removing Sample Plug.

Fig. 10 Sample Plug from Girder 138 Gl Showing Crack in Weld at End of Stiffener -- Near Side

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Fig. 11 Macro-etched Section of Sawcut Showing Cracks into Web and Weld
Fig. 12 Tie Down Arrangement for Moderate Size Girders During Rail Shipment
Fig. 13 Schematic of Deformation of Girder Web at Support Block when Cut Short Stiffeners are Adjacent to Block

Fig. 14 Transverse Charpy V-Notch Data for Web Plate of Girder 172 Gl
Fig. 15  Fatigue Crack Striations in Web Plate @ 7500X
Fig. 16 Retrofit Detail for Two Types of Cracks at the Bottom End of Stiffeners
9.2 FATIGUE/FRACTURE ANALYSIS

of

CUYAHOGA RIVER BRIDGE (No. CUY-80-1843)

-- A Brief Summary --

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1. DESCRIPTION AND HISTORY OF THE BRIDGE

   Brief Description of the Structure

   The Cuyahoga River Bridges are twin structures on Interstate I-480 carrying east and westbound traffic on separate roadways over the Cuyahoga River, the Ohio Canal, the B & O Railroad and abandoned Penn Central Railroad tracks. The twin bridges are located near Independence, Ohio. The total length of each bridge is 4150 ft. (1266 m), and the width is 71 ft. (22 m) over 15 spans. The spans adjacent to the west and east abutments are 220 ft. (67 m) and 180 ft. (55 m), respectively. Eleven inner spans are 300 ft. (92 m) long, and the two remaining spans are each 225 ft. (69 m) long. The overall view of the bridges is given in Fig. 1.

   Each roadway is supported by four continuous main girders with two stringers resting on a transverse beam between girders. The main girders are about 10 ft. (3 m) high and shop-welded with field assembled bolted splices. The main girders are haunched at the supports between west abutment and the pier 11. The schematic plan and profile of the twin bridges are given in Fig. 2. The typical main girders and the bridge cross-sections are shown in Figs. 3a and 3b, respectively.

   The structure was designed to satisfy the 1965 AASHO Specifications including 1966-1967 Interim Specifications and the Ohio "Supplement" to these specifications. The design loading is HS-20-44 and Interstate Alternate Loading.

   Brief History of Structure and Cracking

   In April 1972, while erection of structural steel was progressing from west to east, the inspection of a wind damaged girder revealed a crack in the
web at the open end of an intermediate stiffener. This resulted in examination of other girders on the ground and piers resulting in the discovery of several cracks near the bottom and of stiffeners cut short of tension flange (1).

All cracks were found to be fatigue related (2). They were caused by the relative out-of-plane movements and bending of the short length of web between the stiffener and the web-to-flange fillet weld (2,3). The relative movement is believed to be caused by the cyclic swaying motion of the girders while in transit to the construction site and/or wind induced motion during storage on the ground. A typical crack is shown in Fig. 4.

2. FAILURE MODES AND FAILURE ANALYSIS

   Location of Fracture, Failure Modes and Stages

Several girders with the cracked stiffener details were examined on April 17, 1973 at the Cuyahoga River Bridge site (2,3). The results of the examination of a typical girder is summarized in Fig. 5. The location of a sample plug taken out of Girders 1204CL is also shown in Fig. 5 with a small circle at the end of the stiffener. The field examination of other girders showed that the cracks indicated in Fig. 5 are typical for all the cracked girders (2).

The field examination and detailed evaluation of the sample taken from the main girder plugs indicated that all cracks were fatigue cracks. They were grouped into four categories as follows (2,3).

   (1) Cracks between the Stiffener Weld and Web: This type is shown schematically in Fig. 6. The plug removed from Girder 1204CL is shown in Figs. 5, and Figs. 7 and 8 exhibit this kind of crack. Cyclically applied loads caused fatigue cracking which peeled the weld away from the web surface.
(2) **Cracks across the Weld:** These cracks were observed at the ends of many stiffeners and, in some cases, did not propagate into the web. This type of crack is shown schematically in Fig. 9 and is also visible in the sample plug in Fig. 7. The polished and etched surface shown in Fig. 10 indicates the crack was started in the heat affected zone.

In several instances, the cracks in the weld at the end of the stiffener extended completely across the weld and penetrated into the web as shown schematically in Fig. 9. All cracks exhibited characteristic of fatigue crack propagation.

(3) **Cracks at Fillet Weld Toes:** These cracks formed at the weld toes at the end of stiffeners adjacent to the tension flange (see Fig. 5). This type of crack is shown schematically in Fig. 11. The sample plugs indicated that the crack had propagated into the web at the end of the fillet weld and turned and moved up the web after a short distance into the web as shown in Fig. 12.

(4) **Cracks in Web Surface Opposite Stiffener:** Several surface cracks were observed in the web opposite the stiffener. The sample plug removed from Girder 2406AR contained this type of crack. This crack was initiated on the net surface and did not join the crack propagating into the web from the end of transverse stiffener detail. Crack initiation from the web surface opposite the stiffener indicates the presence of large cyclic strains on the web surface.

Eighty to ninety percent of the cracks observed in the main girders of the Cuyahoga River Bridge were type 1. The remaining cracks were distributed among other types described. The cracks detected in the girder at the stiffener details were a new phenomena at the time they were observed. This type of cracking had not been detected prior to their occurrence in Ohio.
All macroscopic visual evidence from the field inspection and subsequent laboratory examination of the crack surfaces of the sample cores removed from the girders indicated that the cause of crack formation and propagation was cyclic loading.

- Cyclic Loads and Stresses (known or field-recorded)

It is known that the main girders of the Cuyahoga River Bridge were shipped from Chicago to the bridge site by rail transportation (2,3). Because of their size a single girder was transported on three flatbed cars as shown in Fig. 13. All girders were supported at two locations using the tie-down arrangement given by section A-A in Fig. 13. The two supports were either located on the first and third flat cars or both supports were placed on the center flat car over the axles. One inch steel rods were used to provide longitudinal restraint to movement. This restraint system permitted relative movement between the top and bottom flange during rail shipment. The relative movement was caused by swaying of the girder and resulted in large cyclic out-of-plane bending stresses in the short length of web between the end of the stiffener and the flange-to-web fillet welds. Similar cyclic stresses could have occurred due to wind induced swaying motion during storage. The cyclic motion of the girder cross section is shown schematically in Fig. 14.

Thus a condition was created which resulted in large cyclic strains and fatigue crack propagation.

- Temperature and Environmental Effects (known or field-recorded)

Temperature effects, if any, did not contribute to the fatigue cracks in the stiffener details.
The cyclic out-of-plane movements of girder webs in the narrow gap between the end of the stiffener and the bottom flange would also have been caused by wind induced vibrations during storage.

Material "Fracture, Physical and Chemical Properties Analysis" in Failure Regions

The web plates were fabricated from ASTM A588 steel. Mill test reports indicated that the steel met the physical and chemical requirements (3,4). Longitudinal Charpy V-notch (CVN) tests at 40°F (4°C) were reported for information from the heaviest gage plate in a lot. The mill reports show absorbed energy that varied from a low of 24 ft-lbs. (33 Nm) to a high of 130 ft-lbs. (176 Nm) (3). Since the web plates were 123 in. (3124 mm) wide, they had been cross-rolled, and hence, the CVN values are indicative of the toughness in both the transverse and longitudinal directions (3).

The cracks in webs did not show any evidence of cleavage or brittle fracture.

Visual and Fractographic Analysis of Failure Surfaces

None of the fracture surfaces exhibited any cleavage or brittle fracture characteristics.

The fractographic examination of failure surface revealed the presence of striations which verified the existence of fatigue crack propagation (3). Figure 15 shows the fatigue striations found using the electron microscope on the surface of the cracked weld shown in Fig. 7. Figure 15 shows the striations in Region 1 adjacent to the stiffener near the point of crack initiation at a magnification of 19000X. The striation spacing is roughly 10-15 micro-inches (2.5-4.0 micro-meters) and height of the photograph is about 0.0001 in. (.0025 mm).
On the basis of the striation spacings observed on the surface of several cracks between 1000 and 10,000 cycles of stress were needed to propagate the crack (3).

Crack Growth (Initiation and Propagation) and Probable Causes of Fracture/Failure

The field inspection, visual and fractographic examination of the crack surfaces of sample cores showed that the cause of crack formation was cyclic loading or fatigue (3). Fatigue cracks developed at the end of stiffeners as a result of large cyclic out-of-plane bending stresses in the short gap between the stiffener end and the flange-to-web fillet weld. The small out-of-plane movements of the top flange relative to the bottom flange due to cyclic swaying motion of the girder occurred during rail transportation to the bridge site and/or wind induced motions while in storage. The effects of the relative lateral deformation between the two flanges would be concentrated in the gaps between the end of the transverse stiffener and web-to-flange fillet weld (see also Fig. 14).

The stiffener welds were terminated very close to the web-to-flange weld. The residual stresses due to stiffener-web welds, will create high tensile residual stresses between the stiffener end and the flange (3). The presence of residual tensile stresses in the gap means that each cycle of web bending stress is effective in causing crack growth at the ends of the stiffeners (2,3). The cracks followed the direction of the cyclic bending stress and propagated in planes parallel to the longitudinal direction of the girder.
3. CONCLUSIONS/RECOMMENDATIONS

- Conclusions Regarding Fracture/Failure

The cracks at the end of stiffener welds in the main girders of the Cuyahoga River Bridge exhibited the characteristics of fatigue cracks. No cleavage or brittle crack propagation was observed (3). All cracks were caused by large cyclic out-of-plane bending stresses in the gap between stiffener ends and the flange due to the swaying motion of the girder during rail transit to the site and wind induced motions during storage (2,3).

The cracks are primarily parallel to the longitudinal direction of the girder and to the nominal bending stresses. The high toughness of web material and the orientation of fatigue cracks indicate that there is no danger of sudden brittle fracture of the main girders. Construction and erection loads should have no effect on the cracks (2,3). Hence, it was concluded that repairs can be made while the erection and construction are continued.

- Recommendations for Repair/Fracture Control

In order to prevent possible subsequent crack growth during the service life of the Cuyahoga River Bridge the following repair procedures were used (3).

1) Cracks between Stiffener Welds and Webs: Since these cracks are parallel to the stress field, then need not be repaired. The short length of the weld cracked from the web at the end of the stiffener has no significant influence on the performance of the web stiffeners and there will not be any crack propagation under subsequent traffic loads.

2) Cracks in the Stiffener Welds: The cracks that have propagated into the weld near the stiffener end should be repaired by grinding out the crack. The ground out region should be checked by non-destructive inspection to insure removal of the crack.
(3) and (4) **Cracks Propagated into the Web Plate:** These cracks are of concern during the service life of the structure and further propagation in the web or into the flange must be prevented.

This can be accomplished by drilling 7/16 in. (11 mm) holes at the end or the tip of each crack as shown in Fig. 16d. The cylindrical surface of the hole should be ground smooth.

**Sample Plug Holes:** The holes cut into the web to prevent removal of the sample plugs will serve as crack arrestors and should remain provided the crack tip has not extended into the web plate. To eliminate any remaining crack tips, holes can be drilled adjacent to the plug holes as shown in Fig.16d. These holes can be ground smooth, painted, and filled up with special non-corrosive filler.

4. **ACTUAL REPAIR/FRACTURE CONTROL EFFORTS**

Ohio-DOT and the contractor of the Cuyahoga River Bridge superstructure concurred with the recommendations given in (3) regarding the repair and fracture control.

5. **POSSIBLE ADVERSE EFFECTS OF REPAIR**

No adverse effects are expected.

6. **ACTUAL OR ESTIMATED REPAIR/REHABILITATION COSTS**

The original total cost of the structure (or total bid price) is $19,747,802.36 in 1972 which includes the superstructure only. The substructures were contracted separately.

The total cost of the field inspection and repair was $169,495.28.
REFERENCES

1. Inter-Office Communication, Ohio-DOT (from J. D. Jones, Shop Supervisory Engineer to B. Pfeifer, Engineer of Bridges, dated April 4, 1973.


3. Fisher, J.W., Yen, B.T. and Pense, A.W. "Report on Structural and Fractographic Investigation of Cracked Stiffener Details on Girders for Bridge over Cuyahoga River (No. CUY-80-1843 L&R)".

Fig. 1 General View of Cuyahoga River Bridge (No. CUY-80-1843)
Fig. 2 Plan and Elevation of Cuyahoga River Bridge (No. CUY-80-1843)
Fig. 3.a Typical Main Girders
Fig. 3.b Typical Cross Sections and Diaphragms
Fig. 4 Typical Cracks at End of Transverse Stiffener and Stiffener-to-Web Fillet Welds
Longitudinal stiffeners at D/5.
2 Transverse stiffeners on both sides at CB, near side only at other locations.
3 9-in. weld return at end of stiffener web weld.
4 All cracks are only visible on the near side of web. No through cracks observed.

Fig. 5 Typical Cracks in Girder 1204 CL of Cuyahoga River Bridge (CUY-80-1843)
Fig. 6 Schematic of Crack Growth between Transverse Stiffener Weld and Web -- Crack Type (1)
Fig. 7 Sample Plug from Girder 1204 CL
(Note crack in weld and crack between weld and web)
(Note Fig. 5 for location of sample plug)

Fig. 8 Macro-etched Section of Stiffener Weld
showing crack between weld and web -- Crack Type (1)
Fig. 9 Schematic of Crack Growth at End of Stiffener into Weld and Web
-- Crack Type (2)
Fig. 10 Macro-etched Section of Weld and Web
Showing Crack Surface in Weld (see also Fig. 7)
Fig. 11  Schematic of Crack Growth into Web at End of Stiffener Welds
-- Crack Type (3)
Fig. 12 Macro-etched Section of Right Portion of Saw Cut Sample Plug from Girder 1102G
(Note Crack Growth Into and Up Web)
Note: Two point support for transportation. AA above wheels of two end cars or the middle car.

Fig. 13 Tie Down Arrangement for Larger Girder During Rail Shipment
Fig. 14 Out-of-Plane Motion of Girder Web

Relative Moment of Top Flange

\[ \pm \Delta \]

120" [Fig. 14]

Stiff.

Truncations of Stress \( \perp \) to Crack

Enlarged

Web Bending Shown Exaggerated

Flange
Fig. 15  Fatigue Crack Striations in Region 1  
(see sketch and Fig. 7) --
Height of the photo about 0.0001 in. - 19000X
FIGURE 18 SCHEMATIC OF SUGGESTED REPAIR PROCEDURES

Fig. 16 Schematic for Suggested Retrofit/Repair Procedures
One of the most common sources of fatigue cracks during the past seven or eight years is the cracking in the web gaps at the ends of floor beam connection plates. These plates have been welded to the web and often the compression flange of the main longitudinal girders, but have not been attached to the tension flange. At least 28 bridge structures have developed cracks at these connections.

Distortion of the transverse connection plate web gap in the longitudinal girders occurs as a result of deformation in the floor beam. The floor beam end rotation caused by bending of the floor beam and the relative end movements creates large cyclic stress conditions that leads to rapid development of fatigue cracks when the structure is on a high volume road. Even less heavily traveled bridges crack with time.

A detailed discussion of two typical cases is provided hereafter for bridges I.D. 13 and I.D. 58. The Poplar Street Bridge (I.D. 13), was one of the first bridge structures that exhibited cracking in the web gap at floor beam connection plates.

After discovery of similar web cracks in floor beam-girder bridges in 1978, Iowa DOT conducted a detailed survey of 39 structures with comparable details. The Polk County Bridge near Des Moines (I.D. 58) is typical of the type of structure and illustrates the cracking that developed and the retrofit procedures used to correct the condition. Twenty-one of the 39 structures in Iowa were found to have some evidence of cracking.

Cracks have been found in at least six states with this type of bridge and general detail at the floor beam girder connections.
10.1 FATIGUE/FRACTURE ANALYSIS OF "POPLAR STREET COMPLEX" BRIDGES -- A Brief Summary --

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1. DESCRIPTION AND HISTORY OF THE BRIDGE

   . Brief Description of Structure

The Poplar Street Complex Bridges are located on the east bank of the Mississippi River in East St. Louis, Illinois. The complex is one of the largest and busiest interchanges in the State of Illinois. It serves as a focal point for Interstate Highways I-55, I-70 and U. S. Route 40 which join and cross the Mississippi River. It starts with the Poplar Street Bridge over the Mississippi River on the west and ends northeast of Broadway in East St. Louis, as shown in Fig. 1. It consists of several ramps and viaducts consisting of multispan continuous two girder type of bridges, as shown in Fig. 2.

The majority of the two-girder I-beam bridges in the complex are on horizontal curves with radii approximately 1800 ft. (550 m). The torsional and side-sway (transverse) rigidity of the system are provided by closely spaced transverse floor beams which are rigidly connected to the two main girders. The floor beams support the longitudinal stringers and the reinforced concrete deck, as shown in Fig. 3.

   . Brief History of the Bridge and Cracking

The Poplar Street Complex Bridges were designed in 1964. The entire complex was designed by a consulting engineering firm for the Illinois Department of Highways Bridge Division. The design of the welded details and transverse stiffeners conformed to the AASHO and State of Illinois design specifications and current practice in effect in 1964 and throughout the design period. The structures were built between 1967 and 1971.
In late 1973, the complex was inspected in depth for the first time by a bridge inspection team from the Illinois Department of Transportation. During this inspection, several types of cracks and web buckling were discovered and reported (1). The fatigue cracks consisted of the following conditions (1,2,3):

- Fatigue cracks in the bottom "web gap" region of main girders at floor beam connection plates
- Fatigue cracks in the upper "web gap" region of main girders at floor beam connection plates
- Fatigue cracks in the girder webs at the ends of bearing stiffeners of the main girders.

These latter cracks were located at the ends of the continuous main girders. In addition to these cracks, there were instances of web buckling at the girder end supports a few inches above the bottom flange (1,2,4).
2. FAILURE MODES AND FAILURE ANALYSIS

- Location of Fracture, Failure Modes and Stages

The general location of the fatigue cracks were predominantly near the supports or adjacent to the piers of the bridges in the complex. These cracks can be grouped into three general types as follows (2,3,4):

1. Fatigue cracks in the web in the bottom "web gap" region between the lower end of the floor beam-to-main girder connecting plate and the bottom flange of the main girder (web cracks in lower or bottom "web gap" region) (see Fig. 4).

2. Fatigue cracks in the web in the upper "web gap" region between the top end of the floor beam to girder connecting plate and the top flange of the main girder (web cracks in the top or upper "web gap" region) (see Fig. 5).

3. Fatigue cracks at the ends of bearing stiffeners which were also used as floor beam-to-girder connection plate.

Types 1 and 3 web cracks were near the end supports of the main girders. However, Type 2 web cracks were only observed at the upper end of the floor beam connecting plates which were located in the negative moment regions. There were no web cracks detected in the positive moment regions of the spans.

Type 1 web cracks were first observed at the end of the girder under the end of floor beam connecting plate which was positioned 7 in. (18 cm) toward the center of the span from the center of the bearing stiffeners. Most of these cracks started near the lower end of the vertical connecting
plate and extended in both directions in the girder web as shown in Fig. 4. The longest crack was 19 in. (48 cm) long. Occasionally two cracks were observed at the same location; one immediately under the lower end of connecting plate, the other among the edge of the web-to-flange weld toe. The former was shorter in length than the latter. The Type 2 web cracks in the upper gap regions were found in the negative moment regions (see Fig. 5) where the floor beam connection plate was not welded to the tension flange.

The Type 3 cracks occurred at bearing stiffeners which were also used as a connecting plate for the end floor beam at a few supports or piers. These cracks were observed at the top end of the bearing stiffener which was not welded to the top flange.

In addition to the fatigue cracks, web buckling was also observed at the end of several girders. This buckling of the web occurred a few inches above the bottom flange and was often associated with the separation of the top bearing plate from the bottom flange of the girder due to the seized expansion bearings. In some cases, web cracking developed above the web-to-bottom flange weld toe. This crack started at the end of the girder and progressed horizontally toward the bearing stiffeners.

. Cyclic Loads and Stresses (Known or Field Recorded)

The mainline I55 and I70 east and westbound structures were subjected to 5250 ADTT (multi units per structure) in 1980. Several of the ramps experience about 200 ADTT. The ADTT has varied between 4000 and 5250 since the mainline structures was opened in 1967.
In order to determine which mode of displacement was causing the web cracking, a series of field measurements were conducted in July, 1975. The web gaps at several floor beams were selected for the strain measurements. Some of the typical locations were (2):

1. At the end of the girder where the floor beam connected to a connection plate 6 in. (152 mm) away from the bearing stiffener.
2. At the end of the girder where the floor beam connected directly to the bearing stiffener.
3. At intermediate floor beams

At the floor beam-bearing stiffener connection, gages were placed at each end of the connection plate-bearing stiffener. All other strain gages were placed near the web gap between the end of the connection plate and the tension flange as illustrated in Fig. 6.

Behavior at End Supports

When the connection plate also served as a bearing stiffener, it was not welded to the flange at either end. No crack had formed at the lower gap, however, cracks had developed at the upper end of stiffener-connection plate. The strain gradient and stress range spectrum at the lower gap were relatively small, i.e. the adjusted root-mean-square stress range was $S_{rRMS} = 6.54$ ksi, while at the upper gap the adjusted stress range was $S_{rRMS} = 14.4$ ksi, as can be seen in Fig. 7. Figure 8 shows the time-stress response at the end support and at interior floor beam connections.

At the end reaction the stress range was always less than 5 ksi which indicated that cracking would not develop, and none was observed.
Since the connection plate at the end reaction was welded to the top flange, no movement could occur and no damage was detected.

. Behavior at Interior Floor Beams (Positive Moment Region)

The connection plates were tight fitted to the tension flange and welded to the top compression flange in the positive moment regions. A stiffener was also welded to the outside web surface and cut short of the flange by about 5/8 in. (16 mm), as shown in Fig. 5. No cracks were observed in the positive moment regions. The strain gage measurements indicated that stresses were below the fatigue limit, as can be seen in Fig. 8.

. Behavior of Floor Beams (Negative Moment Region)

In the negative moment regions where the connection plates were not welded to the top flange, cracks developed. The strain measurements indicated that the stress range at the weld toe was high even at the end of cracked webs, as can be seen in Fig. 8. At the bottom gap, the connection plates were welded to the flange, and no cracks were detected.

. Behavior at Interior Supports

The strain measurements at interior supports showed that web gap strains were small and comparable to those at interior floor beams at positive moment regions. The reason for this is that at the interior support, there is a concentrated reaction force which creates frictional resistance between the ends of the connection plate and the flanges.

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However, at least one location experienced a large crack at an interior support, as can be seen in Fig. 9. This crack was discovered in January 1978. No retrofit holes have been installed at the interior support.

Temperature and Environmental Effects (Known or Field Recorded)

These effects did not appear to play a significant role in the fatigue cracks that developed in the main girder webs of the Poplar Street complex. However, the thermal movement of the girders caused the girder to bend because of the seized expansion bearing. This resulted in a separation between the bottom flange and the top bearing plate and contributed to web buckling, as illustrated in Fig. 10.

Material "Fracture, Mechanical and Chemical Properties"

No information was acquired as it was not considered to contribute to the development of fatigue cracks in girder webs.

Visual and Fractographic Analysis

No crack samples were removed from the girder web.
3. CONCLUSIONS/RECOMMENDATIONS

Conclusions Regarding the Fracture/Failure

The cracks in the web gaps between the end of the floor beam connection plates and the girder flanges are all fatigue related. These fatigue cracks have resulted from cyclic out-of-plane web bending stresses due to out-of-plane displacement of the web in the gap region, as shown in Fig. 6. The severity of this displacement was greatest at the end bearing, at the top flange of intermediate floor beams in the negative moment regions of the girder, and at the top flange of interior supports. This was confirmed by field test measurements in typical gap regions under actual traffic conditions (2). The rotation of the floor beam which was rigidly connected to the main girder pushed the web out-of-plane, which caused high cyclic stresses along the toe of the web-to-flange welds and the stiffener connection plate ends.

The longitudinal orientation of the cracks in girder web relative to the stresses from loading are such that they do not affect the girder strength. However, corrective repairs were needed to prevent the cracks from growing perpendicular to the stresses in girder webs.

Recommendation for Repair/Fracture Control

Based on the observed behavior at the Poplar Street Complex, the following procedures for repair and corrective action were used (2):
At Girder Ends:

1. In general, welding the floor beam connection plates to the flanges would prevent the relative displacement that occurred between the end of the connection plate and the girder flange. Figure 11 shows retrofit procedures for the cracked and buckled ends.

2. At the ends of the existing web cracks along the weld toe of the web-to-flange connections (at either top or bottom flange) 5/8 in. (16 mm) holes were drilled through the web, as shown in Fig. 11a.

3. The web cracks at the ends of connection plates or stiffener-connection plates were also drilled to prevent cracks from propagating.

4. The cracks were gouged and welded up to the hole and then the surfaces were painted.

At Interior Floor Beams and Interior Supports:

1. Holes should be drilled at the end of the cracks similar to the case shown in Fig. 12, and the crack should be grouted with an epoxy and painted over to prevent rust formation. Two holes were drilled near the ends of the crack along the web-flange weld, as shown in Fig. 12. Holes were also drilled on each side of the stiffener to permit the crack to develop between them and accommodate out-of-plane displacements. This will not likely prevent rusting, because of the continued relative movements of the crack surfaces.
Crack Growth (Initiation and Propagation)

and Probable Causes of Cracking

The measurements at all floor beam locations indicated that the primary stresses occurring in the web gap regions were due to an out-of-plane movement of the girder web. This resulted from the rotation of the ends of the floor beams as they were loaded by traffic passing over the bridge structure. Since the floor beam connection plates were welded to the main girder web, and in some cases the compression flange, the floor beam end rotation was transmitted into the web by the connection plate. The connection plate stiffened most of the region of the web where it was attached. Consequently, the rotation of the ends of the floor beam forced the web gap to deform, as shown schematically in Fig. 6. This resulted in high stresses in the web gap regions where the upper and lower flanges of the main girder were restrained against twisting. Fatigue cracks developed at the end of the stiffener welds and along the toe of web-to-flange welds.

4. ACTUAL REPAIR/FRACTURE CONTROL EFFORTS

The recommendations were implemented by the Illinois-DOT in 1975-76. Holes were drilled in the girder webs at 751 locations, as shown schematically in Fig. 12. Ninety-seven of these locations were known to have cracks. In addition, at the girder ends, the stiffeners were welded to the tension flanges, as shown in Fig. 11. Figure 13 shows the retrofitted ends of two girders. Also apparent are the neoprene bearings that were installed in place of the rocker supports. The large crack shown in Fig. 9 was repaired by installing splice plates which can be seen in Fig. 14.
5. **POSSIBLE ADVERSE EFFECTS OF REPAIR**

All web cracks were parallel to the primary bending stresses in the girder web. After the corrective action and repairs were performed, no adverse effects are expected. Routine inspection should permit any crack initiation at the drilled holes to be detected. The retrofit of the web cracks was carried out in 1975-76. No cracks have reinitiated from the drilled holes up to 1981.

6. **ACTUAL OR ESTIMATED REPAIR/REHABILITATION COSTS**

- The original construction cost of the Poplar Street complex was $18,185,555.
- The repairs to the girder ends, the drilling of holes at intermediate floor beams in the negative moment regions and the replacement of bearings cost $3,387,730.80.
- The large crack found at the pier of Ramp C was retrofitted by district maintenance personnel at an estimated cost of $38,522.16.
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Fig. 1 Layout of the Poplar Street Approaches
Fig. 2 Typical Plan and Elevation of Section of Poplar Street Structure
Fig. 3 Typical Cross-Section of Poplar Street Approaches Showing Floor Beam-Girder Connection
Fig. 4 Crack in Web Near End Support
(a) Schematic of Negative Moment Detail

(b) Photograph of Crack on Outside Web Surface

Fig. 5 Crack in Web of Negative Moment Region
Fig. 6 Schematic of Distortion in Web Gap and Location of Strain Gages
Fig. 7 Stress Range Histogram of Web Gap

\[ S_{RMS} = 14.4 \text{ ksi} \] (N = 324)
Fig. 8 Typical Stress-Time Plots for Web Gaps
Fig. 9 Largest Crack at Interior Support

Fig. 10 Web Buckling at End Support
(a) Retrofit for Cracked Girder Ends

(b) Retrofit for Repair of Buckled Webs

Fig. 11 Retrofit Procedures for Girder Ends
Fig. 12 Retrofit Holes at Fatigue Crack Tips
Fig. 13 Retrofitted Girder End Showing Stiffeners and Elastomeric Bearings

Fig. 14 Splice Plates Installed at Lare Web Crack
10.2 FATIGUE/FRACTURE ANALYSIS
OF
DES MOINES (POLK COUNTY) BRIDGE

-- A Brief Summary --

I.D. No. : 58
Town/City/etc. : Four Mile Twp. and Des Moines, Iowa
Railway/Highway/Avenue: Iowa Route 163
County : Polk County, Iowa
State/Province : Iowa
Owner : Iowa-DOT
1. DESCRIPTION AND HISTORY OF THE BRIDGE

Brief Description of Structure:

The Des Moines (Polk County) Bridge carries east and westbound traffic over the Chicago Rock Island and Pacific Railroad (C.R.I. and P.R.R.) tracks and the East Four Mile Creek on Route 163. The bridge is located near Des Moines, Iowa. The east and westbound bridges are two similar, but separate structures.

Each skew bridge structure is composed of two continuous welded girders with transverse stiffeners 590 ft. long supporting a 30 ft. roadway. The east end span is 91 ft. - 0 in. long, and the west end span is 118 ft. - 0 in. long. The two interior spans are each 127 ft. - 0 in. long. The slab is 7-1/4 in. thick reinforced concrete supported by two 18 WF 45 stringers and the two main girders (see Fig. 1). The floor slab includes one-half inch wearing surface. The plan and elevation of the bridge are given in Figs. 1 and 2. Typical intermediate and pier cross sections are shown in Fig. 3.

The two main girder webs are 74 in. deep. The floor beams supporting the stringers have 48 in. x 5/16 in. web plates with alternating transverse stiffeners.

The Des Moines Bridge was designed for H20-SK loading. There was 19 lb./sq. ft. of roadway added for future wearing surface. The design is based on the 1961 AASHO Specifications and 1960 Iowa-DOT Standard Specifications and current Special Provisions and Supplemental Specifications. All members were fabricated from A36 steel.
Brief History of the Structure and Cracking:

The Des Moines Bridge was constructed in 1962 and opened to traffic in 1963 (Ref. 1). The volume of truck traffic crossing the bridge in 1979 was 500 vehicles per day (Ref. 1).

In 1979, several fatigue cracks were discovered in the webs of main girders of the Des Moines Bridge (Ref. 2). These cracks were in the negative moment regions along the upper web-to-flange fillet welds above the floor beam connecting plate. Several vertical cracks were observed in the fillet welds joining the floor beam connecting plates to the girder web (Ref. 2) and in the web several inches below the web flange connection.
2. FAILURE MODES AND FAILURE ANALYSIS

Location of Fracture, Failure Modes and Stages:

Several web cracks were discovered by the Iowa-DOT personnel in 1979 (Ref. 2) in the negative moment regions of the main girders of the Des Moines Bridge. All cracks were detected in the floor beam connection plates nearest to the skewed piers in the negative moment regions. Two types of web cracks were observed: (1) Horizontal fatigue cracks in the main girder webs along the toe of the web-to-flange fillet welds. These cracks were 3 to 6 in. long and are shown schematically in Fig. 5. Figure 6 shows photographs of the inside and outside web surfaces indicating cracking has developed on each side of the web plate. (2) Vertical weld cracks were also observed at the top ends of the floor beam connecting plate, as shown in Figs. 5 and 6. At several locations these cracks stopped after propagating downward for an inch or so and intersected the second web crack shown in Fig. 6.

On-site examination of the web cracks in the main girders of the Des Moines Bridge indicated that all were fatigue cracks (Ref. 1). The principal cause of the cracking is the out-of-plane deformation of the girder webs in the small gaps at the ends of the floor beam connection plates. The fatigue crack development in the girder webs in the negative moment regions is a result of the web being forced out-of-plane by the floor beam end rotation in the small gaps between the end of the floor beam connection plates and web-to-flange welds (Refs. 1, 2). Since the slab constrains the girder top flange at these locations, and the connection plate has its strong axis perpendicular to the web end rotation of
the floor beam forces the web out-of-plane. When the gap is very small, the resulting cyclic stresses are very high along the upper end of the connection plate weld. This causes the welds to crack and separates the connection plate end from the web. As the vertical cracks in the connection plate welds increase in length, the web deforms, and this results in high out-of-plane bending stresses. This also causes web cracks to form several inches below the web-to-flange weld from the double curvature bending in the gap. In skewed bridges the out-of-plane movement is more severe because of the different vertical displacements at the two ends of the floor beams (Ref. 1).

Cyclic Loads and Stresses (Known or Field-Recorded):

The ADTT is known to be 500 vehicles in 1979 (2). No stress measurements are available.

Temperature and Environmental Effects (Known or Field-Recorded):

Temperature and other environmental effects do not appear to have contributed to the development of cracks. The cracks were developed from stresses as a result of truck traffic. They occur from out-of-plane displacements in the girder web gaps at floor beam connection plates in negative moment regions.
. **Material "Fracture, Mechanical, Chemical Properties"**

**Analysis in Failure Regions:**

No material tests were carried out other than those reported on the mill reports. No Charpy V-Notch tests are available.

. **Visual and Fractographic Examination:**

Only visual observations were made on the cracks detected in the welds and web.
3. CONCLUSIONS AND RECOMMENDATIONS

Conclusions Regarding Fracture/Failure:

The Type 1 and 2 cracks discovered in the main girders of the Des Moines Bridges were all fatigue cracks. Each of these skew bridges consists of two main girders with floor beams and stringers. The fatigue cracks developed where the floor beams next to the piers are connected to the main girders. The floor beams next to the piers are bolted to intermediate stiffeners or connecting plates nine inches away from bearing stiffeners. The cracks are caused by the out-of-plane displacements of the girder web in the negative moment regions at floor beam connecting plates welded to the web (see also Fig. 7). The horizontal cracks at the toes of the flange-web fillet weld developed where the web out-of-plane displacements were relatively large because of the skewed bridge configuration.

Vertical cracks along the connection plate girder web welds developed from the upper end of these welds. In several cases, further horizontal cracks developed in the web. These resulted from double curvature bending resulting from the out-of-plane displacement of the girder web due to rotations of the floor beam connection plates under cyclic traffic loads (1,3).

Both types of cracking are shown schematically in Fig. 7.
Recommendations for Repair/Fracture Control:

There are several ways to retrofit the fatigue cracks that developed in the main girder webs of the Des Moines Bridge. These are as follows (1):

(a) One possibility is to weld the floor beam connection plates to the top flange of the girder in the negative moment regions as shown in Fig. 8. This provides a fatigue Category C condition (3). Welding the connection plates to the top flange would minimize the relative out-of-plane movement of the girder web in the negative moment region. Some difficulties may develop in obtaining good welds under field conditions. In addition, holes should be drilled in the web at the ends of all existing cracks to prevent further crack propagation.

(b) An alternate solution is to remove part of the connecting plate and opposite web stiffener, so that a significant gap exists between the end of the transverse plate and the restrained tension flange. This solution is shown schematically in Fig. 9.

Additionally, all web cracks should have holes drilled at their ends in order to arrest further fatigue crack growth.
4. **ACTUAL REPAIR/FRACTURE CONTROL EFFORTS**

The information and data for the retrofit of the Des Moines (Polk County) Bridge were provided by Iowa-DOT. After visual and dye penetration inspection of the fatigue cracks, the following retrofit scheme was used (4):

(a) In the negative moment regions, the top portion of the stiffener (which is also the connecting plate for the floor beam or diaphragms) was removed by flame-cutting, and the surfaces were ground smooth as shown in Figs. 9 and 10.

(b) There were 3/4 in. $\phi$ holes drilled at the ends of the cracks in order to arrest further growth. This is shown in Fig. 11.

(c) In the negative moment regions at the piers, the pier stiffeners were bolted to the girder top flange using a clip angle on each side of the stiffener. In order to facilitate this repair, a rectangular section of the reinforced concrete deck above the pier stiffener was cut out in order to drill holes in the top flange and install the bolts. After installation of the bolts, the holes were filled with concrete, as shown in Fig. 12.

5. **POSSIBLE ADVERSE EFFECTS OF REPAIR**

No adverse behavior is expected from the retrofit.
6. **ACTUAL OR ESTIMATED REPAIR/REHABILITATION COSTS**

. **Cost of Investigation:**

This was approximately $3000.

. **Cost of Repair/Rehabilitation:**

Cost of retrofitting the Des Moines (Polk County) Bridge in 1980 was $35,900.

. **Original Cost of the Bridge:**

The construction contract for the bridge was $438,952 in 1964.

7. **REFERENCES**

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Fig. 1 Plan of the Des Moines (Polk County) Bridge
Fig. 2 Elevation and Side View of the Des Moines (Polk County) Bridge
Fig. 3 Typical Cross-Sections of Des Moines (Polk County) Bridge
Fig. 4 Crack Locations at Floor Beam Connecting Plates in Des Moines (Polk County) Bridge
(Crack Locations are marked with a circle and number)
Fig. 5 Typical Cracks at Web-to-Flange Welds, at Top Ends of Stiffeners and Along the Toe of Stiffener-to-Web Welds (at Location 1).
Fig. 6 Inside and Outside Photographs of Typical Cracks at the floor beam connection plate (or stiffener) Location
Fig. 7 Schematic of Cracking in Connection Plate-to-Web and at Toe of Flange-to-Web Weld
Fig. 8 Recommended Retrofit Scheme for Treatment of Floor Beam Connection Plates - Alternative 1

Fig. 9 Recommended Retrofit Scheme for Treatment of Floor Beam Connection Plate - Alternative 2
Fig. 10 Photographs of Actual Retrofit Detail
(at connecting plate in negative moment region)
Fig. 11 Drilling Holes at the Ends of Crack at Web-to-Flange Weld
Fig. 12 Photographs of Actual Retrofit Detail (Stiffeners at Piers)