INVESTIGATION OF DYNAMIC STRESSES IN HIGHWAY BRIDGES—AN INTERIM REPORT

RECEIVED
MAR 5 1974

By
W. H. WALKER
J. A. RUHL

Issued as an Interim Report on
The Investigation of Dynamic Stresses
in Highway Bridges
Project IHR-85
Illinois Cooperative Highway Research Program

Conducted by
THE STRUCTURAL RESEARCH LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
ENGINEERING EXPERIMENT STATION
UNIVERSITY OF ILLINOIS AT URBANA-CHAMPAIGN

in cooperation with the
STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

and the
U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

UNIVERSITY OF ILLINOIS
URBANA, ILLINOIS
OCTOBER 1973
An investigation of the dynamic response of three continuous highway bridges subjected to both a standard test vehicle and traffic loadings has been conducted. Comprehensive studies of bridge response under traffic loadings were made using data from over 2,700 heavy truck traffic crossings; in addition, deterministic studies of bridge behavior were made using data from some 130 test runs with the FHWA vehicle. The experimental data consists of the vehicle characteristics and sets of strain and deflection records for each truck describing the time-history of response at selected locations on the bridge.

Systems of computer software have been developed taking advantage of recent advances in data recording and processing hardware to allow highly automated recordings, reduction and analysis of the data. A significant characteristic of the study is that the response-histories for each vehicle crossing are retained as permanent records in computer based disk or tape files.

Results are presented and discussed for a limited series of tests; a comprehensive study of the results will be included in the final report of the investigation.
ACKNOWLEDGMENT

This report was prepared as a part of the Illinois Cooperative Highway Research Program, Project IHR-85, "Dynamic Stresses in Highway Bridges," by the Department of Civil Engineering, in the Engineering Experiment Station, University of Illinois at Urbana-Champaign, in cooperation with the Illinois Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.
DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Illinois Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification or regulation.
# TABLE OF CONTENTS

1. INTRODUCTION
   1.1 General .................................................. 1
   1.2 Specific Objectives ...................................... 2
   1.3 Applications to Practice .................................. 3
   1.4 Vibration Theory and Significant Bridge-Vehicle
        Parameters ............................................... 6
   1.5 A General Note on Bridge Selection and Instrumentation. 9
   1.6 Object and Scope of Interim Report ...................... 11

2. TEST PROGRAM
   2.1 Bridges .................................................. 13
   2.2 Vehicle ................................................... 16
   2.3 State Truck Weight Station .............................. 18
   2.4 Bridge Instrumentation .................................. 20
      2.4.1 Strain and Deflection Gages ......................... 20
      2.4.2 Standard Traffic Counter ........................... 22
      2.4.3 Data Recording ...................................... 22
      2.4.4 Control Logic Requirements......................... 23

3. DATA REDUCTION AND ANALYSIS ............................. 27
   3.1 General .................................................. 27
   3.2 Bridge Data Reduction ................................... 30
   3.3 Vehicle Data ............................................... 40
   3.4 Computer-Oriented Data Analysis Interpretation .......... 43
      3.4.1 General ............................................... 43
      3.4.2 Basic Data Processing Capabilities ................. 44

4. PRESENTATION AND DISCUSSION OF PRELIMINARY RESULTS ... 50
   4.1 Scope .................................................... 50
   4.2 Static and Dynamic Bridge Response -- A Review of
        Time-History Data ........................................ 51
      4.2.1 General ............................................... 51
      4.2.2 Static or Crawl Response ............................ 53
      4.2.3 Dynamic Response .................................... 62
   4.3 Characteristics of Heavy Truck Traffic .................. 75
   4.4 Stress History Data for Selected Truck Traffic .......... 80
5. ON DEVELOPMENT OF STRESS HISTORY THEORY -
   A PROBABILISTIC APPROACH .................................................. 92
   5.1 General .............................................................................. 92
   5.2 Dynamic Bridge Response - Deterministic Analysis and Simulation .............................................. 96
   5.3 Probabilistic Stress History - Interpretation and Prediction ............................................................. 100
      5.3.1 General ........................................................................ 100
      5.3.2 An Empirical Approach to the Influence of Vehicle Characteristics on Bridge Response ...... 102
      5.3.3 A Simplified Mathematical Model ......................................................................................... 105

6. SUMMARY ..................................................................................... 110

REFERENCES .................................................................................. 113

TABLES .......................................................................................... 115

FIGURES ......................................................................................... 127

APPENDIX

   A. NATURAL FREQUENCIES AND MODES OF THE SHAFFER CREEK BRIDGE ............................................. 226
# List of Tables

<table>
<thead>
<tr>
<th>Table No.</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>SUMMARY OF DATA ON BRIDGES TESTED</td>
<td>115</td>
</tr>
<tr>
<td>2.2</td>
<td>TEST VEHICLE TIRE CONTACT AREA</td>
<td>116</td>
</tr>
<tr>
<td>2.3</td>
<td>DEFLECTION AND STRAIN GAGE LOCATIONS AND RECORDING</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EQUIPMENT SALT FORK RIVER BRIDGE - 1967</td>
<td></td>
</tr>
<tr>
<td>2.4</td>
<td>DEFLECTION AND STRAIN GAGE LOCATIONS AND RECORDING</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EQUIPMENT SHAFFER CREEK BRIDGE - 4-BEAM PORTION - 1968.</td>
<td>120</td>
</tr>
<tr>
<td>2.5</td>
<td>DEFLECTION AND STRAIN GAGE LOCATIONS AND RECORDING</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EQUIPMENT SHAFFER CREEK BRIDGE - 5-BEAM PORTION - 1968.</td>
<td>122</td>
</tr>
<tr>
<td>2.6</td>
<td>DEFLECTION AND STRAIN GAGE LOCATIONS</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SHAFFER CREEK BRIDGE - 1969</td>
<td>123</td>
</tr>
<tr>
<td>2.7</td>
<td>STRAIN GAGE LOCATIONS CB&amp;Q BRIDGE - 1969</td>
<td>124</td>
</tr>
<tr>
<td>4.1</td>
<td>MAXIMUM STRAINS AND DEFLECTIONS FOR THE TEST VEHICLE</td>
<td></td>
</tr>
<tr>
<td></td>
<td>CRAWL RUNS AT SALT FORK RIVER BRIDGE</td>
<td>125</td>
</tr>
<tr>
<td>4.2</td>
<td>MAXIMUM STRAINS AND DEFLECTIONS AT MIDSPAN FOR THE TEST VEHICLE</td>
<td></td>
</tr>
<tr>
<td></td>
<td>CRAWL RUNS AT SHAFFER CREEK BRIDGE</td>
<td>126</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>--------</td>
<td>-----------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>2.1</td>
<td>Salt Fork River Bridge</td>
<td>127</td>
</tr>
<tr>
<td>2.2</td>
<td>Instrumentation For Salt Fork River Bridge</td>
<td>128</td>
</tr>
<tr>
<td>2.3</td>
<td>Shaffer Creek Bridge</td>
<td>129</td>
</tr>
<tr>
<td>2.4</td>
<td>Instrumentation For Shaffer Creek Bridge</td>
<td>130</td>
</tr>
<tr>
<td>2.5</td>
<td>CB&amp;Q Bridge</td>
<td>131</td>
</tr>
<tr>
<td>2.6</td>
<td>Photograph of Salt Fork River Bridge</td>
<td>132</td>
</tr>
<tr>
<td>2.7</td>
<td>Photograph of Shaffer Creek Bridge</td>
<td>133</td>
</tr>
<tr>
<td>2.8</td>
<td>Photograph of CB&amp;Q Bridge</td>
<td>134</td>
</tr>
<tr>
<td>2.9</td>
<td>Photograph of FHWA Test Vehicle</td>
<td>135</td>
</tr>
<tr>
<td>2.10</td>
<td>Test Vehicle Configuration and Loads</td>
<td>136</td>
</tr>
<tr>
<td>3.1</td>
<td>Flow Diagram For Data Acquisition and Reduction</td>
<td>137</td>
</tr>
<tr>
<td>3.2</td>
<td>Sequence of Events as Recorded on Analog in The Field For The Passage of One Truck</td>
<td>138</td>
</tr>
<tr>
<td>4.1</td>
<td>Crawl Curves for Deflection, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck, Lane 2, Eastbound</td>
<td>139</td>
</tr>
<tr>
<td>4.2</td>
<td>Crawl Curves for Bottom Flange Strain, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck, Lane 2, Eastbound</td>
<td>140</td>
</tr>
<tr>
<td>4.3</td>
<td>Crawl Curves For Top Flange Strain, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck, Lane 2, Eastbound</td>
<td>141</td>
</tr>
<tr>
<td>4.4</td>
<td>Crawl Curves for Bottom Flange Strain, All Beams, Section A, Salt Fork River Bridge -- FHWA Test Truck, Lane 3, Eastbound</td>
<td>142</td>
</tr>
<tr>
<td>4.5</td>
<td>Crawl Curves for Bottom Flange Strain, Beams 3, 4, and 5, Section D, Salt Fork River Bridge -- FHWA Test Truck, Lane 2, Eastbound</td>
<td>143</td>
</tr>
<tr>
<td>4.6</td>
<td>Crawl Curves For Bottom Flange Strain, All Beams, Section A, Shaffer Creek Bridge, 1968 -- FHWA Test Truck, Lane 1</td>
<td>144</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>--------</td>
<td>--------------------------------------------------------------------------------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>4.7</td>
<td>Crawl Curves For Bottom Flange Strain, All Beams, Section A, Shaffer Creek Bridge, 1968 -- FHWA Test Truck, Lane 2</td>
<td>145</td>
</tr>
<tr>
<td>4.8</td>
<td>Crawl Curves For Deflection, All Beams, Section A, Shaffer Creek Bridge, 1968 -- FHWA Test Truck, Lane 2</td>
<td>146</td>
</tr>
<tr>
<td>4.9</td>
<td>Crawl Curves for Bottom Flange Strain, All Beams, Section A, Shaffer Creek Bridge, 1968 -- FHWA Test Truck, Lane 3</td>
<td>147</td>
</tr>
<tr>
<td>4.10</td>
<td>Transverse Distributions of Maximum Crawl Strains and Deflections, Salt Fork River Bridge, FHWA Test Truck</td>
<td>148</td>
</tr>
<tr>
<td>4.11</td>
<td>Transverse Distributions of Maximum Crawl Strains and Deflections, Section A, Shaffer Creek Bridge, 1968, FHWA Test Truck</td>
<td>149</td>
</tr>
<tr>
<td>4.12</td>
<td>Crawl, Total Dynamic and Dynamic Increment Histories For Deflection, Salt Fork River Bridge, Center of Center Span, Beam 4, 59.7 mph, Lane 2, Eastbound, Alpha=.15</td>
<td>150</td>
</tr>
<tr>
<td>4.13</td>
<td>Crawl, Total Dynamic and Dynamic Increment Histories, Salt Fork River Bridge, For Strain, Center of Center Span, Beam 4, 37.5 mph, Lane 3, Eastbound, Alpha = .09</td>
<td>151</td>
</tr>
<tr>
<td>4.14</td>
<td>Time-Histories for Deflection, Beam 4, Section A, Shaffer Creek Bridge, -- FHWA Test Truck, Lane 3</td>
<td>152</td>
</tr>
<tr>
<td>4.15</td>
<td>Deflection Histories, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck, 21.2 mph (α = 0.05), Lane 2, Eastbound</td>
<td>153</td>
</tr>
<tr>
<td>4.16</td>
<td>Bottom Flange Strain Histories, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck, 21.2 mph (α = 0.05), Lane 2, Eastbound</td>
<td>154</td>
</tr>
<tr>
<td>4.17</td>
<td>Deflection Histories, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck, 39.0 mph (α = 0.10), Lane 2, Eastbound</td>
<td>155</td>
</tr>
<tr>
<td>4.18</td>
<td>Bottom Flange Strain Histories, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Vehicle, 39.0 mph (α = 0.10), Lane 2, Eastbound</td>
<td>156</td>
</tr>
<tr>
<td>4.19</td>
<td>Deflection Histories, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck 59.7 mph (α = 0.15), Lane 2, Eastbound</td>
<td>157</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>--------</td>
<td>------------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>4.20</td>
<td>Bottom Flange Strain Histories, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck, 59.7 mph ($\alpha = 0.15$), Lane 2, Eastbound</td>
<td>158</td>
</tr>
<tr>
<td>4.21</td>
<td>Deflection Histories, Beam 3, Section A, Salt Fork River Bridge -- FHWA Test Truck, Lane 2, Eastbound</td>
<td>159</td>
</tr>
<tr>
<td>4.22</td>
<td>Deflection Histories, All Beams, Section A, Shaffer Creek Bridge, 1968 -- FHWA Test Truck, 32.1 mph ($\alpha = 0.07$), Lane 2</td>
<td>160</td>
</tr>
<tr>
<td>4.23</td>
<td>Bottom Flange Strain Histories, All Beams, Section A, Shaffer Creek Bridge, 1968 -- FHWA Test Truck, 32.1 mph ($\alpha = 0.07$), Lane 2</td>
<td>161</td>
</tr>
<tr>
<td>4.24</td>
<td>Bottom Flange Strain Histories, All Beams, Section A, Shaffer Creek Bridge, 1968 -- FHWA Test Truck, 54.0 mph ($\alpha = 0.12$), Lane 2</td>
<td>162</td>
</tr>
<tr>
<td>4.25</td>
<td>Bottom Flange Strain Histories, All Beams, Section A, Shaffer Creek Bridge, 1968 -- Typical 3S-2 Tractor, Semi-Trailer, 72.5 kips, 45.7 mph ($\alpha = 0.10$), Lane 2</td>
<td>163</td>
</tr>
<tr>
<td>4.26</td>
<td>Deflection Histories, All Beams, Section A, Shaffer Creek Bridge, 1968 -- Typical 3S-2 Tractor, Semi-Trailer, 72.5 kips, 45.7 mph ($\alpha = 0.10$), Lane 2</td>
<td>164</td>
</tr>
<tr>
<td>4.27</td>
<td>Deflection Histories, All Beams, Section A, Shaffer Creek Bridge, 1968 -- Typical 3S-2 Dump Truck, 70.1 kips, 45.7 mph ($\alpha = 0.10$), Lane 2</td>
<td>165</td>
</tr>
<tr>
<td>4.28</td>
<td>Bottom Flange Strain Histories, All Beams, Section A, Shaffer Creek Bridge, 1968 -- Typical 3S-2 Dump Truck, 70.1 kips, 45.7 mph ($\alpha = 0.10$), Lane 2</td>
<td>166</td>
</tr>
<tr>
<td>4.29</td>
<td>Top and Bottom Flange Strain Histories and Neutral Axis History, Beam 3, Section A, Shaffer Creek Bridge, 1969 -- Typical 3S-2 Tractor, Semi-Trailer, 79.1 kips, 43.5 mph ($\alpha = 0.09$), Lane 2</td>
<td>167</td>
</tr>
<tr>
<td>4.30</td>
<td>Top and Bottom Flange Strain Histories and Neutral Axis History, Beam 4, Section A, Shaffer Creek Bridge, 1969 -- Typical 3S-2 Tractor, Semi-Trailer, 79.1 kips, 43.5 mph ($\alpha = 0.09$), Lane 2</td>
<td>168</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
<td>------</td>
</tr>
<tr>
<td>4.31</td>
<td>Typical Strain History, Transverse Slab Reinforcing Bar Between Beams 2 and 3, Section E, Salt Fork River Bridge, FHWA Test Truck, 40 mph, Lane 2</td>
<td>169</td>
</tr>
<tr>
<td>4.32</td>
<td>Effect of Vehicle Speed on Transverse Slab Reinforcing Bar Strain, Between Beams 2 and 3, Section E, Salt Fork River Bridge, FHWA Test Truck</td>
<td>170</td>
</tr>
<tr>
<td>4.33</td>
<td>Coding of Truck Types</td>
<td>171</td>
</tr>
<tr>
<td>4.34</td>
<td>Histogram For Truck Axle Types, All Trucks, Shaffer Creek Bridge, 1969</td>
<td>172</td>
</tr>
<tr>
<td>4.35</td>
<td>Histogram for Truck Axle Type, All Trucks, CB&amp;Q Bridge, 1969</td>
<td>173</td>
</tr>
<tr>
<td>4.36</td>
<td>Histogram for Gross Vehicle Weight, All Trucks, Shaffer Creek Bridge, 1968</td>
<td>174</td>
</tr>
<tr>
<td>4.37</td>
<td>Histogram For Gross Vehicle Weight, All Trucks, Shaffer Creek Bridge, 1969</td>
<td>175</td>
</tr>
<tr>
<td>4.38</td>
<td>Histogram For Gross Vehicle Weight, All Trucks, CB&amp;Q Bridge, 1969</td>
<td>176</td>
</tr>
<tr>
<td>4.39</td>
<td>Histogram For Gross Vehicle Weight, 3S-2 Trucks, Shaffer Creek Bridge, 1969</td>
<td>177</td>
</tr>
<tr>
<td>4.40</td>
<td>Histogram For Gross Vehicle Weight, 3S-2 Trucks, CB&amp;Q Bridge, 1969</td>
<td>178</td>
</tr>
<tr>
<td>4.41</td>
<td>Histogram For Axle Weight, All Trucks, All Axles, Shaffer Creek Bridge, 1969</td>
<td>179</td>
</tr>
<tr>
<td>4.42</td>
<td>Histogram For Axle Weight, All Trucks, All Axles, CB&amp;Q Bridge, 1969</td>
<td>180</td>
</tr>
<tr>
<td>4.43</td>
<td>Histogram of Axle Weight, 3S-2 Trucks, All Axles, Shaffer Creek Bridge, 1969</td>
<td>181</td>
</tr>
<tr>
<td>4.44</td>
<td>Histogram For Axle Weight, 3S-2 Trucks, All Axles, CB&amp;Q Bridge, 1969</td>
<td>182</td>
</tr>
<tr>
<td>4.45</td>
<td>Histogram For Axle Weight, All Trucks, Axle A, Shaffer Creek Bridge, 1968</td>
<td>183</td>
</tr>
<tr>
<td>4.46</td>
<td>Histogram For Axle Weight, All Trucks, Axle A, Shaffer Creek Bridge, 1969</td>
<td>184</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>--------</td>
<td>------------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>4.47</td>
<td>Histogram For Axle Weight, All Trucks, Axle A, CB&amp;Q Bridge, 1969</td>
<td>185</td>
</tr>
<tr>
<td>4.48</td>
<td>Histogram For Axle Weight, All Trucks, Axle B, Shaffer Creek Bridge, 1968</td>
<td>186</td>
</tr>
<tr>
<td>4.49</td>
<td>Histogram For Axle Weight, All Trucks, Axle B, Shaffer Creek Bridge, 1969</td>
<td>187</td>
</tr>
<tr>
<td>4.50</td>
<td>Histogram For Axle Weight, All Trucks, Axle B, CB&amp;Q Bridge, 1969</td>
<td>188</td>
</tr>
<tr>
<td>4.51</td>
<td>Histogram For Axle Weight, All Trucks, Axle C, Shaffer Creek Bridge, 1968</td>
<td>189</td>
</tr>
<tr>
<td>4.52</td>
<td>Histogram For Axle Weight, All Trucks, Axle C, Shaffer Creek Bridge, 1969</td>
<td>190</td>
</tr>
<tr>
<td>4.53</td>
<td>Histogram For Axle Weight, All Trucks, Axle C, CB&amp;Q Bridge, 1969</td>
<td>191</td>
</tr>
<tr>
<td>4.54</td>
<td>Histogram For Axle Weight, All Truck, Axle D, Shaffer Creek Bridge, 1968</td>
<td>192</td>
</tr>
<tr>
<td>4.55</td>
<td>Histogram For Axle Weight, All Trucks, Axle D, or E, Shaffer Creek Bridge, 1969</td>
<td>193</td>
</tr>
<tr>
<td>4.56</td>
<td>Histogram For Axle Weight, All Trucks, Axle D, CB&amp;Q Bridge, 1969</td>
<td>194</td>
</tr>
<tr>
<td>4.57</td>
<td>Histogram For Axle Weight, All Trucks, Axle E, Shaffer Creek Bridge, 1968</td>
<td>195</td>
</tr>
<tr>
<td>4.58</td>
<td>Histogram For Axle Weight, All Trucks, Axle E, Shaffer Creek Bridge, 1969</td>
<td>196</td>
</tr>
<tr>
<td>4.59</td>
<td>Histogram For Axle Weight, All Trucks, Axle E, CB&amp;Q Bridge, 1969</td>
<td>197</td>
</tr>
<tr>
<td>4.60</td>
<td>Histogram For Axle Weight, 3S-2 Trucks, Axle A, CB&amp;Q Bridge, 1969</td>
<td>198</td>
</tr>
<tr>
<td>4.61</td>
<td>Histogram For Axle Weight, 3S-2 Trucks, Axle B, CB&amp;Q Bridge, 1969</td>
<td>199</td>
</tr>
<tr>
<td>4.62</td>
<td>Histogram For Axle Weight, 3S-2 Trucks, Axle C, CB&amp;Q Bridge, 1969</td>
<td>200</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>-------</td>
<td>-----------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>4.63</td>
<td>Histogram For Axle Weight, 3S-2 Trucks, Axles D and E, CB&amp;Q Bridge, 1969</td>
<td>201</td>
</tr>
<tr>
<td>4.64</td>
<td>Histogram For Axle Spacing, All Trucks, Spacing A-B, CB&amp;Q Bridge, 1969</td>
<td>202</td>
</tr>
<tr>
<td>4.65</td>
<td>Histogram For Axle Springs, All Trucks, Spacing B-C, CB&amp;Q Bridge, 1969</td>
<td>203</td>
</tr>
<tr>
<td>4.66</td>
<td>Histogram For Axle Spacing, All Trucks, Spacing C-D, CB&amp;Q Bridge, 1969</td>
<td>204</td>
</tr>
<tr>
<td>4.67</td>
<td>Histogram For Axle Spacing, All Trucks, Spacing D-E, CB&amp;Q Bridge, 1969</td>
<td>205</td>
</tr>
<tr>
<td>4.68</td>
<td>Histogram For Time Interval, All Trucks, Shaffer Creek Bridge, 1969</td>
<td>206</td>
</tr>
<tr>
<td>4.69</td>
<td>Histogram For Time Interval, All Trucks, CB&amp;Q Bridge, 1969</td>
<td>207</td>
</tr>
<tr>
<td>4.70</td>
<td>Maximum Strain Range in Beam 4 Plotted versus Gross Vehicle Weight For Selected Traffic and Test Truck Runs at Shaffer Creek Bridge in 1968</td>
<td>208</td>
</tr>
<tr>
<td>4.71</td>
<td>Maximum Strain Range in Beam 5 Plotted versus Gross Vehicle Weight For Selected Traffic and Test Truck Runs at Shaffer Creek Bridge in 1968</td>
<td>209</td>
</tr>
<tr>
<td>4.72</td>
<td>Histogram For Truck Type, 148 Selected Runs, Shaffer Creek Bridge, 1968</td>
<td>210</td>
</tr>
<tr>
<td>4.73</td>
<td>Histogram For Gross Vehicle Weight, 148 Selected Runs, Shaffer Creek Bridge, 1968</td>
<td>211</td>
</tr>
<tr>
<td>4.74</td>
<td>Histogram of Transverse Position, Selected Runs, Shaffer Creek Bridge, 1968</td>
<td>212</td>
</tr>
<tr>
<td>4.75</td>
<td>Length versus Strain Range in Beams, 3S-2 Dump Trucks, Shaffer Creek Bridge, 1968</td>
<td>213</td>
</tr>
<tr>
<td>4.76</td>
<td>Histogram For the Percent of The Total Strain at Section A in Beam 5, Shaffer Creek Bridge, 1968 -- Heavy Truck Traffic</td>
<td>214</td>
</tr>
<tr>
<td>4.77</td>
<td>Variation of Strain Range vs. GVW, Beam 4, Section A Shaffer Creek Bridge, 1968 -- Heavy Truck Traffic, S2 = 5 to 40 ft., Speed = 40 to 65 mph</td>
<td>215</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>---------</td>
<td>-----------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>4.78</td>
<td>Variation of Strain Range vs. GVW, Beam 5, Section A, Shaffer Creek Bridge, 1968 -- Heavy Truck Traffic, S2 = 5 to 40 ft., Speed = 40 to 65 mph</td>
<td>216</td>
</tr>
<tr>
<td>4.79</td>
<td>Histogram For Bottom Flange Strain Range, Beam 1, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic</td>
<td>217</td>
</tr>
<tr>
<td>4.80</td>
<td>Histogram For Bottom Flange Strain Range, Beam 2, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic</td>
<td>218</td>
</tr>
<tr>
<td>4.81</td>
<td>Histogram For Bottom Flange Strain Range, Beam 3, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic</td>
<td>219</td>
</tr>
<tr>
<td>4.82</td>
<td>Histogram For Bottom Flange Strain Range, Beam 4, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic</td>
<td>220</td>
</tr>
<tr>
<td>4.83</td>
<td>Histogram For Bottom Flange Strain Range, Beam 5, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic</td>
<td>221</td>
</tr>
<tr>
<td>4.84</td>
<td>Histogram For Bottom Flange Partial Strain Range, Beam 2, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic</td>
<td>222</td>
</tr>
<tr>
<td>4.85</td>
<td>Histogram For Bottom Flange Partial Strain Range, Beam 3, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic</td>
<td>223</td>
</tr>
<tr>
<td>4.86</td>
<td>Histogram For Bottom Flange Partial Strain Range, Beam 4, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic</td>
<td>224</td>
</tr>
<tr>
<td>4.87</td>
<td>Histogram For Bottom Flange Partial Strain Range, Beam 5, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic</td>
<td>225</td>
</tr>
<tr>
<td>A.1</td>
<td>Shaffer Creek Five-Beam Bridge Model</td>
<td>228</td>
</tr>
<tr>
<td>A.2</td>
<td>First and Second Mode Shapes For Shaffer Creek Bridge</td>
<td>229</td>
</tr>
<tr>
<td>A.3</td>
<td>Third and Fourth Mode Shapes For Shaffer Creek Bridge</td>
<td>230</td>
</tr>
<tr>
<td>A.4</td>
<td>Natural Frequencies For the 30 Degree-of-Freedom Shaffer Bridge Model</td>
<td>231</td>
</tr>
</tbody>
</table>
1. INTRODUCTION

1.1 General

Research on fatigue life expectancy of highway bridges subjected to heavy truck traffic may be subdivided into three major areas;

(1) The collection, analysis, and interpretation of data on strains and stresses occurring at critical locations in the bridge under the action of heavy truck traffic. This is the so-called load-history or stress-history problem and is the subject of the present investigation. In other words, it represents the detailed study of the response of the structure to the load (traffic) environment.

(2) Investigation of material behavior and the development of cumulative damage laws describing the fatigue-induced damage and failure due to repeated, variable loadings or stresses which are representative of the stress histories occurring in highway bridges. Most studies of the fatigue of structural steel members and details have been conducted with constant amplitude fatigue tests using relatively large stress ranges resulting in lives on the order of one million cycles or less. Little work has been done on the effect of stress histories due to actual truck loadings.

(3) The development of probabilistic analyses to: (a) describe the nature of the traffic loading in a form which can be used as input to a structural analysis of the bridge to predict the statistical parameters describing the stresses at locations critical for fatigue, and (b) relate the predicted stresses and the fatigue damage law and
material parameters. One must take into account traffic volume, the proportion of heavy trucks in the traffic stream, a statistical description of gross vehicle weights, axle load distribution, vehicle placement on the bridge, etc., to forecast the stress events to be expected. Given a deterministic description of the stress events, fatigue damage itself is also a random variable.

In this investigation highway bridges of medium to short span only are considered. These have dead to live-load ratios such that stresses induced by the traffic can be a significant proportion of the total stress carried by the bridge structure so that the fatigue problem is of significance. Long span structures are not considered, but there are many short-span elements of long-span bridges such as floor beams, stringers and deck systems which involve fatigue considerations which are within the scope of the present studies.

1.2 Specific Objectives

In meeting research needs associated with the stress history problem, the present investigation has as its specific objectives:

(1) Development of technique and experience in field testing of highway bridges and in data acquisition using magnetic tape recording equipment, giving reliable records in a format for subsequent computer oriented analysis and interpretation.

(2) Development, testing and operation of a group of computer programs for the conversion, reduction, statistical manipulation, plotting and, in part, interpretation of the data. These computer programs are important to the success of an extended study because they permit the efficient handling of large volumes of experimental data.
Collection of a statistically useful body of data for bridge response under actual truck traffic. To accomplish this phase, field tests on three bridges were conducted. Although emphasis was placed on the effects of highway truck traffic, a controlled vehicle was used to study the dynamic response characteristics of two bridges for a known loading. Since major emphasis was placed upon obtaining as complete data as possible on the characteristics of the highway traffic, it was deemed essential to select two test structures adjacent to State truck weighing stations. Tests on one structure emphasized the use of controlled loadings only.

Interpretation of the stress-history data using response predictions based on earlier studies, statistical descriptions of the heavy truck traffic stream, and other problem variables as required.

1.3 Applications to Practice

The immediate application of the present research project is as it served as a pilot study for the expanded field program for the acquisition of data on the effects of traffic on bridges (IHR-301, Life Expectancy of Highway Bridges - Stress History Studies). In the course of conducting the field tests described herein, a start was made on the development of a field test capability in the bridge research group of Illinois Department of Transportation. This capability will make possible the expansion of future field studies including ad hoc studies of problem structures which may be identified during the course of this and future investigations.
The present investigation is one of a number of research efforts contributing to the problem of the prediction of life expectancy of highway bridges. The full impact of the application of the present study and work like it will be felt in the planning and design of bridge structures only after substantial progress has been made in the research areas outlined in section 1.1. However, in the context of the present state-of-the-art of research on the behavior of structural materials under fatigue loadings and cumulative damage theory, the results of the study could be used directly, with extrapolation of the traffic data, to predict the life expectancy of the bridges tested, although such an exercise is not proposed for this project.

From the results of the present investigation and from study of the literature, several conclusions can be drawn concerning the direct application of the present study to practice:

(1) The commonly used cumulative damage law is the Palmgren-Miner hypothesis. Only limited data on the behavior of structural steels under repeated loads of random amplitude is available. Present constant cycle fatigue studies on specific structural steels and connection details must be related to stress predictions in highway bridges. It is virtually impossible to attempt to instrument in the field a wide variety of suspected critical structural details, nor is it possible to instrument or test in the laboratory a wide variety of structural details and steel types. A unifying theory is essential.
(2) There is available limited data on truck traffic volume and correlated data on gross vehicle weight for a given bridge. This data must be collected and incorporated into a theory. The prime information lacking is that on weight and dimension of the vehicles; usually the average proportion of heavy truck traffic to passenger car traffic is reasonably well known. Very limited data is available on the distribution of trucks and correlated gross weight in the various lanes of a multi-lane structure.

(3) The influence surfaces or lines for static and dynamic stresses incorporating most of the significant parameters, are reasonably well known for certain classes of bridges. These influence surfaces now must be studied as functions of random variables taking into account the statistical descriptions of traffic loadings to produce a rational prediction of expected stress histories.

Under present (1973) conditions of traffic volume and vehicle size, bridge fatigue behavior has been, with some important exceptions, satisfactory. However, bridge engineers, law enforcement officials, and planners are faced with the problem of evaluating requests to increase allowable gross vehicle loads and to formulate guidelines for designs to accommodate increasing traffic volumes on existing and future components of the highway system. As an aid to rational decision making, analytical models and methods are also needed to determine the real cost to the user and the taxpayer of the use of the national highway system by various transportation modes.
While the present study cannot provide a prediction of the exact time remaining until structural failure is imminent or an estimate of remaining life to balance against the cost of replacement, the study does provide a base of data for selected bridges corresponding to present adequate structural behavior, from which one can extrapolate to evaluate future changes such as increased legal load limits. Certainly, before any substantial increase is permitted in loads or changes made in design criteria, one should have an adequate picture of the present conditions under which the bridges are performing satisfactorily.

Finally, comment should be made concerning the deflection data which has been collected during the investigation. In addition to being closely correlated with strains, these data are potentially useful in developing criteria for allowable deflections of bridges. Vertical deflections (and accelerations) and the characteristic natural bridge frequencies of vibration are significant in determining the degree of pedestrian or user response to bridge vibration (1).

1.4 Vibration Theory and Significant Bridge-Vehicle Parameters

The dynamic response of short and medium span highway bridges subjected to the effect of moving vehicles has been studied extensively, including Project IHR-9, Investigation of Highway Bridge Impact, which preceded the present study. The theoretical studies, here and elsewhere, include work on simple span, continuous and curved types bridge configurations making use of various idealizations of the bridge structure and vehicle.

*Numbers in parenthesis refer to items in REFERENCES
Extensive work was conducted at the University of Illinois at Urbana-Champaign (2-5) making use of an idealization of the bridge as a beam, that is, an idealization in which the entire bridge cross section deflects as a unit. This work was successfully verified in comparisons with the results of the AASHO bridge tests (6, 7). A more complete theory for the simple-span bridge was developed by Oran (8); he used an idealization of the bridge which was more nearly exact, that is, as a flexible plate supported on longitudinal girders. More recently Eberhardt and Walker (9) have completed a study of a more refined representation of the bridge making use of a finite-element model. This latter study has been extended to include both simple span and continuous bridge types and can be modified to handle skewed or curved structures. It has been verified by comparisons with the earlier work of Oran, with laboratory model studies, and field measurements of the transverse distribution of moments and deflection in bridges and with solutions from the earlier beam idealizations.

Based on a relatively complete theoretical background and using certain simplifications for convenience, one can summarize the significant parameters of the problem:

1. Vehicle speed.
2. Gross weight of the vehicle relative to that of the bridge.
3. Spacing of the axles relative to the span length.
4. Frequency of vertical vibration of the vehicle relative to the fundamental natural frequency of the bridge.
5. Damping in the vehicle suspension, with particular emphasis on friction in the vehicle springs.

6. Roughness or unevenness on the approach roadway and bridge deck.

7. Parameters related to the transverse distribution of deflections, moments or accelerations across the bridge. These include aspect ratio, skew, stiffness of the girders and slab, torsional stiffness of the girders, stiffness and geometry of transverse stiffnesses or diaphragms, etc.

The effects of the various parameters of the problem have been analyzed in detail (4, 5). No attempt will be made to summarize these findings, except to note that three parameters, speed, roadway unevenness or roughness, and transverse location of the path of travel of the vehicle, are important.

Making use of the theories noted above, direct computer simulation of the effect of a sequence of vehicles is an alternative approach for the determination of stress histories. A realistic simulation requires a sufficiently complete bridge idealization so that stresses in various locations in the structure can be evaluated; a simple beam model is usually not adequate for this purpose. Based on work with the finite-element model, a very complete bridge idealization and using also a good vehicle idealization, it was found that the cost of solutions, particularly for continuous bridges, became prohibitive when large numbers of events must be simulated.
However, these theoretical developments provide a guide for interpretation of the data obtained in the field and a rational basis for summarizing and extrapolating the results obtained. Further comments on the use of theory is made in Chapter 5.

1.5 A General Note on Bridge Selection and Instrumentation

In planning the test program some general criteria influenced the choice of test bridges, instrumentation and test procedures. These are described in this section; additional comment on the application of these criteria will appear in Chapter 2, as appropriate.

The primary constraints on the choice of test bridges were:
(1) that characteristics of the truck traffic crossing the test structure must be measured by some means, basically by the use of a State truck weighing station, and (2) that at least one structure be so situated as to make convenient the use of a controlled test vehicle at various speeds along various paths of travel. A secondary consideration was proximity to either the University campus at Urbana-Champaign or to Ottawa, Illinois, the home base of bridge research group of the Illinois Department of Transportation.

Although the State of Illinois maintains a number of recently built, well equipped, weighing stations, only a small number of these are adjacent to suitable test structures. Two other requirements increased the difficulty of bridge selection in the early stages of the investigation: (1) need for an uncomplicated bridge structure; and (2) no intersection on the highway intervening between the weigh
station and the test bridge. Also, for considerations of safety and minimum interference with traffic it was necessary to reject bridges on two-lane highways and consider only those on limited access, divided highways.

Ideally, the test structures should be a right, simple-span steel-girder and slab type structure of 60 to 80 ft. (18m to 24m) in length. If such were not to be found, a comparable structure of either two or three span continuous construction, without skew, was thought to be acceptable. It was desirable that the bridge be across either a stream, minor secondary road, or railroad to have safe and convenient access to the underside of the bridge for the installation of instrumentation.

Effort was made by visual inspection to choose those structures appearing to be structurally "clean", i.e., without details such as unusual transverse expansion joints or longitudinal deck joints which were partially filled which would contribute to high structural damping or uncertain structural response predictions.

For at least the bridges where the test vehicle was used, it was hoped that it would be possible to have a straight approach to the bridge on a level grade so that maximum vehicle speeds could be obtained.

With respect to instrumentation and test procedures, it was felt essential that the entire program be developed around a fully automated data acquisition and reduction system. It was felt that conventional light-beam oscillographic equipment was outmoded and that analog magnetic tape equipment should be the basis for all major recording in the field. Implementation of this data acquisition system required analog-digital conversion equipment, computers and a
substantial amount of programming. Analog magnetic tape recording equipment was available in the Department of Civil Engineering, University of Illinois at Urbana-Champaign (UIUC), to meet the needs of the field study.

1.6 Object and Scope of Interim Report

This interim report covers work on all phases in the work program for the subject investigation, but with particular emphasis on the physical aspects, testing procedures, and methods of data reduction and analysis. A preliminary discussion of the bridge response based on selected, partial results of the data reduction and analysis is presented. It is intended to sketch the broad outlines of results which are anticipated in the study based on a limited amount of bridge data and to introduce lines along which the analysis of results will be developed in the final report.

Chapter 2 contains a description of the bridges tested, the FHWA test vehicle, the Illinois truck weighing stations used, bridge instrumentation, and field test procedure. In Chapter 3 the reduction and analysis of both bridge and vehicle data is described. Particular emphasis is placed on the programs for the computer oriented data acquisition, reduction and analysis system. The work on a data acquisition and data handling system and the successful implementation of the computer programs described represent an important developmental aspect of the investigation and have permitted an expanded field test program (Project IHR-301).
In Chapter 4 representative results are presented and discussed. Selected bridge response quantities which are correlated with vehicle data are introduced. Although the results are preliminary in nature they provide an over-view of the general levels of strain response. Although only partial data on bridge response are considered in this report, complete data on the vehicle gross weight, wheelbase, and type will be summarized and discussed in Chapter 4. Of particular importance are the statistical descriptions, mean, variance and probability density functions, needed to relate the data to a more general theory for the prediction of stress histories.

Some preliminary comments on the development of a stress history theory are given in Chapter 5. Finally, in Chapter 6 a brief summary is presented.
2. TEST PROGRAM

2.1 Bridges

A summary of data describing the structures tested during the investigation is given in Table 2.1; sketches are shown in Figs. 2.1 - 2.5 and photographs are shown in Figs. 2.6 - 2.8. The three bridges are designated and described as follows:

(1) The Salt Fork River bridge, on U.S. Route 150 at the west edge of St. Joseph, in Champaign County, Illinois. This structure was chosen because of its proximity to the University and was tested using the FHWA test vehicle.

(2) The Shaffer Creek bridge, on Interstate Route I-280 and I-74 near the Quad-Cities in the village of Coal Valley, Illinois. The bridge tested carries the eastbound lanes over Shaffer Creek. It is located on an interstate segment which will eventually form a south bypass of the Quad-Cities area; however, at the time of testing the bridge over the Mississippi River which will complete the bypass was not under construction. The Shaffer Creek bridge is within a mile (2km) of a weight station, without intervening intersections.

(3) The CB and Q bridge on I-80, east of the Quad-Cities. Located on a north-south segment immediately east of the Mississippi River, the bridge carries the west bound interstate lanes over three tracks of the CB and Q railroad. The bridge is located approximately 3 miles (5km) from the weigh station; however, there is an intervening intersection with Illinois State Routes 92 and 84.
Structures (2) and (3) were chosen because of their locations near weight stations. Both have moderately high traffic volumes. Shaffer Creek bridge was close enough to the weight station so that a positive identification of the weighed vehicles could be maintained between the station and the bridge. The three structures are relatively simple designs, without skew, and of longitudinal steel girder and reinforced concrete deck construction. Span length, beam size and other design details are included in Table 2.1. Both the Salt Fork and CB and Q bridges are three span continuous; the former has a ratio of side-span to center-span of 0.8, a common value, whereas the latter was a smaller span ratio, 0.66. The Shaffer Creek bridge is a short two-span continuous structure with a longitudinal separation joint located at the roadway centerline marking. The bridge tested, which carries eastbound traffic over Shaffer Creek, thus consists essentially of two structures side by side with a free longitudinal separation joint, one having four longitudinal beams and the other five beams.

Fundamental frequencies of the bridges computed on the basis of beam theory, corresponding measured frequencies and values of damping in the fundamental mode expressed as a percent of critical are included in Table 2.1. Computed frequencies are reported for both full composite and non-composite action. The damping values are determined from study of free vibration records. In the damping analysis it was found that both the primary beam mode and torsional or antisymmetric modes of vibration were both present during the free vibration era in some records; thus it was sometimes difficult to make an accurate determination of the logarithmic decay in the vibration amplitude. However, the
apparent damping values are low. The frequency and damping values reported correspond to the unloaded condition of the bridge; under loads, it is anticipated, particularly in the case of the Shaffer Creek bridge, that there would be some increase in the effective damping because of the breakdown in composite action between the deck and the beams.

A study of the modes and frequencies of the Shaffer Creek bridge using a finite element model was made by Eberhardt and is reported in detail in Ref. 9. Although not supported by the project, these results are summarized in Appendix A. The values of natural frequency reported are based on stiffness values corresponding to approximately 50 percent, partial, composite action. The fundamental frequency corresponding to the lowest symmetric mode of vibration, 6.6 Hz, is bounded by the full composite and non-composite frequencies reported in Table 2.1 for the beam idealization of the bridge.

All bridges tested were of noncomposite design; however, in the field they exhibit varying degrees of composite action, which is actually time dependent. Further discussion of this point, supported by experimental data and theoretical analysis, is given in Chapter 4.

The deck surfaces for all bridges were in good condition. A longitudinal deck profile measurements have been made on the Shaffer Creek and the CB and Q bridges and are available and were used for input to analysis for dynamic response, but will not be discussed.
2.2 Test Vehicle

The FHWA test vehicle is a three-axle, tractor-semitrailer combination which can be loaded to simulate various axle load configurations up to a maximum gross weight of about 75,000 lbs (330kN). Usually the vehicle is loaded to simulate the AASHO HS20-44 vehicle. The vehicle is shown in Fig. 2.9 and pertinent data are given in Fig. 2.10, including gross weight and individual axle loads for the vehicle as used in the various phases of the program. A half-load as well as a full load condition was used in the studies on the Salt Fork River bridge. Data on the wheel contact areas is given in Table 2.2; these were obtained for the full load configuration during the Salt Fork River test.

FHWA Test vehicle instrumentation includes:

(a) Electric resistance strain gages on the trailer axle and tractor drive axle to measure moment or shear in the axle as an approximate measure of the dynamic load carried by the tires. This data is recorded on a conventional light beam oscillograph mounted in the truck. Measurements of tire pressures or wheel hub accelerations were not available.

(b) An indication for the longitudinal position of the vehicle. A microswitch mounted on the front bumper of the vehicle connected to a vertical rod was positioned so as to strike a block placed on the pavement at the desired reference point. The signal initiated by the tripping of the microswitch is recorded on the oscillograph records taken in the truck.
(c) A speedometer and tachometer, which are part of the normal vehicle instrumentation. These were sufficiently accurate so that the driver could keep a nearly constant predetermined speed. Independent speed measurements were made using the measured travel time from entrance to exit of the bridge.

(d) An FM radio system providing voice communications between the instrumentation van, the test vehicle and an observer near the bridge.

The vehicle could attain maximum speeds of approximately 60 mph (27 m/s) on a level grade provided that sufficient approach roadway was available for acceleration. This latter limitation was of consequence only in the case of the Salt Fork River bridge where runs from both directions were made and a short approach run was available on the westbound crossings, limiting the maximum westbound speed to about 40 mph (18 m/s).

The natural frequencies of vertical motion of the test vehicle were determined during the Shaffer Creek operations from strain-time records, taken while the vehicle was in motion, for the drive axle, both left and right sides, and for the similar arrangement on the trailer axle. Qualitatively it was determined that vibration of the vehicle on its tires was characteristic of most of the records. Because of the relatively smooth pavement surfaces, the vehicle oscillated primarily on the tires and the vehicle suspension springs were apparently blocked by interleaf friction forces. Occasionally it was possible to detect a shift in frequency for a short time which would be consistent
with the vehicle vibrating on the combined suspension spring and tire system. Based on approximately 20 determinations of frequency it is estimated that the frequency of vibration of the vehicle on its tires alone \(f_t\) is approximately 3.3 Hz. The frequency of vibration on the combined tire and spring suspension system \(f_{ts}\) is approximately 2 Hz. These values are consistent with measurements reported for similar vehicles, principally those used on the AASHO road test (6,7).

2.3 State Truck Weight Station

The investigation made use of two State truck weight stations:

(1) For the Shaffer Creek bridge tests: Station No. 23, on I-74 and 280, located 2.33 miles (3.75 km) east of the I-74 interchange. Average daily traffic in 1969 was 6100.

(2) For the CB and Q bridge tests: Station No. 24 on I-80, located 1.5 miles (2.42 km) south of Illinois State Route 84. Average daily traffic in 1969 was 10,700.

Descriptive information on these weight stations is available in the document "Illinois State Truck Weigh Survey - 1968" and corresponding data for 1970*.

The weigh station electronic scales consist of four individual platforms arranged in tandem: A 50 ft by 10 ft (15.3m by 3.1m) platform for the trailer, two 5.5 ft by 10 ft (1.7m by 3.1m) platforms.

*Tabulations for the 1970 volume were furnished in unpublished form by R. R. Knox, Engineer of Traffic Studies, Bureau of Planning, Illinois Department of Transportation.
to accept the tandem axles of the tractor and one 20 ft by 10 ft (6.1m by 3.1m) platform to accept the steering axle on the tractor. Thus with one placement of the vehicle, the scale can weigh individually the steering axle, two tractor driving axles and the total load of all the trailer axles for the common tractor-semi trailer combination. Under the direction of the scale operator, the truck driver can be asked to place the vehicle in several positions to get individual measurements for all axles. The scale platforms are supported on an electric-resistance strain gage load cells of the self-aligning type, which are sensed on automatic ranging electronic balance detectors. An electro-mechanical adding and printing machine is used for automatic tabulation of individual axle weights, gross vehicle weight, time of printing and date.

The scales are periodically checked by the manufacturer. Also the scales are calibrated by the Illinois Division of Feed and Fertilizer and the tolerance allowed during calibration is plus and minus 2 lb per 100 lb weight. The scales at Stations 23 and 24 for both Summer 1968 and Summer and Fall 1969 were calibrated at approximately one month intervals bracketing the test periods.

During testing the procedure was as follows: The weight station attendant was asked to make a printed recording of every vehicle which crossed the scales during the time period when data was being taken at the bridge. Concurrently project staff measured individual axle spacings on a 100 ft tape measure fastened to the side of the
scale pit, recorded the vehicle type, provided a brief description, assigned a vehicle number, and at Shaffer Creek noted the progress of the vehicle while in view from the weight station.

For the Shaffer Creek bridge, Station 23 was sufficiently close so that trucks were in view of the weight station nearly until they reached the test bridge; thus, passing maneuvers or stopping on the shoulder which would disturb the sequential identifications of the vehicles could be seen. For the CB and Q bridge an interchange intervened between the bridge location and the weight station; also they were separated by approximately 3 miles (5 km) and a continuous observation of the vehicles was not possible.

2.4 Bridge Instrumentation

2.4.1 Strain and Deflection Gages

The basic instrumentation for all tests consisted of strain and deflection gages placed to measure structural response on selected cross-sections of the bridge. The major emphasis was placed on describing the response of the longitudinal beams at the center of both spans in the two-span structure, and at the 0.42-points of the side spans and at midspan of the center span, in the three-span continuous structures. Also, strains were recorded at cutoff points of cover plates (when present), at selected additional locations on the more heavily loaded longitudinal beams, and at sections over the piers to obtain an indication of negative moments. In addition, strain measurements were made on selected transverse reinforcing bars in the deck slab.
At points on the longitudinal girders chosen for strain measurements, pairs of gages were mounted on the desired cross section line, 2 in. (50mm) from the edges of the flanges on the bottom surface of the bottom flange. To check the location of the neutral axis at selected locations, single strain gages were placed 2 in. (50mm) from the edge of the undersurface of the top flange. At the coverplate cutoff points, pairs of gages were located approximately 1 1/2 in. (40mm) from the end of the coverplates, 2 in. (50mm) apart, symmetrically placed on the bottom surface of the beam lower flange.

The number of data channels (gage locations) recorded in a given test ranged from 55 at the Salt Fork River bridge to 12 channels for the Shaffer Creek and CB and Q bridges in 1969.

The deflection gages were a standard type devised by the FHWA and used widely for bridge field testing. The gage consists of a cantilever beam, fixed end attached to the bridge, of constant thickness and linearly varying width which is a maximum at the fixed end; the free end is anchored to the reference datum. Strains are induced which are proportional to deflection. Deflection measurements were not made for CB and Q railroad bridge since suitable anchors could not be established readily because of the presence of the tracks.

A summary of gage locations for strain and deflection measurements during the four test periods is presented in Tables 2.3 - 2.7.
2.4.2 Standard Traffic Counter

To provide a count of all traffic at the test sites consistent with standard traffic surveys, a traffic counter was installed at each test bridge. The counter, with pneumatic-tube detector, provided printed record of counts at one hour intervals over the entire test calendar period, not just during bridge data acquisition.

2.4.3 Data Recording

During the investigation several combinations of FM analog magnetic tape recorders, oscillographs and auxiliary equipment were used. For the tests of the Salt Fork River bridge and the Shaffer Creek bridge in 1967 and 1968, respectively, three magnetic tape recorders from the University and one from the FHWA were used. Recorders were used both parallel with the FHWA CEC System D Oscillograph equipment and directly with signal conditioning equipment supplied by the University. The task of providing a reliable time synchronization pulse for all recorders was not accomplished for all tests. No attempt was made to transfer the start pulse or position marker pulse from the CEC System D equipment to the tape recorders in 1967. The scheme used in the 1968 Shaffer Creek tests for establishing a vehicle entrance pulse on all data channels was not reliable, although some use of this pulse was possible. However since the data interpretation requires only qualitative study of the waveforms, little difficulty ensued, although a certain amount of trial and error manipulation was required to give approximate time correlation, before computer processing was completed. Care was taken to put data channels which might require
simultaneous study with an absolute time reference on a single tape recorder. In the early field tests more channels of data was taken than was needed in order to provide back-up information in case unexplainable phenomena were encountered.

Field test procedures evolved to take advantage of equipment acquisition by the University so that in the last two field tests, in 1969 at Shaffer Creek and the CB&Q bridge, all data channels were recorded on a single 14-channel machine. The features of the logical control required and developed for this system is described in the next section. The system used for these two tests, because of its successful and convenient operation, served as a model in specifying the operating characteristics of the data recording system acquired by the State of Illinois for the cooperative effort in the IHR-301 program.

2.4.4 Control Logic Requirements

Automation of the data acquisition and reduction process requires certain information to be recorded in the field on the analog tapes to permit control by the computer based data reduction system. The traffic conditions and the data taken are characterized by discrete identifiable events, the individual truck crossings, for which individual records of finite length are taken. Each record is composed of one or many channels of data, but is considered as a single event which is identified and stored in the computer as matrix of data.

It is important for the operation of the automated data system, which will be described in Chapter 3, to have consistent recordings in which
the amplitude of the data, the calibration steps and the timing and
vehicle position information are of consistent quality so that limits on
the computer programs and logical programming steps can be set initially
and function consistently. The needed calibration steps and timing
signals are applied by a switching system which introduces the calibra-
tion steps, timing signals and initiates the recording process in a
sequential, timed, operation. A detector is used to turn on the entire
system as the vehicle approaches the bridge.

Thus, a scheme for detecting the presence of vehicles is
used for two tasks in the automated data acquisition system; first to
detect the presence of an approaching truck to start the equipment and
second to mark the instant at which a vehicle enters the bridge and
later when it exits from the bridge.

Two devices were used to detect vehicle entrance onto and
exit from the bridge: (1) a pneumatic tube and pressure switch assembly
-- used for the Salt Fork River bridge and the Shaffer Creek bridge in
1968, and (2) a photocell and light source system -- used in subsequent
tests.

To start the recording equipment for automated field operation,
three different devices were used during the course of the investigation.
These were as follows:

1. A pneumatic tube and switch -- used for the Salt Fork
   River and Shaffer Creek, 1968.
2. A photocell and light source device -- used for Shaffer
   Creek in 1969.
3. A sound level detector -- used for CB&Q bridge, 1969.
In use, the device was placed a sufficient distance before the bridge so that, considering the usual vehicle speeds expected in the traffic, the time interval after detecting the vehicle was long enough so that tape recorders could be brought up to speed before the beginning of the switching of the logical events required for control of data acquisition. The first two schemes listed detected the presence of all vehicles crossing the line of sight of the photocell system or the pneumatic hose and started the system for all crossings. Since data acquisition was not desired for passenger cars, such events had to be edited out of the data by hand or on the basis of the computer-based test during the data reduction process. Alternatively, the automatic-start system could be defeated manually in the field and unwanted passenger car records not taken. This latter approach was used at Shaffer Creek in 1968.

The inefficiency associated with taking unwanted data and the fact that an excessive amount of recording tape was used, led to the development of the sound level detector and its introduction in 1969. The sound level detector consisted of a microphone placed near the side of the road and an amplifier and relay which upon the impingment of a sufficiently high sound level closed a circuit which started the recording system. Calibration of the system was not required; the audio level was simply adjusted until it was reliably activated by the noticeably louder truck engines and not passenger cars. Only in very rare instances would trucks pass without initiating a start of the system.
Of the two schemes noted for detecting the presence of the vehicle on the bridge, the photocell was the most satisfactory. The pneumatic tubes tend to be somewhat unreliable and do not produce a consistent pulse shape via the pressure sensitive switch. On the other hand, the photocells are essentially instantaneous in their reaction and are sensitive enough to give a pulse shape, depending upon the height of the light beam above the road surface, which detects the passage of individual axles and is even sensitive to objects hanging below the truck, such as fuel tanks and props. Under ideal conditions, the photocell can produce a signal which constitutes a "signature" which could be used even to identify the truck type or axle configuration.

With regard to test procedures, the photocell devices, even those for the start signal, were sensitive to changing ambient light conditions during the course of testing. Adjustments had to be made in the sensitivity depending on the sun angle and spurious reflections from chrome, hubcaps, etc. It was essential for the data reduction process that the pulse shapes generated by the photocell system be consistent in shape and have a clean rise from the zero level to the peak so that detection by an amplitude test could be made without errors. Deficiencies which resulted in signals of unanticipated length and amplitude could be corrected by software modifications but these changes must be consistent for substantial numbers records since they cannot be included as variables during the course of a particular data processing run.
3. DATA REDUCTION AND ANALYSIS

3.1 General

The significant data reduction problems are related to the bridge; although both truck and bridge data reduction will be discussed, the latter will receive the most attention. A general flow diagram for the data processing is shown in Fig. 3.1. An important end product of the research project is the development of a computer based, data acquisition system. The degree of automation and the amount of user manipulation required varied with the type of analog-digital conversion equipment used. As the equipment available changed, the data acquisition system evolved. During the course of the investigation three systems were used:

1. A single-channel analog-digital converter with punched card output. This system, used only for part of the Salt Fork data at the beginning of the investigation, was slow. Only one channel at a time could be reduced and a common time reference was not established for all data channels.

2. A multi-channel analog-digital conversion system which was a part of an IBM 1800 installation at the University Digital Computer Laboratory. Because of uncertainties in the continuing availability of this equipment, software was not developed for automated control logic. Experience with this system contributed greatly to the development of the logic for subsequent improvements. A substantial number of records for Shaffer Creek (1968) were reduced on this system before it was taken out of service.
(3) A multi-channel, analog-digital conversion system, including both software and hardware, using a mini-computer purchased by the Department of Civil Engineering, University of Illinois at Urbana-Champaign. This system, which became available to the investigation on a continuing basis, has been developed extensively.

Not discussed in this chapter are the problems associated with the reduction of conventional paper oscillograph records. These were digitized (i.e., coordinates describing the time-histories were measured, scaled and digitized) in an operation using a Benson-Lehner Oscar E oscillograph reading machine. Tedious operator effort was required, but the resulting output was on cards for computer input. The computer based data handling system could then be used and the procedure is basically the same as that of system (1), above.

With system (3), analog magnetic tape records taken in the field which contain the proper logical control information, can be reduced to digital form, assigned identifying labels, and stored on computer compatible digital magnetic tape ready for further analysis on the central computer system (IBM 360) without the need for frequent operator intervention during analog-digital conversion process. This is in contrast to the reduction of paper oscillograph records or the single channel system (1) where tedious hours were required for slow operating card punch equipment to prepare data for transfer to the computer. The present system digitizes data in a "real time" operation.
It should be noted that large volumes of data are handled or transferred from one computer to another (Spiras-65 computer system for A-D conversion to the IBM-360/75) by means of magnetic tape. Even under carefully controlled conditions of maintenance and user care such an operation involves equipment subject to operational difficulties. Without elaborating on technical details, delays are often encountered in getting computer production runs which insure the complete transfer of error-free data. A direct (wired) link between the computers will perhaps provide amelioration for this difficulty for future work (IHR-301, perhaps).

The acquisition of data on the heavy vehicles in the truck traffic stream was carried out at the truck weighing stations described previously. The reduction of data for individual vehicles was straightforward and included initial examining to check consistency, computation of individual axle weights or spacings not obtained in the field, key punching of the data and finally loading of pertinent information for each vehicle into a computer file system on magnetic disk storage or tape.

Finally, it should be emphasized that these techniques are based on the premise that traffic conditions are such that vehicles cross the bridge and response is recorded as events which are identifiable, in sequence, or nearly so. A moderate number of multiple cross events can be handled. Under very heavy traffic conditions this approach will not be feasible when a steady-state, multi-lane flow of truck traffic develops. However, modifications can be made to take
samples for specified periods in the time domain rather than identify
discrete vehicle crossing events. Basic decisions related to the
amount of digitized time history data to be retained for future analysis
and the procedures for determining periodic calibration values and
no-load response levels will be required.

3.2 Bridge Data Reduction

A basic decision underlies this discussion, namely, that it
is essential to retain a representation of the entire bridge response
time-history. Data reduction is simple and storage space is greatly
reduced if it is deemed adequate to use a single quantity to represent
a time-history: for example, total strain range. If strain range is
sufficient, it is only necessary to measure and record the excursion
from maximum positive to maximum negative strain. For such, paper
oscillograph records are feasible, but not desirable. Similarly,
processing data on magnetic tape is greatly simplified when one need
search for a single maximum strain range, or, on a more elaborate basis,
for the relative excursions or partial ranges for several successive
maxima and minima in the record. Although a large number of computations
and comparisons are still required to search for maxima, the storage
needed for the final result is much less.

The primary method used to record the analog signals was
analog magnetic tape. As has been noted, these records at first were
digitized on a single-channel Northern Scientific Corporation Model 554
analog-digital converter and serial, low speed card punch to provide
output. This relatively slow process took about twenty minutes to
digitize and punch cards for one time-history for one channel. Calibration data was digitized separately. This method was unacceptable for digitizing large numbers of traces.

A new system for A-D conversion became available for processing the Shaffer Creek 1968 data. This system used an IBM 1800 computer at the Digital Computer Laboratory of the University of Illinois at Urbana-Champaign which was equipped with a high speed, multi-channel analog-digital converter. The process was controlled by the user interacting with the computer in a remote terminal operation. The operator must input general information to the computer concerning the test run to be digitized, locate the test data on the analog magnetic tape, start the computer and the tape playback, stop the tape recorder at the end of the run, and give the computer appropriate terminating commands. The digitized values were written on computer magnetic tape for storage. Calibrations were digitized in separate operations. The system was used to digitize seven channels of data simultaneously. The entire process took about five minutes; the actual digitization required only a fraction of this time. This system represented a major improvement and was used to digitize about half of the data from Shaffer Creek 1968.

The current digitizing system uses a small digital computer with a high speed, multi-channel analog-to-digital conversion capability and IBM compatible magnetic tape as the output device. The basic unit is a Spiras 65 mini-computer and is part of the instrumentation
laboratory of the Department of Civil Engineering at the University of Illinois at Urbana-Champaign. It is used in an automated, continuous process requiring a minimal monitoring by the operator. Additional data is taken in the field to allow this automation. The time-histories from all gages and certain logical information are recorded on one 14-channel tape recorder in a standard format (i.e., control signal form and time sequence). In addition, calibration steps are put on each channel before each truck run. Automation of the data reduction was achieved by software which makes decisions based on control amplitudes and event sequences. The analog magnetic tape from the field is played back into a multi-channel (16 maximum) analog-digital converter; the output from this device is input to the mini-computer, and software controls the flow of data. In the current application, the system digitizes all 12 data channels for each run, stores the values on nine-track digital magnetic tape, and proceeds directly to digitizing the next run. Processing all channels of bridge data for one truck crossing is done in real time (about five seconds or less). The computer logic and software for the Spiras-65 was developed by Spiras Systems, Inc., the suppliers of the system hardware; however, the broad logic of the process was part of the work of the investigation. A number of modifications to the Spiras software have been made on a continuing basis by V. J. McDonald. This system was used to digitize more than half of the data from Shaffer Creek 1968 and all data for Shaffer Creek 1969 and CB & Q.
The sequence of events recorded in the field on the analog tape during the passage of one truck is presented in the sketch of the time-variation of an analog recording shown in Fig. 3.2; critical points in the recording have been labeled. Consistency in the sequence of these points is essential to the automatic operation of the program.

At point (A) the tape recorder is started by a signal from the vehicle sensing device. After a pre-set length of time to allow the tape recorder to reach operating speed, a calibration step is applied on each data channel at (B); this calibration step is terminated at (D) after a preset time interval. Meanwhile a 1.0 kHz timing signal has been started on channel 1 at (C). The truck enters the bridge at (E) and the event is marked by change in the output of the photocells (channel 2); a positive voltage (trace deflection) indicates the entrance of a vehicle. At (F) the truck leaves the bridge as is indicated by a negative voltage on channel 2. The timing signal is terminated at (G), and at (H) the tape recorder is stopped ready for another cycle. If another truck passes the vehicle sensing device before point (G) is reached, then the time to (G) is extended to allow that second vehicle to cross the bridge and be part of the same record.

The software logic that controls the acquisition process is based on a standard sequence of events. The program monitors channel zero until the timing signal starts at "C". It is required that channel zero remain at or near zero for some fixed amount of time if the program is to proceed. This step insures that the tape recorder was up to speed.
after its period of rest between crossings (A) and, also, prevents spurious starting of the acquisition routine. Application of the timing signal at (C) occurs after the application of calibration signals to all channels (B). After a short time delay (0.2 seconds typically) to permit decay of any switching transients, the systems samples and stores in its memory sixteen sequential values from each data channel. The average of these values is the calibration amplitude. Following this computation, channel two is monitored until a drop in its amplitude indicates that the calibration has been terminated. Following another delay for transient suppression and to insure that all calibrations are off, another sixteen values for each channel are obtained. The average of these values is the calibration zero level and is also used as the initial level for each channel (base or zero signal level).

The individual calibration and zero values, the average values for each channel, certain identifying and descriptive data are written as the first record or "Header" block of data on the digital tape for that specific crossing. The Header identification specifically includes a run number which serves as a unique identifier of the data to follow.

Following the writing of the Header Block, the program continuously samples channel two and waits for a positive photocell indication of vehicle entry to the bridge. When a positive voltage is sensed at the photocell the program samples, digitizes, subtracts the zero value from each measurement and writes on digital magnetic tape sets of values for channels two to thirteen. Each record block contains sixty samples for each channel. Groups of samples are
The data sampling continues for a preset "time-to-go" after the photocell has again dropped to zero from its positive level. A total sample time of three seconds has been used as adequate not only to permit vehicle exit from the bridge but some free motion of the structure following the exit. The photocell data is used for this start control and is also processed for the data which permits vehicle velocity computation. After the "time-to-go" has been used, the system will go to a 'wait' mode if the time channel (1) is still on and will revert to its starting conditions if the time signal drops to zero. During the 'wait' mode, entry of another vehicle, restarts the digitizing and storage routine without writing a new header block of calibration data.

Because of the moderate sampling rate all computations can be performed between samples and the analog tape can be played back at the speed at which it was recorded. The end product is a digital magnetic tape with groups of record blocks consisting of one header followed by the data. The header record block contains the calibration step and zero values for each channel, and the data record blocks contain the unscaled but zero-corrected digital values for all data channels and the photocell channel.

The digital tape generated by the Spiras-65 in the above computer operations is compatible with the IBM 360 system at the Digital Computer Laboratory of the University of Illinois at Urbana-Champaign, and all subsequent data processing, storage, and analysis is done on the IBM 360.
The raw digital data must be calibrated, sorted, catalogued, and stored in disk files or on magnetic tape before it can be analyzed. Two systems of software have been developed to perform these functions, but only the most recent of the two (called TAPE STORE) will be described.

The TAPE STORE program searches the raw digital data on magnetic tape (i.e., that was written by the Spiras-65) for a specified header record block which it then reads followed by the corresponding data record blocks. Calibration factors for each channel are calculated using the calibration step and zero values from the header and other input information, gage factors, gage resistance, etc., supplied by the user. Each data ordinate is scaled using the calibration factors to the appropriate engineering units, i.e., strain or deflection, and the bias in the baseline of each trace is readjusted so that they all start at zero. The speed of the truck is calculated from the output of the photocells, and the time abscissa is scaled to yield a nondimensional set of position coordinates in which the time for a single axle to cross the length of the bridge represents unity. Each time-history is catalogued with a unique identifying number and finally the sets of scaled ordinates and abscissas are stored either in a random access disk file or in a sequential file on magnetic tape. The set of abscissa values for each time-history for any one run are identical, so they are only stored once. This process is repeated for each truck run.
Checks on Data Quality. The software controlling the digitization process and for loading the raw digital data to storage files thrives on consistency of the logical sequence of the recorded signals. There are abnormalities which the software cannot readily be programmed to handle. The most common difficulties are noise spikes on the control data channels, cars causing unwanted excursions on the photocell channel, malfunctions of recording equipment causing erroneous calibration steps, and trucks that were going too fast for the timing sequence. As a result, it was necessary to monitor and edit the raw digital data for the occurrence of such events and take steps to anticipate and avoid their effect on the software logic.

Also, it is desirable to complete a careful check of the data which will be loaded to the permanent storage files in relatively small batches so that mistakes can be corrected with a minimum of expense.

It is desirable to make corrections without time consuming interventions by the operator, but in practice considerable effort is required. Individual data points which are clearly in error cannot be sorted or "weeded-out" by scanning of the plots visually if data for a large number of response events are to be processed. This course of action was feasible for the Salt Fork River bridge because relatively small amounts of data was handled on punched cards. However, it is impossible when data transfer is by magnetic tape or disc since only complete blocks of data can be deleted or flagged to be ignored in subsequent operations. Deletions and
corrections must be automated and part of the computational process.

The analog-digital conversion process yields a vector of digitized data ordinates for each event. The abscissa or time coordinates are computed from the known sampling rate. For economy, digitizing of data should start at the time of entrance of a vehicle onto the bridge and be terminated as soon as possible after the vehicle has cleared the bridge. The programmed logic for doing this ideally should take into account occasions when multiple vehicles are on the bridge. In practice, however, it is necessary to note the multiple vehicle events and adjust with appropriate input parameters the length of record to be put in storage. Also at this stage, it is possible to adjust the amount of data to be input to permanent storage, i.e., it is possible to store every third, fourth, or fifth data point of the sampled data vector, since sampling is almost always at a rate faster than needed to define the time-histories adequately for analysis and plotting.

Also needed at this stage is a verification that appropriate calibration offsets or levels for each data channel are available. Only a simple computation is required to determine and apply the calibration constant, but a check is still needed to confirm the presence of a calibration offset for each data channel, as is assumed in the routine operation of the program. If a calibration offset is present for the main control data channel which controls the logic,
and not for the others, then the whole process can proceed without apparent difficulty, yet yield erroneous results.

When the calibration data is not complete, it is necessary to make an analysis of other calibrations available during the test and select a statistically reasonable, expected value, to be used for those instances where calibrations are not available. This process was required for some portions of the Shaffer Creek 1969 data and yielded interesting information on the stability of the measuring system. It was seen that the mean of the calibration offsets were extremely stable and the coefficient of variation for these offsets was less than 1 to 2 percent.

Finally, the recorded time histories may contain unwanted noise or "spikes" which by knowledge of the physical phenomena involved and by preliminary inspection of the records, are clearly false information superimposed on the data. Since the quantity of data is large, and for reasons noted previously, the unwanted data points cannot be rejected individually. The sorting of unwanted noise or spikes must be automated or a scheme developed to eliminate the unwanted signals. The basic operation used in the case of noise of a random nature is that of successive curve smoothings; it is conducted in part of the analysis system described in Section 3.4. For those records which contain spikes which are too large to suppress by smoothing, a spike detection routine is used to locate
spikes by comparing the increment of change between successive ordinates. When the change exceeds a prescribed limit, the spikes are considered to exist and the data record is flagged, i.e., its identification number stored. These record blocks can either be dropped from storage entirely or ignored in subsequent analysis and interpretation. When the number of spikes is not excessive, the records are usually not dropped from storage. Spikes are of concern in processing for maximum strain range or partial strain ranges.

3.3 Vehicle Data

The reduction of data taken to describe the heavy truck traffic is straightforward. It should be noted that the body of data available on truck characteristics represents in many instances a more extensive collection of data than is normally taken in the annual surveys at the respective weight stations. The data is useful both to define the characteristics of individual vehicles which are to be associated with the correlated bridge response events, and also to treat from a statistical point of view for use later in development of theory.

It may be recalled that data was taken on the following basic parameters:

(1) gross vehicle weight
(2) individual vehicle axle loads
(3) axle spacings

(4) vehicle type and description including axle configuration

(5) vehicle arrival time

(6) vehicle speed

(7) vehicle sequence (i.e., assignment of sequential identification numbers)

Also of interest, but not available, is data on the natural frequencies of vibration of the vehicles, vehicle suspension characteristics, damping and other factors which simply cannot be measured.

When a one-to-one relationship between vehicles and bridge events was established, Shaffer Creek 1968, the reduction of the data required checking to assure that the sequence of vehicles leaving the weight station was the same as the sequence crossing the bridge, in effect a verification that the identification number assigned at the weight station as the vehicles crossed the test structure. This required (1) transcription of field notes on weight, axle load, axle spacings, arrival time, etc. into a form suitable for punching on the data cards for loading to the computer file, and (2) a correlation of the vehicle descriptions available at the weight station with the description of vehicles on the voice track of the bridge analog
magnetic tape. For Shaffer Creek 1968 the vehicle speeds were computed by means of a timer operated during the crossing and thus the vehicle speeds were available immediately.

For field work subsequent to Shaffer Creek 1968 the vehicle speed was computed using the photocell traces which provided entrance and exit marks from which the transit time can be computed. This is adequate unless a multiple crossing event occurs; that is, when either two or more trucks are on the structure simultaneously or a passenger car passes the truck so that the sequence of the photocell pulses is not identifiable for the vehicle of interest. Possible error because of multiple vehicle events is handled by comparing the computed speed with an estimated upper limit. Normally the presence of a second vehicle would result in artificially high computed speed because of the false indication of a short time crossing. Thus when the speed computed exceeds the limit the result is noted and rejected. Obviously the photocell system for determination of vehicle speed is unworkable with a high volume of multiple vehicle crossings. Further the development is needed in this area and an alternate means of speed determination, perhaps radar, should be investigated.

When available, the vehicle speed is stored with the other vehicle parameters in the data files. Also, it is usual practice to normalize the time abscissa values of the response time histories in terms of the total time required for the vehicle to cross the bridge. Thus the vehicle speed and bridge length contained in the data file
are used to compute the scale values for the abscissa for plotting. When a vehicle speed is not known the time scale is normalized based on the mean traffic speed (about 55 mph). This scheme was chosen for consistent format when using the time-history plot routines.

The vehicle arrival time was noted by observers both at the weight station and the bridge. When heavy traffic conditions existed, arrival time measurements were not sufficiently reliable to define vehicle headway with a desirable degree of accuracy; the observers at the weight station were instructed to put emphasis on accurate recording of axle spacings and to check that these were consistent with the weight recorded by the scale printer. This difficulty prompted the use in Project IHR-301 of a digital clock which could be recorded automatically on the analog magnetic tape.

3.4 Computer-Oriented Data Analysis Interpretation

3.4.1 General

The important features of the data analysis and interpretation programs developed on the project are described in this section. The programs were developed over a four year period with credit for the early work due to E. S. Perry and for extensive modifications to J. A. Ruhl. The mathematical operations involved in the data reduction and scaling are simple and require little description. A great deal of effort went into the development of logic and checks to ensure that the data loaded into permanent storage was appropriately checked, identified, etc.
These programs are hardware dependent and are of interest mainly to the specialist. Also, a description would soon be of limited value because of the continuing updating and modification of the computer software. Detailed information is available in the project files.

While no attempt will be made to append listings of programs, it should be noted that the permanent data storage is arranged so that data - mainly time histories of strain - can be made accessible for transfer to other researchers using magnetic tape. Cards can of course, be generated but would be prohibitive in bulk, cost, and difficulty in shipment unless only a small number of time-histories were of interest. It is anticipated that the statistical summaries of the data presented in this and the final report will be sufficient for most uses.

### 3.4.2 Basic Data Processing Capabilities

Computer software (denoted DISK DATA) was developed to make computations and comparisons useful for studying the strain-histories of bridge response and the truck data. The basic structure of the system of software was developed by E. S. Perry; J. A. Ruhl was responsible for modifications to the basic system to extend its capabilities and provide for problems particularly related to the digital data tapes produced by the Spiras-65. This software is the main tool for data analysis on the investigation, and a summary description of its various functions are given.

The options available in the system generally are used in two situations: (1) for processing primarily individual time-histories, and (2) for processing a sequence of time histories to
accumulate data - for example, on strains range - to determine a histogram. The latter function is basic to the usual definition of "stress history."

The options primarily for processing individual time histories are:

1. **Curve Smoothing** As noted previously, the time-histories may contain high frequency noise which is desirable to remove before further analysis or study of the data. This is done by either of two curve smoothing subroutines; one passes a second order curve through three successive data points and the other passes a third order curve through five successive data points. It was found that up to five smoothings can be made without a significant change in the measured maximum response or apparently removing any dynamic components of the bridge time history.

2. **Spike Check** For processing traces which contain noise spikes; these are extremely large changes in amplitude between two adjacent points. Such spikes are easy to detect but difficult to remove by curve smoothing. A subroutine is available which detects their presence and, if found, flags the time-history for rejection if desired.

3. **Output** Three forms of output of the time-histories are available including: (1) a printed table of the data ordinates and abscissas, (2) the same information punched on computer cards, and (3) a computer generated plot of the data, appropriately scaled and labeled.
(4) **Extreme Values and Range of Response** A subroutine searches the vector of amplitude points representing the time history for the largest positive and largest negative values; the difference in these values is the total strain range.

(5) **Partial Strain Ranges** A subroutine is available to analyze the time-history for the amplitude of partial strain ranges. This terminology is defined and the process is described in more detail in Chapter 4.

(6) **Fourier Series Analysis** A subroutine is available to determine the coefficients of a Fourier series representation of the time-history; the results are presented in the form of a spectrum of coefficients versus frequency.

In addition to the above analyses which are made on the individual time-histories, subprograms have been developed for studies that are to be made on groups of time-histories and as needed, the corresponding truck data. However, curve smoothing and spike checking can be performed on each time-history included in a group to be studied. The subroutines available for processing groups of data are:

1. **Dynamic Increments** For tests made with the FHWA test vehicle, the dynamic increment time-history curves for strain or deflection for a particular speed run can be calculated by subtracting the appropriate crawl-history from the dynamic-history. The problems associated with selecting compatible dynamic and crawl curves are discussed in section 4.2.3.
2. **Neutral Axis Location**  The position of the neutral axis is calculated as a function of time for stringer locations where time-histories for both top and bottom flange strains are available.

3. **Transverse Distribution of Strain**  For a given truck run, this program studies the bottom flange strain-histories for all beams at a given section of a bridge. It determines the total maximum bridge strain as the sum of the maximum strains for each beam and the percent of this total strain carried by each beam.

4. **Total and Partial Strain Ranges**  The same routine that determines the total and partial strain ranges for an individual time-history will also perform the same function for a series of time-histories and accumulate a count of strain events in specified intervals of amplitude.

5. **Linear Regression Analysis**  A subroutine is available to perform a linear regression analyses to relate maximum positive strain or strain range to a specified vehicle parameter for a selected group of test runs. Gross vehicle weight, length, speed, and axle spacing $S_2$ can be considered as independent variables. The subroutine plots the data points, calculates and plots the line of best fit, and calculates the total error on deviation and the correlation coefficient.
6. **Multiple Regression Analysis** There is a subroutine which performs a linear multiple regression analysis to determine the plane of best fit for two independent variables and a dependent variable; gross vehicle weight and spacing $S_2$ are used as the independent variables and strain range is used as the dependent variable.

To complement the above system of software, additional individual programs have been written to perform such statistical functions as counting for histograms, calculating relative frequencies and plotting such data.

In making use of the data system, two procedures are usually used for specifying identification numbers of runs to be studied. The data identification number contains information describing both the channel and the individual truck crossing; the user normally studies the records for specified single data channel, i.e., a given gage location on the structure. Depending on the nature of the study, the user then specifies perhaps several or a long sequence of truck events. For the data in disc storage and, e.g., Shaffer Creek 1968 and 1969, the user must provide an identification number for each run desired, but in any random sequence. To facilitate this, a simple program written to generate the desired run numbers, or as in the case where relatively small number of runs are under consideration, the I.D. numbers are punched by hand. For the data from CB & Q - 1969 permanent storage is on magnetic tape, hence efficient random access to the data is not possible. Thus one normally specifies a sequence of trucks in
their order on the tape to facilitate the search of the tapes for reading. In practical usage this means that one might specify runs corresponding to a particular day or part of the day or longer time period. It should be noted, however, it is possible to locate individual runs and plot them in the same fashion as if random access were available using magnetic disc. To prevent damage and undue wear this operation is not frequently undertaken.

In the process of loading data to the permanent computer storage lists of the individual run numbers which have been loaded, together with certain other data concerning the runs as loaded are generated. This "directory" is valuable in locating particular portions of the stored information. Also retained in the files is an indication of the block numbers assigned in the original analog-digital conversion. These are usually correlated with the footages on the analog magnetic tape taken in the field so that if it is necessary to identify and resolve questions concerning an anomalous record, one can go back if need be to the original field records, without a time consuming search.

Based on the experience in the IHR-85 program a modified system scheme for doing many of the 'housekeeping' chores of filing information on run numbers, etc., has been developed for on-line remote terminal operation using a data files on the POLO system, a part of the Civil Engineering Systems Laboratory at the University of Illinois at Urbana-Champaign. Extensive use of the system has been made on the IHR-301 investigation.
4. PRESENTATION AND DISCUSSION OF PRELIMINARY RESULTS

4.1 Scope

Selected results from tests of the three bridges will be presented and discussed to give an indication of general nature of the bridge response, the truck traffic observed and the types of statistical information that is being developed on strain (stress) histories. The results will be presented, discussed and interpreted in detail in the Final Report.

Bridge response data for the passage of both the controlled FHWA test vehicle and highway truck traffic is considered. The effect of the FHWA test vehicle is presented and discussed in Section 4.2; both static or crawl and dynamic response are considered with emphasis on the characteristics of the time histories of deflection and strain.

To set the stage for discussion of the response of the bridge to selected heavy truck traffic, in Section 4.3 a review is made of the data on the characteristics of all heavy truck traffic measured during the investigation. The statistical data for the vehicle population considered during the entire study are presented in abbreviated form to indicate the nature of the histograms for the relevant parameters describing the vehicles. The relationship of the vehicle characteristics in the selected runs (for which strain data is presented) to the total vehicle population is discussed.

The discussion of bridge response under the action of selected truck traffic is presented in Section 4.4. The discussion is developed using time histories of response, histograms of peak strain range, studies of the location of the neutral axis and selected spectra for effects such as the relationship of strain range to gross vehicle weight, vehicle speed, vehicle length and vehicle transverse position.
Strains induced in the transverse reinforcement of the deck slab have been measured at sections close to the point of entrance on all test bridges. This data depicts the response of an element of the structure rather than of the entire bridge, and is discussed separately in Section 4.5. Selected experimental results are presented and the problems of predicting strains in the reinforcement are discussed. It should be noted that the analysis of the data requires a more refined study of the transverse placement of the vehicle because of the sensitivity of the slab strains to the position of the individual wheels. The gross behavior of the bridge is not nearly as sensitive to the transverse location of the vehicle path of travel.

4.2 Static and Dynamic Bridge Response -- A Review of Time-History Data

4.2.1 General

The FHWA test vehicle was used in studies on the Salt Fork River and the Shaffer Creek bridges. The former was studied almost exclusively for the effect of the test vehicle, whereas for the latter both truck traffic and the test vehicle were considered. Thus the test vehicle serves in effect to "calibrate" the bridges in the sense of giving information on expected response levels under a known vehicle. The objective of this section is to introduce the effect of the significant parameters and comment on problems associated with predicting the bridge response with available analysis methods. The emphasis is on the characteristics of individual response time histories measured under controlled conditions.
A basic difficulty in interpreting structural behavior is the determination of stiffness properties of the longitudinal girders under load. Although the test structures are of noncomposite design, that is without shear connections, a common mode of behavior under light loads is that of full composite action because of the friction present between the girders and the slab. In the case of the Shaffer Creek bridge, a large proportion of the loadings were found to be sufficiently large so that as to cause at least some degree of breakdown in composite action. No measurements of relative slip between the beams and the slab were made.

For the Shaffer Creek bridge the effect of variable composite action is seen also in the occurrence of residual deflection and strains at the end of the vehicle crossings. This phenomenon is to be expected since the structural behavior during and after the vehicle crossing is nonlinear, influenced by the frictional interacting force between the beam and slab.

Two approaches are used to assess the influence of partial composite action. The most straightforward of these was to attempt to locate the neutral axis by study of strains on the top and bottom flanges of the bridge beams. Alternatively by indirect means the degree of composite action could be deduced by comparisons between field results and theory. For example, a static analysis which treated the bridge structure as a gridwork of longitudinal and transverse beams was used; properties were assigned to these beams assuming some predetermined degree of composite action. By repeated trial and error modifications of section properties one could estimate the degree of composite action
by comparing measured and predicted deflection. A similar analysis, using a finite-element model of the structure which could incorporate different degrees of composite action, was made by Eberhardt (9).

4.2.2 Static or Crawl Response

To illustrate the static behavior of the bridges under the action of the test vehicle typical crawl or influence curves are presented. These crawl curves resemble the influence line under a unit load, but are, of course, a composite curve superimposing the effects of the three axles of the test vehicle. Data for Salt Fork River and Shaffer Creek bridges will be used to show:

(1) The general shape of the crawl history including the relative maximum and minimum values;
(2) The transverse distribution of strains and deflections, and
(3) The influence of the degree of composite action.

An understanding of the nature of the crawl response is important in interpreting the dynamic bridge response. The total dynamic response can be taken as consisting of two components, the static or crawl history plus a dynamic increment. It will be seen that the crawl component dominates because the increment due to dynamic effects which is superimposed on the crawl response is usually relatively small, on the order of 15 to 30 percent for the bridges studied. Only when a pavement discontinuity or rough surface conditions are present can one expect significantly higher dynamic responses. Note that it is
reasonable to exclude for highway bridges moving oscillating loads or unbalanced forces which could produce extremely large dynamic response.

The static results also form the basis for developing simplified formulas for influence surfaces which will be incorporated in a probabilistic theory for the prediction of stress histories. This latter aspect of the problem will be discussed in Chapter 5.

Salt Fork River Bridge

The Salt Fork River bridge, a three-span continuous structure, was tested under controlled loadings with FHWA test vehicle; representative crawl curves are presented in Figs. 4.1 through 4.5. Strains and deflections are presented for locations in both side spans and the center of the center span, i.e., Sections A, C and B, respectively, as shown in Fig. 2.1. Selected results are presented for negative moments over the first interior pier and at the coverplate cutoff in the span nearest the entrance. The peak ordinates for these curves are summarized in Table 4.1. Data is presented for three loading conditions, denoted by lane 1, lane 2 and lane 3. As noted previously, lane 1 denotes eastbound travel in normal traffic lane, lane 2 denotes travel of the vehicle centered on the center line of the structure, i.e. straddling the traffic centerline markings, and lane 3 denotes westbound travel in normal traffic lane. The structure carries two-way traffic on a lightly traveled two-lane highway. The bridge is a five beam structure so that beams 1 and 5 are farthest removed from the load and beam 3 is centered under the load when traveling in lane 2.
For the symmetrical loading conditions, (lane 2) strains and deflections at the center of the center span shown in Figs. 4.1 and 4.2. The basic features of the crawl response are seen in these figures and may be summarized as follows:

(a) The crawl curves for deflection are regular in shape and do not contain cusps. This is an expected result based on knowledge of the structural behavior of the system even on the basis of an idealization of the bridge as a single beam.

(b) The curves for strains in the beams close to the load path contain cusp which mark the instant of passage of the heavy axles over the section. These cusps are quite distinct. On the other hand the cusps are absent from beams 1 and 5, the beams farthest from the load path.

(c) The distribution of strains in the beams across the bridge at any instant is nonuniform, but is essentially symmetric consistent with a symmetric loading condition; when the loading condition is nonsymmetric the transverse distributions reflect this lack of symmetry. A quantitative indication of the nature of the transverse distribution of strains and deflections will be presented.

(d) There is evidence of a residual strain or deflection; this is felt to be the result of breakdown in composite action. Evidence and a discussion of this point will be presented subsequently herein and in greater detail in the Final Report.

In Fig. 4.3 the top flange strains are presented which correspond to the bottom flange strains presented in Fig. 4.2. The
fact that the strain response in the top flange is negligible under low loads (that is, when the vehicle is not near the center of the bridge) but increases when the vehicle is near the center of the center span, is an indication of a change in the degree of composite action. The strains in the top flange must increase if the neutral axis moves downward; such a change would be consistent with a decrease or breakdown in the friction forces between the slab and the beam flange. This effect is more clearly shown in the data for the Shaffer Creek bridge. Also, upper flange crawl curves do show a sharp influence of the individual axles with a cusp at the individual points.

Strain response due to the unsymmetrical loading conditions is illustrated in Fig. 4.4. Bottom flange strains at Section A are presented. As before, the general shape of the curves is consistent to what would be expected from the classical influence line. Beams 3 and 4 which are close to the path of travel of the vehicle show cusps corresponding to the passage of each axle. The cusps are evident in the curve for Beam 2 but absent for curves in Beams 1 and 5; this fact is attributed to the remoteness of the load in the case of beam 1, but for beam 5 is seen to be a feature of the influence surface for the structure. The transverse distribution of the maximum strain levels across the structure is non-symmetrical, consistent with the loading condition.
Shaffer Creek Bridge

Many of the general comments on the qualitative nature of the crawl curves for the Salt Fork River bridge apply also to the selected crawl curves for the Shaffer Creek bridge presented in Figs. 4.6 through 4.8. It will be recalled that the Shaffer Creek bridge is divided by a longitudinal joint into a five-beam and a four-beam portion which are free to move independently. The four-beam portion carries the passing lane and because of the traffic pattern at the location carries a relatively small number of heavy vehicles. The traffic at this location is influenced by the fact that the most of the heavy trucks are still accelerating after leaving the weight station; passing maneuvers are not common.

Runs with the FHWA test vehicle were made on three paths of travel, all eastbound, designated lane 1, 2 or 3. Lane 1 denotes vehicle travel centered on beam 3, the center beam of the five-beam structure; lane 2 denotes vehicle travel centered on the normal travel lane; and, lane 3 denotes a path of travels such that one line of wheels passes over beam 5 and the other travels on the four-beam portion of the structure.

Selected strain crawl curves are shown in Figs. 4.6, 4.7 and 4.9, for loading on lanes 1, 2, and 3; deflections are shown (Fig. 4.8) only for load on lane 2. In general a familiar pattern is seen; the shape is consistent with the appropriate influence line for a two-span beam. In Fig. 4.6 cusps appear in the response curves for beams close
to the load, but not for beams 1 and 5 which are remote from the load. Although the loading is symmetric the structure is not because of the curb over beam 1 which stiffens the structure and "attracts" moment. This results in higher strains in beam 1 than in beam 5.

The vehicle in lane (2), produces a nonsymmetrical loading and consequently nonsymmetrical strain histories; strain crawl curves for this case are shown in Fig. 4.7. Herein the transverse distribution of strains is much less uniform than for lane 1, but the shape for the heavily loaded beams is as before. The beam remote from the load again show no evidence of cusps; of course the strain levels in beam 1 are extremely low.

Finally, a set of crawl strain histories for the vehicle in lane 3 is shown in Fig. 4.9. Consistent with the extreme eccentric loading conditions, only beam 5 exhibits cusps corresponding to the individual axles. The nonuniformity of strains in the various beams is noticeable and in fact, the sign of the strain response in beam 1 is reversed - compression in beam 1 when beam 5 has reached maximum value in tension.

Deflection crawl histories for vehicle travel in lane 2 are shown in Fig. 4.8; these correspond to the strains which were presented in Fig. 4.7. As expected the deflections are more uniform than strains that do not exhibit distinct cusps. The distribution of maximum deflections is nonuniform and the largest deflections occur in beam 5. Perhaps the most significant feature of the deflection crawl
history is the residual value after the vehicle leaves the bridge (i.e. at the point where the curve plot is discontinued). This residual is attributed to the nonlinear behavior arising from the breakdown in composite action.

Transverse Distribution of Maximum Static Response

The transverse distribution of maximum strains and deflections in the Salt Fork River bridge at Sections A and B is shown in Fig. 4.10. The wheel positions corresponding to three travel lanes used in loading the bridge are shown at the top of the figure. Maximum values for both strains and deflections are plotted versus transverse position; the data points are for strain or deflection in the beams, and the lines connecting these points are drawn to aid interpretation and are not to be considered as representative of intermediate values of slab strain or deflection.

The observations made in the previous section are confirmed by the results in Fig. 4.10. The strains or deflections at both Sections A and B are distributed approximately symmetrically for loading in lane 2. There is some evidence of lack of symmetry in that both deflections and strains in beams 4 and 5 are slightly larger than corresponding values for beam 1 and 2. This is attributed to some lack of symmetry in the behavior of the bridge, perhaps in the breakdown in composite action, since the structure itself is apparently symmetrical. This lack of symmetry is seen also at Section B that the response for load in lane 1 is not a mirror image of the response in lane 3. It will be
recalled that lane 1 and lane 3 loadings are vehicle paths in the normal lanes of traffic and are thus equal antisymmetric loadings which should correspond.

When the load is in either lane 1 or lane 3, it is seen that the distributions of both deflections and bottom flange strains are highly nonuniform with maximum deflection and strain occurring usually in beam 4, close to the resultant of the loading. The general level of magnitude of the response can be seen from the values given in Table 4.1; maximum bottom flange strains are on the order of 130 microinches per inch when the load is in lane 3. This strain corresponds to a live load stress on the order of approximately 4,000 psi \((28,000 \text{kN/m}^2)\).

With the data it is possible to estimate the effect of a second vehicle on the structure. Specifically one can combine the values tabulated for load in lane 1 and load in lane 3 to simulate the effect of side-by-side placement of two vehicles on the bridge. The results of this combination are shown as a dashed line in the plot for Section B in Fig. 4.10; the maximum strain when two trucks are on the bridge is 190 microinches per inch in beam 4. This is an increase of 60 microinches per inch above the maximum strain for a single vehicle. Thus the increase in strain due to the placement of the second vehicle on the bridge is the order of 46 percent. Again for the combination of load in lanes 1 and 2 the lack of symmetry in the combination is evident. However, in general the distribution of
strains is more uniform than in the case of the single vehicle. This discussion has considered only the side-by-side placement of two vehicles; additional vehicles placed longitudinally would decrease the maximum response because the span length is short enough to result in an unfavorable placement for maximum effect.

The sum of the maximum strain for all five beams at a given section is proportional to the total moment at that section, neglecting the longitudinal moment carried by flexure in the slab. Thus the sum of maximum strains should remain constant regardless of vehicle transverse position. This is confirmed by the data in Table 4.1; for example at section B the sums of the five maximum beam strains are 374, 375, and 382 microinches per inch for lanes 1, 2, and 3, respectively. The difference in these sums is on the order of only 2 percent.

Plots of the distribution of maximum bottom flange strain and deflection at midspan of the west span of Shaffer Creek bridge are presented in Fig. 4.11. The positions of the wheel loads for the various lanes are shown in the top figure; the second wheel load for lane 3 which is on the adjacent four-beam bridge is not shown. Deflections are distributed nearly symmetrically about beam 3 when the truck is in lane 1, with the exception that beam 1 has a somewhat lower deflection than beam 5. These differences are expected since the curb over beam 1 increases its composite moment of inertia and stiffness. The bottom flange strains for the vehicle in lane 1 are distributed in approximately the same manner as the deflections with the exception that beam 1 has
a higher strain than beam 5. Loading in lane 2 produces a highly nonuniform distribution of deflections and strains with beam 1 being virtually unstressed by loads in the normal traffic lane. The non-uniformity in transverse distribution is greatest for loading in lane 3 where only one line of wheels loads is placed on beam 5. For lane 3 the maximum strains, which are produced by only one half the vehicle acting on the bridge, are approximately the same as for load in lane 2 where the entire vehicle is on the bridge. The eccentricity of lane 3 loading is great enough so that bottom flange strains in beam 1 in compression rather than tension.

The sum of the maximum strains for all five beams for the Shaffer Creek bridge shows again that the total strain is again nearly a constant regardless of vehicle position, i.e. either lane 1 or lane 2, when the entire truck is on the bridge; the sum of maximum bottom flange strains are 334 and 349 \( \mu \text{in.}/\text{in.} \) for lanes 1 and 2, respectively. The sum of maximum strains for load on lane 3, 206 \( \mu \text{in.}/\text{in.} \) is not equal to one half for sums for lanes 1 and 2. This result implies that there is a difference in breakdown in composite action for the various beams under the two loading conditions and thus superposition does not hold for total moments.

4.2.3 Dynamic Response

The dynamic behavior of the Salt Fork River and Shaffer Creek bridges will be illustrated using selected time histories of strain and deflection for crossings by the FHWA test vehicle. In the case of
Shaffer Creek bridge, response curves for selected heavy trucks will be used to demonstrate their relation to the response induced by the test vehicle and to give an indication of the general effects of traffic from the time history point of view as background for discussion of the statistical data on stress histories presented in Section 4.4.

Analytical results are available for three-span and two-span continuous bridges similar to Salt Fork River and Shaffer Creek. The general features of the measured response are familiar and consistent with these analyses. It was not the purpose of this investigation to develop a detailed correlation with theory for the bridge-vehicle response or to investigate in detail the effects of the bridge-vehicle parameters. From experience with the FHWA test vehicle, many of the parameters are difficult to measure and introduce into the analysis; roadway roughness and vehicle suspension characteristics are good examples. As has been seen in the discussion of static response, further complications are introduced as a non-uniform breakdown in composite action occurs. Breakdown in composite action is more complex to handle in the dynamic case than in the static. A brief comment on correlation with theory has been made in Ref. 9. The study of analytical results for the dynamic response has been aided by the consideration of the so-called dynamic increment history (4, 5). This quantity is defined as the difference between the ordinate to the dynamic response curve and the ordinate to the corresponding static influence or "crawl" curve.
The general appearance of the dynamic increment history (DI) is greatly influenced by the vehicle speed. The effect of vehicle speed is conveniently interpreted using the non-dimensional speed parameter 
\[ \alpha = \frac{vT_b}{2L} \] where \( v \) is the vehicle speed, \( T_b \) is the fundamental bridge period and \( L \) is the span length. The parameter \( \alpha \) has two direct influences on the appearance of the DI curve: (1) The characteristic amplitude varies in direct proportion to the value of \( \alpha \), and (2) the wavelength of the oscillations in the curve varies directly with \( \alpha \). Thus, to predict the occurrence of a combination of a positive increment and a positive maximum of a static influence line, a simple procedure can be developed. If one nearly sinusoidal component predominates in the DI, the location of the maximum can be written directly as a function of \( \alpha \). Since the location of the peak in the static influence line is known, the critical value of \( \alpha \) for a maximum response can be computed directly since it is a measure of the wavelength of the DI. The critical \( \alpha \) value varies over a relatively small range. Also, the amplitude of the waves changes with \( \alpha \). Within a typical range of speeds, say forty to seventy mph, there is usually only one or perhaps two combinations possible for a maximum; in some cases it may not be possible for relative maximum to occur and the largest response occurs at crawl or very low speed.

The above discussion will also serve to explain the shape of the spectrum curve of maximum total response versus vehicle speed. It will be seen that these spectra are undulatory in appearance and
the relative peaks and valleys represent combinations of maximum positive or negative increments with the peak static effects.

Other problem parameters, notably vertical oscillations of the vehicle as it enters the bridge and surface roughness on the bridge deck can greatly affect the dynamic increment history in wave form and amplitude. Hence the simple speed (a) influence on the variation in response maxima just given must be reinterpreted; however a degree of dependence on speed has been found (4, 5) when roughness and initial oscillation are important.

In contrast to an analytical approach, dynamic increment history curves are difficult to generate from field measurements because of the uncertainty in specifying a consistent crawl curve. Of prime importance are (1) a consistent transverse position of the vehicle in both dynamic and static cases, (2) enough reference points in time, that is, vehicle location or reference marks, so that the two curves are comparable on the abscissa, and (3) the same degree of composite action present during both static and dynamic crossings of the vehicle. Also, one must assume that a uniform vehicle speed is maintained in both static and dynamic situations. Where uncertainties exist one must adjust the relative positions of the abscissas of the two curves until a reasonable appearance is obtained for the resulting dynamic increment history. These problems in the determination of dynamic increment histories were recognized in the computer-based data analysis programs and adjustment procedures are provided.
The synthesis of a dynamic increment history curve is illustrated in Fig. 4.12 for midspan deflections in the Salt Fork River bridge under the FHWA test vehicle. Four time-histories are shown. The first, part (a), is the crawl curve. Part (b) is the corresponding dynamic history for a 60 mph crossing. The overall shape of the dynamic history is similar to crawl curve with an additional dynamic component having an oscillatory character added. In part (c) the difference or increment [curve (b) - curve (a)] has been plotted in absolute scale, that is, deflection in inches. Finally, in part (d) the dynamic increments have been normalized with respect to the maximum static deflection, that is, the maximum ordinate of curve (a). From curves (c) and (d) it is seen that the dynamic increment is oscillatory in nature; in the early portions of the history, until the vehicle reaches about midspan, the positive and negative peaks are approximately equal. In the last third of the history there is an upward shift in the entire curve. This shift is characteristic of the experimentally derived dynamic increments and is due to inconsistent static and dynamic influence curves. The nondimensional amplitude of the oscillations is approximately 0.10. In general, the normalized dynamic increment can be related to the speed parameter \( \alpha \), which for the particular test run shown had a value of 0.15. Nieto (5) showed that approximate formulas could be developed in terms of \( \alpha \) and that the response amplitude ranged from approximately \( \alpha/2 \) to \( \alpha \) for the normalized dynamic increment. The normalized quantity has meaning only for the
most heavily loaded beam where the static ordinate is nonzero and a maximum. A similar set of results are shown for strains at midspan in Fig. 4.13, but for a vehicle speed of 37.5 mph. The overall behavior is much the same as for deflections except that there are more oscillations as a consequence of the longer time required for the vehicle to cross the bridge.

A dynamic increment history for Shaffer Creek is shown in Fig. 4.14 for deflection at midspan of the entrance span, beam 4, FHWA test vehicle crossing in lane 3 at 21 mph. This figure is similar in format to Figs. 4.12 and 4.13 except that the normalized increment history is not included.

Two features should be noted. First, the dynamic increment history contains two predominant frequencies, a higher frequency component corresponding to the bridge fundamental and which appears throughout the history but is most apparent in the last third and a lower frequency component appearing in the first two-thirds of the history. The latter corresponds to the fundamental frequency of vibration of the test vehicle. Second, the last third of the DI history is displaced above the zero amplitude line because of the variability in composite action. However, the resulting DI history is still useful for studying the frequency content of the response and the waveform near the point of maximum total response.

In view of the uncertainties in computing and interpreting the dynamic increment history curves, the remaining discussion will emphasize time history curves for total response.
A series of time-history curves are shown in Figs. 4.15 through 4.21 for the Salt Fork River bridge and in Figs. 4.22 through 4.20 for the Shaffer Creek bridge. These are presented to show the general transverse distribution response under dynamic conditions, the effect of vehicle speed and the general waveform of the response curves. Results for the Salt Fork bridge will be shown for both Sections A and B. For the Shaffer Creek bridge deflections and strains are presented for the center of the first span only. Results will be mainly for test vehicle crossings centered in the normal traffic lane. For Shaffer Creek bridge selected additional response histories are presented illustrating effects due to two vehicles selected from traffic; namely, a 3S-2 tractor-semitrailer and a 3S-2 tractor-semitrailer dump truck.

Considering first the response of the Salt Fork River bridge, Figs. 4.15 and 4.16, 4.17 and 4.18, and 4.19 and 4.20 contain response histories for deflection and strain paired respectively for vehicle speeds of 21, 39 and 60 mph (9.5, 17.5 and 26.7 m/s). In each instance the FHWA test vehicle was operating eastbound in lane 2. In general, for each speed the pattern for both strain and deflections is similar. The response curves are dominated by frequency content which is that of the fundamental period of the bridge. The overall shape of the curve is, as demonstrated previously, determined by the crawl response and, consequently, the transverse distribution of dynamic effect is substantially the same as in the static case. It appears for both strains and deflections that the oscillations of the bridge for all beams are substantially in phase. As would be expected from the
previous discussion the effect of increasing speed is to decrease the number of oscillations in the history during the time the vehicle is on the bridge.

In Fig. 4.21 deflection histories are shown for beam 3 the Salt Fork River bridge at Section A (which is at the 0.42-point in the entrance span). Again the FHWA test vehicle is operating eastbound in lane 2. The first, third, and fourth curves shown in this figure correspond to the response at Section B discussed above. Only the response for one beam has been shown, but various speed values are included. The effect of speed on the response histories is essentially the same for this section as for the center of the center span. Evident in Fig. 4.21 is a residual upward deflection at the instant the vehicle leaves the span. The magnitude of the residual deflection is nearly the same as in the crawl case.

Response data for the FHWA test vehicle crossing the Shaffer Creek bridge is shown in Figs. 4.22 through 4.24 for Section A, midspan of the entrance span, with the vehicle operating in lane 2. Figures 4.22 and 4.23 are for deflection and strain respectively for vehicle speed of 32 mph (14 m/s). Figure 4.24 is for strains with the vehicle crossing at 54 mph (24 m/s). The general observations concerning these results are similar to those made for both the Salt Fork River bridge; however, the influence of the fundamental mode is somewhat less prominent. The transverse distribution of dynamic response is again quite similar to that seen for the crawl case. Considering
deflections of beam 1 which are of interest in the response of pedestrian user of the bridge, there is almost no static or crawl component and only a small oscillation corresponding to the fundamental frequency of the bridge. The maximum deflections and strains experienced by the bridge occur in beam 5; it is seen that the FHWA test vehicle, which simulates the AASHO design vehicle, induces strains on the order of 180 μin./in.

In Figs. 4.25 through 4.28 selected deflection and strain histories for all five beams of the Shaffer Creek bridge at Section A are presented for two typical vehicles selected from the traffic. Figures 4.25 and 4.26 represent response for a typical 3S-2 tractor, semi-trailer combination weighing 72.5 kips (32,800kg) traveling at 46 mph (21m/s) in lane 2. In Fig. 4.27 and 4.28 strains are shown for all beams at Section A for a 3S-2 dump truck weighing 70.1 kips (31,800kg) traveling at 46 mph in lane 2. The 3S-2 tractor-trailer combination is longer than either the test truck or the 3S-2 dump truck. Given the relatively short span of Shaffer Creek, the influence of the increase in wheelbase is pronounced. In Fig. 4.25 a relative minimum in strains occurs about the first third of the response history so that two distinct and nearly equal peaks in the strain response are evident for the longer truck. The waveform reflects the influence of the tandem axles instead of the single axles on the drive axle and trailer axles. For strains and deflection in both cases, the fundamental mode is more strongly excited in beam 1; hence a pedestrian user of the bridge would find the crossing of either the semi-trailer
combination or the dump truck more noticeable* than the FHWA test vehicle. In contrast to the effect of the longer semi-trailer, Fig. 4.26, it is seen that the dump truck, Fig. 4.28, which has a much shorter trailer, produces only a single major positive peak or half-wave in the history. The 3S-2 dump truck is a compact load and was found to be the most severe loading encountered during the test program on the Shaffer Creek bridge. This will be confirmed by the data presented in Section 4.4. The deflections induced by the dump truck are quite pronounced in beam 1. As a general observation it would appear that the fundamental mode is somewhat more strongly excited by the 3S-2 dump truck than by the 3S-2 tractor trailer combination; both vehicles are traveling at the same speed and have nearly the same gross vehicle weight of approximately 70 kips (32,000kg).

The variation in composite action and the location of the neutral axis for the Shaffer Creek bridge has been noted. Data is shown in Figs. 4.29 and 4.30 for the time variation of the position of the neutral axis at Section A of beams 3 and 4 during the crossing of a 3S-2 tractor-trailer combination. The vehicle is one of the heaviest encountered, a permit overload weighing 79.1 kips (35,800kg), traveling at 44 mph (20m/s) in lane 2. Each figure contains at the top time histories of top flange strain, as a dashed line, and bottom flange strain as a solid line; below is a plot of the position of neutral axis versus time.

*This is not intended to imply that the level of bridge vibration would be objectionable.
(position of the vehicle). The portions of this curve corresponding to small values of top and bottom flange strain are unreliable and are ignored. Thus the portion of the abscissa between approximately 0.2 and 0.7 is relevant to this discussion. The positions of the neutral axis corresponding to full composite action and non-composite action are indicated by the upper and lower horizontal dashed lines, respectively. For both beams 3 and 4 it is seen that the neutral axis location initially corresponds to essentially composite action; as the strains become larger as the vehicle crosses the bridge the position of the neutral axis drops until it is approximately at that of fully non-composite behavior. This migration of the neutral axis from the fully composite to the noncomposite location is irregular in time. There is no consistent position of the neutral axis for the free vibration portion of the record. Of course this portion of the record also corresponds to extremely small strains which are not reliable.

The discussion thus far has been concerned with the overall behavior of the bridge structure which is related to the gross properties of the vehicle. On the other hand, the behavior of the slab is primarily influenced by the properties of individual axles. To investigate slab behavior strain gages were placed on bottom transverse reinforcing bars at slab midspan at a location approximately 5 ft. (1.53m) from the entrance. This was done for all test structures. Significant strain levels were obtained only for the Salt Fork River bridge. This was due to two factors: (1) only the Salt Fork River
bridge had a relatively wide beam spacing of 7 ft-6 in. (2.29m) and (2) only for this structure was the test vehicle placed so as to have a line of shell loads at midspan of the slab between beams. Normal traffic on the bridge would not induce such large strains.

For the Salt Fork River bridge, strains were measured on five adjacent transverse reinforcing bars located in a region beginning at a point 5.4 ft. (1.65m) from the entrance of the bridge at the center of the slab span between beams 2 and 3. After chipping away approximately 1.25 in. (30mm) of concrete cover to expose the bars it was found that these reinforcing bars, consistent with the original plans, were alternately 1/2 in. (13mm) square and 3/4 in. (19mm) round bars on 5 in. (130mm) centers. In Figs. 4.31 and 4.32 strain response is illustrated for a gage (1502) on a 3/4 in. diameter round bar, it is the second in the sequence of five bars and was located approximately 5 ft. - in. (1.78m) from the entrance of the bridge.

In Fig. 4.31 a time history of strain response for reinforcing bar strain is shown for a short portion of the record; the remaining portion is of little interest because significant strains occur only when the vehicle is in the immediate vicinity of the gage. The time history shown is for a vehicle speed of 40 mph (19 m/s). The response is characterized by three sharp peaks corresponding to the passage of each of the three axles of the test vehicle over the gage. For reference these peaks are labeled A, B, and C and they correspond to the steering axle, the tractor drive axle, and the trailer axle, respectively. The strain response is large. For this crossing maximum peaks for axles B and C were approximately 500 \(\mu\text{in.}/\text{in.}\) or thus a live load stress of approximately 15,000 psi (104,000 kN/m\(^2\)).
The relative size of the peaks A, B, and C and the magnitude of the total strain varied with vehicle speed. This is illustrated in Fig. 4.32 where the peak strain values identified separately as A, B, and C are plotted versus vehicle speed. There is a dependence upon speed with relative maxima which occur at 20 mph (9 m/s) and between 50 and 60 mph (20 and 30 m/s). The maximum dynamic effect is at 20 mph; this was found to be true for all five reinforcing bar locations considered in the study. Maximum amplification of strain corresponding to peak C is approximately 1.5. At a higher speed of 50 to 55 mph (20 - 22 m/s) which is more characteristic of heavy truck traffic, the amplification was about 1.3.

Although similar data for reinforcing bar strains are available for the Shaffer Creek bridge, the strain levels are by no means as high as those discussed above; the maximum crawl strains are on the order of 75 μin./in. or a live load stress of approximately 2,000 psi (14,000 kN/m²). This difference is due largely to the fact that in the normal traffic patterns in the test series, and for the crawl and speed test runs of the FHWA test vehicle, the wheel paths were not close to midspan of the slab. Also, the difference in slab span length directly affects the stresses induced in the reinforcing. Finally, the wheel loads experienced by the slab under the action of the test vehicle as significantly higher, basically by a factor of two, because of the use of single axle instead of tandems on the FHWA test vehicle. A legal vehicle of maximum GVW usually would be a five axle combination having tandems on the tractor and trailer. However, the possibility remains
that in design combinations of beam spacing, slab thicknesses, and a
path of travel could lead to relatively high live load stresses in the
reinforcement.

4.3 Characteristics of Heavy Truck Traffic

The procedures for gathering the vehicle data presented in
this section were described in Article 2.4 and 3.2. Included is data
taken during the three field programs. The data taken in 1969 is
presented in the form of histograms of axle types, gross vehicle
weight, individual axle weights, axle spacings and vehicle arrival
intervals. For Shaffer Creek 1968, only histograms of gross vehicle
weight and individual axle weights are presented. The characteristics
of the vehicles are similar in the three data gathering periods.

The designations used for naming vehicle types and identifying
the individual axles is shown in Fig. 4.33, and has been used extensively
in various investigations of heavy truck traffic characteristics. Of
particular interest from the design point of view are vehicles type
2S-1 and 3S-2 which are the common semi-trailer combinations. Vehicles
in these classes are the basic prototypes corresponding to the AASHO
design vehicle.

Figures 4.34 and 4.35 show the axle type and distributions
for Shaffer Creek and CB&Q in 1969. In both, the preponderance of
the 3S-2 axle configuration should be noted. These percentages are in
reasonable agreement with vehicle data gathered by the Illinois Division
of Highways. These surveys have also shown that the 3S-2 vehicle
occurs in traffic with a frequency increasing from a minuscule number in 1950 to present percentages on the order of 60 to 70 percent, typically.

Gross vehicle weight histograms for all trucks are shown in Figs. 4.36 through 4.38. It should be noted that all of the data shows that gross vehicle weight distributions tend to be bi-modal with peaks at approximately 30,000 lbs. (13600 kg) and 70,000 lbs. (31700 kg). The 1969 data for Shaffer Creek has the highest proportion of lightly loaded vehicles to heavily loaded vehicles. This is perhaps explained by the location of the Shaffer Creek bridge on the incomplete by-pass (I-280) which has not been connected to carry major through traffic but rather carries a substantial amount of local traffic in the Quad-Cities, Illinois, area. Also, it is suspected, although not verified by data, that the time of day at which records are taken would affect this distribution. Perhaps the heavily loaded vehicles leaving the warehouses and factories in the area, e.g. for the Chicago market, would be passing the bridge location either in the very late hours in the evening or very early morning hours. Conversations with the weight station personnel confirmed this, but no quantitative data were obtained.

Conversely, the CB&Q bridge has a lower ratio of lightly loaded trucks to heavily loaded trucks as might be expected from the fact that the bridge is on interstate I-80 carrying transcontinental truck traffic, a substantial portion of which one would expect to be heavily loaded. The relative proportion of heavy and light vehicles in the 1968 data for Shaffer Creek is affected by a substantial number of 3S-2 gravel trucks which were in operation during the test period carrying materials to interstate construction east of the bridge.
Histograms of gross vehicle weight for only the 3S-2 trucks are shown in Fig. 4.39 and 4.40. Again the bi-modal nature of the distribution is observed and the two maxima correspond approximately to an empty and a fully loaded condition, respectively.

Figures 4.41 and 4.42 are axle weight histograms including all axles of all vehicles. Again the larger ratio of lightly loaded axles to heavily loaded axles in the 1969 Shaffer Creek data is apparent. Figures 4.43 and 4.44 include all axles of only 3S-2 vehicles and when compared with Fig. 4.41 and 4.42 illustrate again the strong influence of the 3S-2 truck traffic on the data.

Histograms of individual axle weights for all trucks shown in Figs. 4.45 through 4.59. The axle weights show either a uni-modal or a bi-modal distribution depending on the axle considered; for a given axle the peaks occur at roughly the same weight level in all three data sets. For the bi-modal histograms there is again a variation in the ratio of peaks corresponding to the lightly loaded axles and to the heavily loaded axles; note the relative sizes of the double peaks for axles B, C, D, and E. This behavior is not as apparent for axle A because the steering axle normally does not carry a substantial portion of the pay load carried by the vehicle and tends to remain more constant. Histograms of individual axle weights for 3S-2 vehicles only are shown in Figs. 4.60 through 4.63 for the CB&Q bridge. Histograms for the 3S-2 trucks differ only in that the features of the distributions are slightly less pronounced and the variance of the data is somewhat larger.
Figures 4.64 through 4.67 are axle spacing histograms for the CB&Q bridge data. The corresponding axle spacing histograms for the 1969 Shaffer Creek data are similar and are not presented. The histograms are characterized by peaks which have relatively simple interpretation. The spacing interval C-D exhibits a bi-modal form. From Fig. 4.35 it can be seen that the data sample includes a noticeable proportion of 2S-2 trucks. Thus, the first peak at the smaller spacing in the histogram corresponds to a trailer tandem axle in the 2S-2 configuration, while the second peak corresponds to the length of the trailer in the 3S-2. The number of vehicles represented by the peak for the smaller axle spacing is essentially equal to the number of 2S-2 vehicles indicated in Fig. 4.35.

Figures 4.68 and 4.69 are vehicle arrival histograms. These represent a plot of the time elapsed between the arrival of consecutive vehicles. This data seems to be typical of that to be expected for a sequence of random arrival events and confirms the common assumption of a Poisson distribution for arrival of vehicles in the traffic stream. However, quantitative tests of goodness of fit do not seem appropriate until more data is available.

One significant item of data is not available for the truck traffic, namely the transverse location of the trucks as they cross the bridge. Rough data on transverse positions taken during the test runs at various speeds indicates that the ability of a driver to control deviations of the vehicle from the center of the traffic lane is nearly normally distributed. Further discussion of this point will be made in Section 4.4.
Summary comments are in order with regard to the data which has been presented. For the dominant vehicle, the 3S-2 tractor-trailer combination, the maximum weights observed correspond closely to the design level used by AASHO, 72,000 lbs. (32,600 kg). The major difference is that the AASHO assumes the total load is carried on a three axle tractor semi-trailer while in actuality it is always carried on a five axle unit. In terms of the overall behavior of the bridge, on the basis of even simple static analysis, for all but very short spans, the difference between placing the two 32,000 lbs. (142 kN) loads on single axles or on a tandem axles with a spacing of about 4 ft. (1.2m), is small. However, from the point of view of the behavior of the deck slab, the concentration of a 32,000 lb load on a single axle would be severe compared to the tandem axle. From the data presented it would seem easy to define a standard or statistically based 3S-2 vehicle for design.

Finally, it should be noted that there are very few overweight vehicles represented in the data. During the entire study only two illegal vehicles were encountered. A very small number of legal permit vehicles with loads as high as about 90,000 lbs. (41,000kg) were encountered during the various test periods. The maximum loads measured do correspond quite closely to the Illinois legal limits; quite logically, then, truckers tend to load their vehicles to the maximum permissible loads. It certainly would be worthwhile investigating methods to get more complete data on the upper extremes of the GVW
distribution -- that is, on the expected illegal loads. The presence of a weight station, operated for law enforcement, eliminates substantially all possibilities of extreme overweight vehicles.

4.4 Stress History Data for Selected Truck Traffic

The purpose of this section is to introduce a sample of data which has been collected on the stresses (strains) induced in the bridges under normal heavy truck traffic. This is the heart of the investigation and represents the starting point for the development of theory and empirical formulas for predicting stress histories induced by traffic. This discussion will indicate the motivation for analyses and techniques which will be used to interpret the data in the Final Report.

Many of the standard statistical techniques for developing and testing hypotheses for the relationship between variables are not needed for the present study since deterministic analyses have given a good indication of the parameters of the problem and their influence on the bridge response. However, it is useful and instructive to use simple statistical analyses such as linear regression, two parameter multiple regression and to adopt the expected value or mean and variance as the basic descriptors of the data. Although histograms for strain are useful, it is far more important to have information on the mean and variance expressed as standard deviation or coefficient of variation. In Chapter 5 a scheme for predicting mean and variance of the strain or stress is discussed.
Of the parameters of the problem, intuitively gross vehicle weight is the most significant. Other factors such as roadway roughness and speed are known to be important from the point of view of dynamics. The transverse position of the vehicle is important from the point of view of the elemental structural behavior of the bridge system. But clearly, a light passenger car does not induce anywhere near the level of strain as a loaded heavy semi-trailer dump truck. Thus one starting point in the study of the strain-range data, even for a small number of selected runs, is to plot maximum strain range versus the corresponding gross vehicle weights of the trucks. Such plots are presented in Figs. 4.70 and 4.71 for maximum strain range in beams 4 and 5 respectively for the Shaffer Creek bridge in the 1968 test. These plots are for a 140 selected traffic runs and test vehicle crossings. This sampling was deliberately chosen to contrast the effects of test vehicle, selected truck traffic and also a small group of heavily loaded 3S-2 semi-trailer dump trucks. A histogram of truck type for this group is shown in Fig. 4.72 and the histogram for gross vehicle weight is shown in Fig. 4.73. The group of 3S-2 dump trucks is of interest because it represents a severe loading (as was seen in Section 4.2) and also represented a small population which crossed the bridge many times with consistency in type and speed.

The gross vehicle weight histogram, Fig. 4.73, is greatly influenced by the test truck and 3S-2 dump trucks which appear as the dominant spikes above 70,000 lbs. (32,000kg). However, if these
are ignored, the same pattern is seen as in the presentation in Section 4.3, namely, a group of 35-2 vehicles which are essentially empty weighing between 20,000 to 30,000 lbs. (9,000 to 14,000kg), a number of fully loaded vehicles with GVW above about 65,000 lbs. (29,000kg), and a scattering of partially loaded vehicles over the middle weight range. Of course the shape of the histograms is influenced by the fact that only a small sample of data is considered.

In studying the results in Figs. 4.70 and 4.71 it is obvious that the strain range does increase nearly linearly with gross vehicle weight over the range of gross weights from about 65,000 lbs. (29,000kg). The wide scattering of data in the region between approximately 65,000 and 80,000 lbs. (29,000kg and 36,000kg) represents the influence of the special vehicles just noted. For both beams 4 and 5 it is seen that the group of gravel trucks produced the largest strains of all vehicles with one exception including the most severe run by the test truck itself. The test truck in fact has a slightly larger GVW than the gravel trucks. The array of data points along a vertical line at 74,000 lbs. (33,500kg) represents the test truck runs and these are further subdivided by different data point designations into runs for lanes 1, 2, and 3 respectively. The effect of changing path of travel is as would be expected; beam 4 experiences the greatest strains when traffic is in lane 2 or closest to being centered over the beam. As would be expected from the results of the discussion of static response, loading in lane 1 is not severe. Finally, when the load is lane 3
there is only one line of wheels acting on the bridge (over beam 5) and the strain distribution to beam 4 is greatly reduced. For beam 5 lane 2 produces the largest strains but now lane 3 yields strains on the same order but in the lower bracket of the strains induced with the traffic truck operating in lane 2. Lane 1 loading is least severe for beam 5 since is the path most removed from the beam.

The results presented in Fig. 4.70 and 4.71 suggest several interpretations. First there is clearly a relationship between strain range and gross vehicle weight and this can be investigated for the various types of trucks in the traffic stream by a linear regression analysis. The use of linear regression is not novel to the present project and has been used by several investigators.

One factor demonstrated by these curves is the influence of transverse position of the vehicle. In the normal flow of traffic usually vehicles do not vary in path of travel with large deviations; this is shown in Fig. 4.74. The distribution of deviations is essentially Gaussian with a near zero mean, that is, an expected travel at the center of the lane.

Some variability in the strains could be induced by differences in speed and vehicle length. As an example, the variability in the strains induced by the 35-2 dump trucks can be related in part to variation in truck wheelbase. This is demonstrated in Fig. 4.75 where maximum strain range is plotted as a function of length. There is a general trend of decreasing strain with increasing length; as expected, spreading the total load decreases the magnitude of induced
moments and thus strains. A large variability remains which must be accounted for in the effect of other parameters.

From the above and discussion in Section 4.2 it has been seen that beam strains are sensitive to the transverse position of the vehicle. This of course may be confirmed by the development of an influence surface for the structure, which when appropriately combined with various wheel loads representing the vehicle will yield a distribution of moments or strains in the structure. For a small variation in transverse position of the vehicle, a relatively simple approximate influence function might be suitable. While this will not be attempted herein, it is instructive to look at the percent of the total strain across a bridge section carried by a specific beam for a set of traffic runs. In Fig. 4.76 the histogram for the percent of total strain in beam 5 at Section A for traffic moving in lane 2 is presented. These data are again for the selected group of trucks for Shaffer Creek, 1968. The resulting histogram is regular in form; this result is to be expected since the distribution in transverse position of the vehicle was seen to be one which was nearly normal Gaussian and when combined with what would be a fairly regular influence function should indeed yield a nearly normal variability in the distribution of percent of total strain. The mean percentage of the total strain distributed to beam 5 is seen to be 34 percent. The corresponding standard deviation is 4.6 percent.

It is interesting to note that the expected average percentage of strain or moment taken by beam 5 as determined from the crude average
of a number of dynamic tests does not differ appreciably from the percentage or moment taken under static conditions as was seen in Section 4.2. The variability associated with this result, as reflected in the standard deviation, is a consequence of dynamic factors which modify in part the way in which the moment is distributed to the beams.

In Figs. 4.77 and 4.78 the strain range is again plotted as a function of gross vehicle weight, but the traffic runs only are included. Superimposed on these results are the best fit or linear regression line for the mean value of strain as a function of weight; this is shown as a solid line. The dashed lines represent strains at a level one standard deviation above or below the mean regression line. Also indicated on the figures are the values of the correlation coefficient \( \rho \) which is approximately 0.9 for each. The latter value confirms statistically the high degree of correlation between GVW and strain range.

But obviously there is considerable scatter in the results; a number of data points fall outside a band of plus or minus one standard deviation. Other parameters associated with this data, spacing S-23 of the tractor axle and the trailer axle ranged between 5 to 40 ft. (1.5 to 12 m) and vehicle speed ranged between 40 to 65 mph (19 to 31 m/s). Although test vehicle runs have been excluded so that there is not the broad scattering of data seen in Fig. 4.70 and 4.71 the influence of the gravel trucks can be seen with the cluster of data points at about 70,000 lbs. GVW. This regression line should not be used for extrapolation of this data since it is obviously greatly influenced by a particular group of vehicles.
In using the linear regression plot of strain versus gross vehicle weight, it should be emphasized that the line which has been fitted to this data should be thought of as representing the expected or mean value of strain as a function of gross vehicle weight; depending upon other factors such as wheelbase, speed and vehicle initial conditions, the variability or scatter in the data at any given level of gross vehicle weight may vary and be quite large. This variability and the slope of the line depends also on the beam considered as can be seen from comparing Fig. 4.77 and 4.78. Of course, the slope of the line would be different for a different geometry and type of bridge.

Up to this point, the discussion of strains induced in the bridge has been related to specific parameters of the problem, notably gross vehicle weight. Rarely, however, should specific vehicle weights in the traffic be of direct concern to the designer, particularly with reference to design against fatigue. Even for research in fatigue, a simple histogram or probability density function describing the relative likelihood of the occurrence of different levels of strain or stress is of prime importance. To indicate the general nature of such histograms, information on strain undifferentiated with respect to weight or other variables will be presented. Two types of data are available for a presentation. The most straightforward is the histogram for maximum strain range for the desired population of truck crossings for each beam; one strain range value is obtained for each crossing event. The shape of the histogram will be influenced
by the counting interval, that is, the strain interval, used. The frequency ordinate may be scaled either in percent of events occurring in the interval or in the form of density such that the total area under the histogram is unity. When presented in the latter form the ordinates are scaled so that the histogram can be compared directly with a theoretical probability density function.

The other form of strain data which yields histograms related to the total strain range histogram is based on what have been termed partial strain ranges. The term partial strain range is introduced to denote those changes in strain level between relative maxima and minima occurring in the time history and which correspond also to reversal in sign of the strain. The counting computer program assumes that as a record starts a sign reversal occurs; this assures that the first major excursion in the strain history will be recorded in the partial strain range count. To illustrate, for the time history shown in Fig. 4.28, for beam 5, one gets a partial strain range count as the strain goes from zero at the origin to the first positive maximum, another count for the range between the maximum positive to the maximum negative and then another count for the maximum negative to the first point in the free vibration era where it goes to a positive value. Thus the partial strain range analysis conveys information on reversals when a number of 'waves' in the history occur. The strain range analysis considers, only a single excursion from positive to negative for each truck crossing in the count. The counts
in the strain intervals generated in these two approaches of course differ depending on the beam location and the type of bridge (i.e. shape of the influence line) and generalization is difficult. It can be seen from static results that the partial strain range will have always three counts for a two-span continuous bridge, provided that the counting process is terminated at the time the vehicle leaves the bridge. Thus, a decision must be made concerning at what amplitude free vibration oscillations are to be counted. The counting programs used have a strain tolerance level within which such oscillations are ignored. In general this has been set such that oscillations in the free vibration era do not appear in the counts.

A third alternative, not presented here, is to use a complete partial strain range count in which the excursions between consecutive relative minimum and maximum points are counted; thus, every "hill" and "valley" is in the count and its presence is represented. This latter scheme was developed with a view towards being able to reconstruct, to some degree, the actual time histories. Since the utility of this information is not apparent yet, this approach has not been developed.

Histograms are shown in Figs. 4.79 through 4.83 for strain range in the five beams of the Shaffer Creek bridge for the selected 222 traffic runs taken in 1968. This data corresponds to that used to develop the linear regression plots, Figs. 4.77 and 4.78. Also indicated on the figures are values of mean and standard deviation for the histograms. For convenience these are tabulated below:
<table>
<thead>
<tr>
<th>Beam</th>
<th>No. of Vehicles</th>
<th>Mean Strain</th>
<th>Std. Deviation</th>
<th>Coeff. of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>236</td>
<td>26.4</td>
<td>12.3</td>
<td>0.46</td>
</tr>
<tr>
<td>2</td>
<td>230</td>
<td>41.8</td>
<td>21.6</td>
<td>0.52</td>
</tr>
<tr>
<td>3</td>
<td>239</td>
<td>79.3</td>
<td>43.0</td>
<td>0.54</td>
</tr>
<tr>
<td>4</td>
<td>241</td>
<td>110.6</td>
<td>56.6</td>
<td>0.51</td>
</tr>
<tr>
<td>5</td>
<td>236</td>
<td>129.0</td>
<td>66.4</td>
<td>0.51</td>
</tr>
</tbody>
</table>

From the table above it is seen that the mean values of strain for these histograms have approximately the same transverse variation as the static strain response due to test vehicle seen in Section 4.2; that is the largest mean strain occurs in beam 5, the smallest mean strain in beam 1 and the distribution between these points roughly of the same shape as the static case. The latter result is not surprising since one would expect the static behavior of the bridge to dominate the dynamic response. The traffic is largely in the normal traffic lane of the bridge with relatively small deviations from this path. Also, since the bridge has a longitudinal joint, traffic in the passing lane does not induce strain in the five beam portion of the bridge. For all except beam 1, there is evidence of a bi-modal appearance to the histograms. For example, in the results for gage 125 on beam 5 there is a relatively large peak centering on about 90 μin./in. and a lower peak at 240 to 280 μin./in. This bi-modal shape is of course consistent with that seen for the GVW histogram. This suggests that it will be possible to make a reasonable simulation of at least the static component of these strain responses by
considering directly the population of trucks, provided that the gross vehicle weight and wheelbase data are known. This latter possibility is being explored and will be presented in the Final Report.

Also of interest are the highest strains experienced. It is seen that strains do reach levels between 280 and 300 $\text{mil}./\text{in.}$ for beam 5, that is, stress levels on the order of 7000 to 9000 psi (48,000 to 62,000 $\text{kN}/\text{m}^2$). Again it should be remembered that the frequency of high stress levels is influenced by the presence of the group of 3S-2 dump trucks.

Concerning the statistical descriptions tabulated for these results, it is interesting to note the consistency in the coefficient of variation, i.e., the ratio of standard deviation to the mean. For all beams it ranges between 46 and 54 percent. Statistically these represent a considerable variability in data and one problem in the final interpretation of the results will be to explore ways of predicting this variability. It can be attributed in large part to the variability in the gross vehicle weight; also the effects of variability axle spacing and speed can be investigated.

In Figs. 4.84 through 4.87 histograms are presented for the partial strain ranges for beams 2 through 5. These histograms are related to those just discussed, with the important difference that they are dominated by the large number of counts for lower strain values. There are for each crossing event at least two additional counts and sometimes many more corresponding to oscillations in the trace which induce reversals. For the most heavily loaded beams,
beams 4 and 5, it is seen that the coefficient of variation becomes large, greater than 70 percent. This greater variability arises because more features in the time history are taken into account and influence the partial strain range count. The mean values for these histograms are of course significantly lower than the corresponding means for the total strain ranges (Figs. 4.80 through 4.83). The shape of the partial strain range histograms suggest that they might be represented by the log-normal probability density function. To illustrate this, log-normal density functions for the indicated mean and standard deviations have been superimposed on the histograms.

The results presented in this chapter suggest ways in which theory may be developed to aid in interpreting and predicting the expected strains induced by traffic. The influence of the static component of response is clear. The relationship between the strain ranges and the gross weight of the vehicle is both expected and confirmed by data which has been presented. Effects of speed, transverse position and wheelbase are represented in the variability of the data. In Chapter 5 some approaches to the problem of prediction will be explored; these draw their motivation in part from the observations which have been made concerning the data presented in this chapter.
5. ON DEVELOPMENT OF STRESS HISTORY THEORY - A PROBABILISTIC APPROACH

5.1 General

The present study is one of the several research efforts needed to develop predictions of fatigue life expectancy of highway bridges, as noted in Chapter 1. In this chapter the nature of the problems associated with prediction will be discussed and ways in which research results can be interpreted and summarized will be suggested so that more effective use can be made of the data.

At the outset, it appears essential to adopt probabilistic approaches to the analysis of fatigue behavior rather than a deterministic approach often specified for bridge design. The development of fatigue analysis is not within the scope of the present investigation, but this topic is receiving considerable attention elsewhere including laboratory study and analyses drawn from work in metallurgy, fracture analysis, and structural reliability and safety. Research on probabilistic approaches to design is on-going and several examples will be mentioned. Major problems lie in the area of defining a fatigue damage mechanism, specifying a satisfactory critical level of damage, an acceptable risk level or probability of occurrence of that damage, and consistent with these predicting the probable life of the structure. An acceptable damage level may mean damage which can be repaired at reasonable cost - provided that the damage (e.g. subcritical cracks or flaws) can be
detected by a reasonable inspection scheme. The expected life or number of cycles of loading to a specified damage level is of increasing practical importance in highway bridge designs; recent experience has shown that the highway traffic volume and specifically heavy truck traffic may greatly exceed expected levels early in the design lifetimes of highway bridges and pavements. Of course, the basic design capacity of the highway system may have been exceeded as well.

Critical to the prediction of life expectancy is the hypothesis or theory used to describe and quantify cumulative damage to the structure or its elements when subjected to many stress events of random amplitude. In practice, the definition of a stress event is difficult; for the present we consider an event to be the major stress-time cycle associated with each vehicle or each set of multiple-vehicles crossing the bridge at the same time. The damage arising from the superimposed small amplitude stress oscillations resulting from the dynamic response of the bridge is at present uncertain; it may be of minor significance. The effect of the dynamic component on the total stress range, however, is significant.

Data on the fatigue life of metal structural elements, in particular as needed for the verification of theory, must be drawn from laboratory studies. In contrast, for example, to the aircraft industry, where full scale tests of prototype structures are feasible, the civil structural engineer normally can test only specimens of the parent structural material and selected structural members or details.
Thus, the fatigue life of full-scale structures must be inferred from these small scale laboratory tests. Much of the work on fatigue behavior of structural materials, mainly steel, and structural elements and details has been conducted using cyclic loadings of constant amplitude.

A number of approaches are in common use for simulating the actual loads encountered in the life of a structure: (1) Constant cycle fatigue tests summarized in the form of S-N curves and interpreted by means of a cumulative damage theory. (2) Tests with block loadings having a prescribed number of load levels arranged in either a specified or random order. (3) Tests with loadings corresponding in a statistically valid way to an actual random process. These approaches are discussed at length in an excellent state-of-the-art paper by Swanson (10).

It should be noted that much variable or random fatigue testing assumes a zero-mean Gaussian narrow-band amplitude load of relatively high amplitude. In contrast, highway bridge fatigue falls in a relatively unexplored range of material behavior in which there are very large numbers of cycles of loading with non-zero mean reflecting the large influence of dead load stresses. The live loads produce a mean stress range is relatively low; the probability density function for stress range is skewed such that the frequency of high stresses is low.

A large number of parameters affect fatigue life of structures subjected to random loadings. Fatigue behavior is a random phenomenon
even for repeated stresses of constant amplitude. One must then adopt a probabilistic approach to the survival of an element when damage accumulates under repeated cycles of either constant or random loading.

The strategy adopted by Schilling et al (11) in the NCHRP Program 12-12 to establish stress spectra for use in the laboratory involved study of the available field data from various sources on three types of bridges under traffic. This data was then represented by a probability density function which would be the basis for design of the experimental load pattern. Two probability density functions were investigated, the Rayleigh and the Erlang. The Rayleigh distribution was selected. A key problem was determining upper and lower cutoff stress levels. A minimum stress level may be assigned to the distribution -- that is, shifting the probability density function away from a zero origin. Based on this Rayleigh model, a random sequence of half-cycle stress applications was synthesized. That is, the individual load amplitudes to satisfy one of a family of Rayleigh probability density curves, but are arranged in a random sequence, were determined. The spectra are then output in the form of a control tape which is cycled continuously throughout the duration of the fatigue tests.

Researchers reporting data on stress histories in highway bridges, see for example Refs. 12 and 13, have made use of the Miner hypothesis for estimating cumulative fatigue damage. Consistent with the Miner damage model, a random parameter describing damage level
may be defined (14). The parameters of this formulation are random variables and analysis of the expected life, considering the finite probability of failure or damage, may be made -- provided that the needed input data is known.

Thus, from the point of view of developing a life expectancy theory, the results of field investigations should be interpreted from a probabilistic point of view; expected or mean stress levels, variance of the stress level and ultimately a guide for the rational selection of a probability density function for stresses critical locations in the structure should be the result of the studies. Laboratory tests require the definition of realistic stress-blocks or random load programs, and basically it is the results of field investigations and extrapolations which will be used to determine appropriate laboratory loading conditions until such time as a theory for predicting traffic patterns and loads on bridges has been developed and verified.

5.2 Dynamic Bridge Response - Deterministic Analysis and Simulation

The response of bridges to moving vehicles or moving concentrated loads has been studied extensively and it is not the purpose of this discussion to summarize these studies beyond the mention made in Section 1.3, but rather to consider the role of bridge structural analysis in the development of a theory for predicting stress histories. Although it was shown in the AASHO Road Test bridge studies (6, 7) that theoretical response predictions compared very well with measured bridge response, one must view this agreement in
light of the geometry of the AASHO test bridges and the carefully controlled and instrumented vehicles. The AASHO bridges were narrow, one-lane structures, without curbs, which behaved like simple beams, i.e. without significant transverse variations in response. The vehicles were instrumented so that it was possible to make reasonable estimates of the initial conditions of the vehicle as it entered the bridge.

In the context of the AASHO bridge tests and behavior, a theory which idealizes the bridge as a beam was useful in explaining the sensitivity of the bridge response to the various parameters. With this same theory it would be possible to simulate the effect of a traffic stream on one of these bridges using multiple computer solutions. A major difficulty will be encountered in determining input parameters for the bridge-vehicle system for each vehicle crossing. No explicit relationships between maximum or average response and factors such as vehicle motions, speed, and roughness exist; at least several computer solutions would be required to choose a representative solution for each vehicle crossing. Alternatively, a Monte Carlo scheme could be used to randomize the input parameters. Direct computer simulation, even for a simple bridge and a modest number of vehicle combinations, is a formidable task.

Some of the difficulties in determining input parameters could be avoided by a more extended analysis of the roadway, bridge and vehicle system. If an appropriate profile for the approach pavement and deck can
be specified, then the initial conditions of the vehicle could be determined by direct computation. The remaining problem parameters are less difficult to evaluate or estimate - except for the current mechanical condition of the vehicle suspension system. This plan of attack has at least two drawbacks: (1) Computer cost is greatly increased since solution times must be longer to cover a reasonable duration of vehicle behavior on the approach pavement; for a short-span bridge this might double or triple the solution time. (2) The roadway roughness function to be used in the simulation remains uncertain. Little data is available which is useful except in the case of specific test bridges. The studies conducted on vehicle response to pavement roughness which are intended to define the dynamic character of axle loads shed light on this difficulty; see for example the work by General Motors in NCHRP Project 15-5, Ref. 15.

If instead of the beam idealization, the problem of the three-dimensional, multigirder, continuous bridge is considered, the structural analysis is complicated and the problems cited above are multiplied. There is a significant expansion in the number of parameters to be evaluated. Furthermore, a computer program which treats all of the parameters of the problem, for example the work by Eberhardt and Walker (9), is too costly to use for direct simulation of a stream of vehicles. Simulation of traffic is complicated by the need to either ignore or to model passing maneuvers on the bridge or multiple vehicles
entering the bridge simultaneously at either the same or different speeds. Further refinements in computers, methods of solution of the existing formulations and the application and probabilistic methods to the input for such a theory can help alleviate some of these difficulties.

What then is the role of deterministic analysis? It has been useful in defining the relative sensitivity of the bridge-vehicle system response to the parameters. It is seen that the dynamic response of the structure can be interpreted as a static component or static influence line upon which is superimposed a dynamic response increment. The static component can be evaluated by relatively simple static solutions or by approximate or empirical influence lines or surfaces determined for the selected locations in the structure. The input parameters for the static analysis depend upon the geometry and dimensions of the bridge and the dimensions and static weight of the vehicle. The static component can be predicted with a reasonable degree of certainty.

The amplitude and waveform of the dynamic component is less certain. However, based upon the experience with deterministic analysis it is seen that it can be approximated by a simple time-function. The amplitude of this function can be treated as random variable, making use of a probabilistic analysis of the available data on the dynamic increment. Since the dynamic increment is superimposed upon a relatively large static component, the effect of the uncertainties in the approximation is moderated somewhat.
5.3 Probabilistic Stress History Interpretation and Prediction

5.3.1 General

An approximate method for predicting statistical descriptors of stresses occurring at critical sections in a bridge subjected to heavy truck traffic is proposed. It might serve as an interim stage in the development of a more complete formulation or modeling of the highway bridge stress history problem. Ideally it would be desirable to include as components of an analytical model:

1. Forecasts of traffic volume and the relative occurrence of heavy vehicles. This would include the classification by truck type.

2. The predicted gross vehicle weight or alternatively the relative likelihood that a particular vehicle or class of vehicles will be empty, full loaded or in some specified, intermediate, conditions of loading.

3. The relative occurrence of truck traffic in the various individual traffic lanes on the structure. From a structural point of view, this problem is ameliorated by the fact that a given longitudinal beam or stringer is affected significantly by no more than say three lanes of traffic and usually by only two.

4. The predicted response of the bridge, that is, the analysis for stresses at a specified critical section given a statistical description of the vehicle loads. The structural theory is well developed for the multi-girder and slab type bridge - at least for the prediction of stresses in main load-carrying members. However, the
analysis of stresses in bracing, attachments and at points of stress concentration may need further development and field study. The loading should be appropriately described for this purpose in (1) through (3) above.

Each aspect of the problem listed above is an appropriate subject for research and in many cases work currently is being conducted; each is related to other problems in traffic, pavement and structural engineering, including bridge impact factors, pavement design, traffic capacity, etc. The prediction may be attacked by making certain assumptions regarding stochastic or statistical models of traffic flow, arrival time, GVW, etc. and a direct analysis for the required stress (or strain). Each step, however, needs verification and intermediate data which is often lacking (e.g. gross vehicle weight correlation for specified traffic lanes).

Field measurement of stress histories (the Illinois program and others) represents a technique of study which integrates many of the above factors; the desired stresses are determined directly but usually cannot be readily extrapolated to other traffic situations and bridges. However, in the present series of tests the availability of vehicle data which are correlated with bridge response time-histories, makes possible an evaluation of some of the needed bridge-vehicle relationships. At first empirical relations will be used, but these should be substantiated by and then replaced by theory in the future.

As an example of the latter statement, it is well known that the distribution of flexural stresses or moments in the beams is
nonuniform across the bridge cross section. This phenomenon has been studied extensively for static loads and to some degree for moving vehicles (dynamic loads). The relative distribution factors for loads in a given lane to a specified longitudinal girder in the bridge can be determined, provided that the bridge parameters, vehicle weight, axle spacing, the transverse location of the loads and the longitudinal positioning for maximum affect are known. But such is not always the case. Thus in the interpretation of the results of the present investigation it will be suggested that an empirical percent distribution of load to a specific beam be used. The distribution factor will be a random variable because of the uncertainties involved, but the mean value of the percent of the moment carried will be consistent with static theory and adequately represented as a Gaussian normal variate.

5.3.2 An Empirical Approach to the Influence of Vehicle Characteristics on Bridge Response

Before describing a simplified mathematical model for bridge response, it should be noted that it is possible to explore the effect of vehicle characteristics on bridge response by empirical means. Sufficient data has been gathered in the course of the investigation, to investigate the significant parameters of the problem including the transverse position of the vehicle, gross vehicle weight, vehicle length and critical axle spacing, and vehicle speed. Several tools are available for such an analysis: Sorting of the data into subsets according to critical parameters, (2) linear regression analysis of the subgroupings or (3) multi-regression analysis of the data.
A weakness in this approach is obvious -- it is not possible to obtain a statistically independent subsets of data in which all variables are constant with the exception of the variable under study. It is seen that there are certain vehicle lengths and weight characteristics which are definitely not independent and a definite correlation between these parameters over certain ranges and classes exists. For example, there is a high frequency of occurrence of short, fully loaded 3S-2 dump trucks and where these are included in a particular data set they greatly influence the analysis. Also obvious is the fact that truck speed is partially correlated with the gross vehicle weight, particularly when a grade or passing maneuver occurs before entrance to the bridge. Transverse position is an important factor in determining the response of each beam; however, statistically it has very small effect on the bridge response because of the relatively rare occurrence of vehicles in the passing lane during the course of the study. It has been shown and noted previously that vehicle speed is an important parameter in the dynamic analysis; it appears explicitly in the theoretical formulations. However the results of the preliminary analysis of the data shows that speed does not have a great influence because the range of speeds occurring in the free-flowing truck traffic at the test bridges is relatively small, i.e. between 40 and 70 mph with a mean of about 55 mph.

The use of empirical methods to investigate the influence of truck characteristics will be presented and discussed in detail in the final report and has been a major sub-topic in the dissertation work of Ruhl.
Without the use of a mathematical model it is possible, however to draw some conclusions with regard to the statistical variability of the data and the general consistency of the various measurements. More specifically it is seen that the level of mean stress or strain occurring in the various bridges is consistent with static predictions although the question of composite action in the bridge must be resolved before precise prediction of the mean level of stress induced can be made. Analysis of the histograms for variance shows that the standard deviation or coefficient of variation for strain induced in the bridge is on the same order as the coefficient of variation for the gross vehicle weight. This result is consistent with the fact that the gross vehicle weight dominates as a parameter of the problem. Linear regression analyses show very high correlations of bridge response and gross vehicle weight. Studies of axle spacing between heavy axles or tandems show that the effect of this parameter is consistent with the predicted influence of axle spacing on the static maximum response. Data and discussion on these points as well as the limitations of the empirical approach, particularly as it is constrained by the small number of events falling in certain subgroupings, will be presented in the Final Report.

It has been seen that it is possible to draw conclusions based on the experimental evidence concerning the influence of the various parameters, but that this is information which cannot be readily extrapolated directly to other traffic situations and bridge
types. A more useful approach is to develop a simplified mathematical model which may be relatively crude from the point of view of structural mechanics, but yet will allow for a rational analysis of the uncertainty involved from a statistical point of view. The discussion of such a model is presented in the following section.

5.3.3 A Simplified Mathematical Model

Explicit or implicit relationships exist between stress induced in a bridge element and the parameters of the bridge-vehicle system; but, to be useful these must be written as mathematical expressions (if such exist) or approximated in some way and interpreted as functions of random variables. For each location specified on the structure and for the time period during which the truck traffic parameters are known, one can determine:

(1) A measure of response of the structure -- expressed in terms of the mean strain or stress ($\sigma$) and the corresponding variance or dispersion, expressed as the standard deviation or coefficient of variation (c.o.v.), and

(2) A probability density function based on the mean and variance defined above, which has an acceptable "goodness-of-fit".

Methods of statistical inference and analysis of uncertainty of functions of random variables can be brought to bear on the problem -- perhaps, however, requiring suitable linearizations or simplifications of the functions describing the influence of the problem parameters. This approach, of course, should be verified by comparisons with field results and theory.
As a starting point for such analysis, based on present knowledge of the behavior of the bridge and the discussion of Chapter IV, the following form for a rational, simplified mathematical model for the strains (or stresses) induced in the main load carrying members of the multi-girder and slab bridge is proposed,

\[ \varepsilon = K_1 \text{GVW} \ \xi_x \ \eta_y \ (1 + C_\gamma + K_2 \sigma_\nu) \]  

(5.1)

Wherein the following notation is defined:

1. \( K_1 \) is a scale factor relating the maximum strain to the gross load; for example for a simple beam with a single-concentrated load \( K_1 = \frac{L}{4} \frac{c}{EI} \). This parameter includes the effect of bridge type, bridge geometry, and beam characteristics in a multi-beam bridge.

2. \( \xi_x \) is a static influence function for a specified location in the bridge. It is a function of longitudinal position of the vehicle and the spacing of the vehicle axles so as to produce a relative maximum.

3. \( \eta_y \) is a static influence function, consistent with \( \xi_x \), and is a function of the transverse position of the vehicle (i.e., for a beam at a specified transverse location).

4. \( \text{GVW} \) is the gross weight of the vehicle. Obviously, strain, stress or moment induced in the bridge is not always a function of the gross weight. For example
for a very short span or a deck slab or floor system, GVW would be rationally replaced by the weight of the individual axle. However, this can be resolved by retaining GVW in the formulation and modifying $K_1$ and/or $\delta_x$ when the influence of a single axle or wheel dominates.

\[ C_v + K_2 \alpha_v \]

is an impact factor expanded to account separately for two dynamic effects. Specifically, $C_v$ is a random variable expressing the influence of the vehicle initial motion and roadway roughness. The constant $K_2$ relates the speed parameter to the level of impact induced in the bridge; both vary with bridge type. And, $\alpha_v$ is the speed parameter which appears explicitly in the analytical formulation of the problem.

Of the parameters included in Eq. (5.1), wheel spacing, transverse position, vehicle speed, and gross vehicle weight are variables of the present study for which data are available. The other factors in Eq. (5.1) may be computed as characteristics of the bridge considering structural theory, or are parameters which are basically uncertain even with field measurements. For example, $C_v$ and $K_2$ must be evaluated on the basis of past analytical experience and engineering judgment. Selection of these parameters, however, can be made more rational by expressing them with a measure of uncertainty, that is, in terms of their means and coefficients of variation. This approach has been investigated by Ang, et al (16), particularly in the
design context. Basically a systematic means for the analysis of the uncertainty using statistical measures is used. This same approach will prove useful in evaluating the overall significance of Eq. (5.1).

The mathematical model of strain history, Eq. 5.1, can be rewritten in a compact form $\varepsilon = g(x_1)$ where $x_i$ are the random parameters of the problem. The expression may be linearized using a one-term Taylor expansion to facilitate study of the variance; this yields:

$$
\frac{S^2}{\varepsilon} = \sum \left( \left( \frac{\partial g}{\partial x_i} \right) \right)^2 \left. \right|_{x_i = x_i} S^2_{x_i} \right\} \quad (5.2)
$$

where $\frac{S^2}{\varepsilon}$ is the variance for strain. Note that the partial derivative is to be evaluated at a value of $x_i = \bar{x}_i$ where the $\bar{x}_i$ are the respective mean values of the variables $x_i$; $S^2_{x_i}$ represents the variance of the variable $x_i$. The significance of this approximation of variance is that even with imperfect data and with crude estimates of the uncertainty in some of the parameters one can still make a rational estimate of the mean and variance of the strain. This latter step of course fulfills the first goal of the discussion in this chapter.

The second goal was to provide a method for determining a suitable probability density function. It has been seen from the preliminary results presented in Chapter 4 that a lognormal distribution has some merit particularly in the case of the partial strain range data.
However the lognormal density function has the disadvantage of predicting a finite likelihood of occurrence of high strains. As noted, Schilling has found the truncated Rayleigh distribution suitable as a model for stress spectra particularly for loadings to be used in fatigue where it is undesirable to include high stresses in the simulation. The computer programs used in the data study can provide information for a goodness-of-fit test although this has not been undertaken at the present time. The problem yet to be resolved is the modeling of the bi-modal nature of the histograms. It has been suggested by Garson et al (17) that the sum of normal distributions might be useful for GVW.
6. SUMMARY

The scope, physical arrangements, data acquisition and reduction systems and selected results for Project IHR-85, Dynamic Stress in Highway Bridges, an investigation of stress histories occurring in highway bridges under truck traffic loadings, have been described. The report has emphasized the test arrangements, bridges, instrumentation, and developmental work associated with the computer based data acquisition and interpretation system. A selected data has been presented and discussed to give an indication of the general nature of the bridge response and the truck traffic observed in the course of the investigation. The results will be presented, discussed and interpreted in detail in the final report of the investigation.

Three bridges were tested during the course of the investigation. These include two three-span continuous structures, the Salt Fork River and CB&Q bridges, respectively, and a two-span continuous structure, the Shaffer Creek bridge. The Salt Fork River bridge was located on a lightly traveled highway and was subject to controlled tests using the FHWA test vehicle. The other two structures were located on Interstate routes and were chosen for their locations near state weighing stations so that data could be collected on heavy truck traffic characteristics when stress measurements were made.

A comprehensive, computer based, data acquisition, analysis and interpretation system has evolved during the course of the investigation. The digitizing of field analog records is controlled using logical information added to the records during field testing. Problems in sorting
and checking of the digitized data prior to loading into magnetic
tape or disk file for computer use, the interpretative computer software
system with a variety of subroutines to perform various manipulations
of data for analysis and interpretation are described. Development
of entire data system comprised a major part of the research effort.

The preliminary results are presented for approximately 200
heavy vehicles at the Shaffer Creek bridge. Typical time histories
for the Salt Fork River Bridge are used to show general characteristics
of the bridge strain and deflection response. In general, strains
or stresses induced in the bridge are low. Mean stresses are on the
order of 2,000 to 3,000 psi; maximum stress levels seldom exceed
approximately 8,000 psi in the case of the Shaffer Creek bridge. A
number of significant parameters are identified and investigated
including gross vehicle weight, wheelbase, vehicle speed and transverse
position of vehicle. Of the parameters of the problem, gross vehicle
weight dominates. Linear regression techniques for analyzing its
influence have been illustrated.

A preliminary discussion of the problems associated with
the development of a stress history theory is presented. Emphasis
is placed on need for a probabilistic approach. It is seen that it
will be a basic goal of the study to seek methods to predict the
expected or mean stress level associated with the stress history
environment and the variance or coefficient of variation. Also
important is the determination of an appropriate mathematical model
for the stress history, that is, a probability density function to be associated with the traffic induced stresses (histogram) at specified critical locations in the bridge structure.

This report is not intended to provide final conclusions regarding the results of the study but is to provide basic information on the nature and scope of the field testing, a preliminary view of the results obtained and a outline of the data acquisition and interpretation methods which have been used. The Final Report will include a detailed interpretation of the stress history results, along the lines outlined herein.
REFERENCES


<table>
<thead>
<tr>
<th>Bridge Designation</th>
<th>Bridge Type</th>
<th>Girders</th>
<th>Deck</th>
<th>Other Design Information</th>
<th>Test Duration</th>
<th>Load Events Recorded</th>
<th>Dynamic Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Salt Fork River</td>
<td>3 span-Continuous</td>
<td>75'-11&quot;; 96'-6&quot;; 75'-11&quot;</td>
<td>7&quot; thick R.C. 32'-4&quot; out to out; 8&quot;x40' curbs</td>
<td>0° Skew Non-composite Design - 1939 H-20 Loading Roller Supports</td>
<td>6/6/67 to 6/20/67</td>
<td>130 test truck crossings</td>
<td>3.1 3.3 2.1 0.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>12 truck traffic crossings</td>
<td></td>
</tr>
<tr>
<td>Shaffer Creek</td>
<td>2 span-Continuous</td>
<td>43'-0&quot;; 43'-0&quot;</td>
<td>7&quot; thick R.C. 43'-4&quot; out to out; 9&quot;x2'-10&quot; South Curb; 9'x1'-10&quot; North Curb</td>
<td>0° Skew Non-composite Design - 1958 H20-S16-44 and modified</td>
<td>7/9/68 to 7/22/68</td>
<td>First Series: 77 Test Truck crossings 302 Truck Traffic crossings Second Series: 905 Truck Traffic Crossings</td>
<td>7.8 8.2 5.2 1.6</td>
</tr>
<tr>
<td>CB and Q R.R.</td>
<td>3 span-Continuous</td>
<td>50'-6&quot;; 76'-3&quot;; 50'-6&quot;</td>
<td>7&quot; thick R.C. 36'-0&quot; out to out; 9'x3'-0&quot; curbs</td>
<td>0° Skew non-composite design - 1961 H20-S16-44 and Alt.</td>
<td>11/4/69 to 11/14/69</td>
<td>1497 Truck Traffic Crossings</td>
<td>5.2 5.5 3.1 1.4</td>
</tr>
</tbody>
</table>

* Bridges designated by geographical feature crossed; remaining information as given on Name Plate

(25.4xin. = mm; 0.305xft. = m)

C = Composite  
NC = Non-Composite
<table>
<thead>
<tr>
<th>Axle</th>
<th>Tire</th>
<th>Contact Area Dimension (in.)</th>
<th>Area (in.$^2$)</th>
<th>Axle Load, lbs.</th>
<th>Pressure, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tractor</td>
<td>Steering Axle</td>
<td>Right</td>
<td>10 1/8 x 9</td>
<td>91</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Left</td>
<td>8 1/8 x 9</td>
<td>73</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Drive Axle</td>
<td>Right Outside</td>
<td>10 1/2 x 9 1/8</td>
<td>96</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Right Inside</td>
<td>10 1/2 x 9 1/4</td>
<td>97</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Left Outside</td>
<td>9 1/2 x 9 1/8</td>
<td>87</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Left Inside</td>
<td>9 1/4 x 9 1/8</td>
<td>84</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Tractor</td>
<td>Subtotal</td>
<td>164</td>
<td>9,400</td>
<td>57</td>
</tr>
<tr>
<td></td>
<td>Trailer</td>
<td>Subtotal</td>
<td>364</td>
<td>34,690</td>
<td>95</td>
</tr>
<tr>
<td></td>
<td>Rear Axle</td>
<td>Right Outside</td>
<td>10 1/2 x 9 1/4</td>
<td>97</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Right Inside</td>
<td>10 5/8 x 9 3/8</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Left Outside</td>
<td>10 1/8 x 9 1/8</td>
<td>91</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Left Inside</td>
<td>10 7/8 x 9 1/4</td>
<td>101</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Subtotal</td>
<td>389</td>
<td>31,590</td>
<td>81</td>
</tr>
<tr>
<td></td>
<td></td>
<td>TOTALS</td>
<td>917</td>
<td>75,680</td>
<td>83</td>
</tr>
</tbody>
</table>

(25.4 x in. = mm; 0.000646 x in.$^2$ = m.$^2$; 6.0 x psi = kN/m.$^2$; 1 lb x 4.45 = N)
<table>
<thead>
<tr>
<th>Gage Number (See Fig. 2.2)</th>
<th>Location (See Fig. 2.1)</th>
<th>Recording Equipment Used</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>BPR Oscillographs</td>
</tr>
<tr>
<td>101</td>
<td>A</td>
<td>*</td>
</tr>
<tr>
<td>102</td>
<td>A</td>
<td>*</td>
</tr>
<tr>
<td>103</td>
<td>A</td>
<td>*</td>
</tr>
<tr>
<td>104</td>
<td>A</td>
<td>*</td>
</tr>
<tr>
<td>105</td>
<td>A</td>
<td>*</td>
</tr>
<tr>
<td>201</td>
<td>B</td>
<td>*</td>
</tr>
<tr>
<td>202</td>
<td>B</td>
<td>*</td>
</tr>
<tr>
<td>203</td>
<td>B</td>
<td>*</td>
</tr>
<tr>
<td>204</td>
<td>B</td>
<td>*</td>
</tr>
<tr>
<td>301</td>
<td>C</td>
<td>*</td>
</tr>
<tr>
<td>302</td>
<td>C</td>
<td>*</td>
</tr>
<tr>
<td>303</td>
<td>C</td>
<td>*</td>
</tr>
<tr>
<td>304</td>
<td>C</td>
<td>*</td>
</tr>
<tr>
<td>305</td>
<td>C</td>
<td>*</td>
</tr>
<tr>
<td>113</td>
<td>A</td>
<td>*</td>
</tr>
<tr>
<td>131</td>
<td>A</td>
<td>*</td>
</tr>
<tr>
<td>122</td>
<td>A</td>
<td>*</td>
</tr>
<tr>
<td>123</td>
<td>A</td>
<td>*</td>
</tr>
<tr>
<td>124</td>
<td>A</td>
<td>*</td>
</tr>
<tr>
<td>125</td>
<td>A</td>
<td>*</td>
</tr>
</tbody>
</table>
TABLE 2.3  DEFLECTION AND STRAIN GAGE LOCATIONS AND RECORDING EQUIPMENT
SALT FORK RIVER BRIDGE - 1967 (continued)

<table>
<thead>
<tr>
<th>Gage Number (See Fig. 2.2)</th>
<th>Location (See Fig. 2.1)</th>
<th>BPR Oscillographs</th>
<th>BPR Mag. Tape</th>
<th>UI Mag. Tape</th>
<th>UI Direct Write Oscillograph</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>211</td>
<td>B</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>212</td>
<td>B</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>213</td>
<td>B</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>214</td>
<td>B</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>215</td>
<td>B</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>221N</td>
<td>B</td>
<td>*</td>
<td></td>
<td>*</td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>221S</td>
<td>B</td>
<td>*</td>
<td></td>
<td>*</td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>222N</td>
<td>B</td>
<td>*</td>
<td></td>
<td>*</td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>222S</td>
<td>B</td>
<td>*</td>
<td></td>
<td>*</td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>223N</td>
<td>B</td>
<td>*</td>
<td></td>
<td>*</td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>223S</td>
<td>B</td>
<td>*</td>
<td></td>
<td>*</td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>224N</td>
<td>B</td>
<td>*</td>
<td></td>
<td>*</td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>224S</td>
<td>B</td>
<td>*</td>
<td></td>
<td>*</td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>225N</td>
<td>B</td>
<td>*</td>
<td></td>
<td>*</td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>225S</td>
<td>B</td>
<td>*</td>
<td></td>
<td>*</td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>313</td>
<td>C</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>321</td>
<td>C</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>322</td>
<td>C</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>323</td>
<td>C</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>324</td>
<td>C</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>325</td>
<td>C</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>003</td>
<td>D</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>004</td>
<td>D</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>005</td>
<td>D</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>Gage Number</td>
<td>Location (See Fig. 2.2)</td>
<td>BPR Oscillographs</td>
<td>BPR Mag. Tape</td>
<td>UI Mag. Tape</td>
<td>UI Direct Write Oscillograph</td>
<td>Remarks</td>
</tr>
<tr>
<td>-------------</td>
<td>-------------------------</td>
<td>-------------------</td>
<td>---------------</td>
<td>--------------</td>
<td>-------------------------------</td>
<td>---------------</td>
</tr>
<tr>
<td>411</td>
<td>F</td>
<td>*</td>
<td>*</td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>412</td>
<td>F</td>
<td>*</td>
<td>*</td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>413</td>
<td>F</td>
<td>*</td>
<td>*</td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>421</td>
<td>F</td>
<td>*</td>
<td>*</td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>422</td>
<td>F</td>
<td>*</td>
<td>*</td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>423</td>
<td>F</td>
<td>*</td>
<td>*</td>
<td></td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>1501</td>
<td>E</td>
<td>*</td>
<td>*</td>
<td></td>
<td></td>
<td>Rebar Strains</td>
</tr>
<tr>
<td>1502</td>
<td>E</td>
<td>*</td>
<td>*</td>
<td></td>
<td></td>
<td>Rebar Strains</td>
</tr>
<tr>
<td>1503</td>
<td>E</td>
<td>*</td>
<td>*</td>
<td></td>
<td></td>
<td>Rebar Strains</td>
</tr>
<tr>
<td>1504</td>
<td>E</td>
<td>*</td>
<td>*</td>
<td></td>
<td></td>
<td>Rebar Strains</td>
</tr>
<tr>
<td>1505</td>
<td>E</td>
<td>*</td>
<td>*</td>
<td></td>
<td></td>
<td>Rebar Strains</td>
</tr>
<tr>
<td>Gage Number</td>
<td>Location</td>
<td>RECORDED EQUIPMENT USED</td>
<td>Remarks</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-------------</td>
<td>----------</td>
<td>-------------------------</td>
<td>---------</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(See Fig. 2.4)</td>
<td>(See Fig. 2.3)</td>
<td>Oscillographs</td>
<td>U. of I. Mag. Tape</td>
<td>BPR Mag. Tape</td>
<td></td>
<td></td>
</tr>
<tr>
<td>106</td>
<td>A</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>107</td>
<td>A</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>108</td>
<td>A</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>109</td>
<td>A</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>126</td>
<td>A</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>127</td>
<td>A</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>128</td>
<td>A</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>129</td>
<td>A</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>306</td>
<td>B</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>307</td>
<td>B</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>308</td>
<td>B</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>309</td>
<td>B</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>326</td>
<td>B</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>327</td>
<td>B</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>328</td>
<td>B</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>329</td>
<td>B</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>116</td>
<td>A</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>117</td>
<td>A</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>216</td>
<td>C</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>217</td>
<td>C</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>226</td>
<td>C</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>227</td>
<td>C</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table 2.4
DEFLECTION AND STRAIN GAGE LOCATIONS AND RECORDING EQUIPMENT
SHAFFER CREEK BRIDGE - 4-BEAM PORTION - 1968 (Continued)

<table>
<thead>
<tr>
<th>Gage Number (See Fig. 2.4)</th>
<th>Location (See Fig. 2.3)</th>
<th>Oscillographs</th>
<th>U. of I. Mag. Tape</th>
<th>BPR Mag. Tape</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>213</td>
<td>C</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Strain</td>
</tr>
<tr>
<td>214</td>
<td>C</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Strain</td>
</tr>
<tr>
<td>215</td>
<td>C</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Strain</td>
</tr>
<tr>
<td>223</td>
<td>C</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Strain</td>
</tr>
<tr>
<td>224</td>
<td>C</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Strain</td>
</tr>
<tr>
<td>225</td>
<td>C</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Strain</td>
</tr>
<tr>
<td>623</td>
<td>D</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Strain</td>
</tr>
<tr>
<td>723</td>
<td>D</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Strain</td>
</tr>
<tr>
<td>823</td>
<td>D</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Strain</td>
</tr>
<tr>
<td>923</td>
<td>D</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Strain</td>
</tr>
<tr>
<td>624</td>
<td>D</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Strain</td>
</tr>
<tr>
<td>724</td>
<td>D</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Strain</td>
</tr>
<tr>
<td>824</td>
<td>D</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Strain</td>
</tr>
<tr>
<td>924</td>
<td>D</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Strain</td>
</tr>
<tr>
<td>1501</td>
<td>E</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Rebar Strains</td>
</tr>
<tr>
<td>1502</td>
<td>E</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Rebar Strains</td>
</tr>
<tr>
<td>1503</td>
<td>E</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Rebar Strains</td>
</tr>
<tr>
<td>1504</td>
<td>E</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Rebar Strains</td>
</tr>
<tr>
<td>1505</td>
<td>E</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Rebar Strains</td>
</tr>
<tr>
<td>1506</td>
<td>E</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>Rebar Strains</td>
</tr>
<tr>
<td>Gage Number (See Fig. 2.4)</td>
<td>Location (See Fig. 2.4)</td>
<td>Recording Equipment Used</td>
<td>Remarks</td>
<td></td>
<td></td>
</tr>
<tr>
<td>---------------------------</td>
<td>-------------------------</td>
<td>--------------------------</td>
<td>---------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>101</td>
<td>A</td>
<td>*</td>
<td>Deflection</td>
<td></td>
<td></td>
</tr>
<tr>
<td>102</td>
<td>A</td>
<td>*</td>
<td>Deflection</td>
<td></td>
<td></td>
</tr>
<tr>
<td>103</td>
<td>A</td>
<td>*</td>
<td>Deflection</td>
<td></td>
<td></td>
</tr>
<tr>
<td>104</td>
<td>A</td>
<td>*</td>
<td>Deflection</td>
<td></td>
<td></td>
</tr>
<tr>
<td>105</td>
<td>A</td>
<td>*</td>
<td>Deflection</td>
<td></td>
<td></td>
</tr>
<tr>
<td>121</td>
<td>A</td>
<td>*</td>
<td>Strain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>122</td>
<td>A</td>
<td>*</td>
<td>Strain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>123</td>
<td>A</td>
<td>*</td>
<td>Strain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>124</td>
<td>A</td>
<td>*</td>
<td>Strain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>125</td>
<td>A</td>
<td>*</td>
<td>Strain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>301</td>
<td>B</td>
<td>*</td>
<td>Deflection</td>
<td></td>
<td></td>
</tr>
<tr>
<td>302</td>
<td>B</td>
<td>*</td>
<td>Deflection</td>
<td></td>
<td></td>
</tr>
<tr>
<td>303</td>
<td>B</td>
<td>*</td>
<td>Deflection</td>
<td></td>
<td></td>
</tr>
<tr>
<td>304</td>
<td>B</td>
<td>*</td>
<td>Deflection</td>
<td></td>
<td></td>
</tr>
<tr>
<td>305</td>
<td>B</td>
<td>*</td>
<td>Deflection</td>
<td></td>
<td></td>
</tr>
<tr>
<td>321</td>
<td>B</td>
<td>*</td>
<td>Strain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>322</td>
<td>B</td>
<td>*</td>
<td>Strain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>323</td>
<td>B</td>
<td>*</td>
<td>Strain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>324</td>
<td>B</td>
<td>*</td>
<td>Strain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>325</td>
<td>B</td>
<td>*</td>
<td>Strain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>113</td>
<td>A</td>
<td>*</td>
<td>Strain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>114</td>
<td>A</td>
<td>*</td>
<td>Strain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>115</td>
<td>A</td>
<td>*</td>
<td>Strain</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### TABLE 2.6
DEFLECTION AND STRAIN GAGE LOCATIONS
SHAFFER CREEK BRIDGE - 1969

<table>
<thead>
<tr>
<th>Gage Number (See Fig. 2.4)</th>
<th>Location</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>101</td>
<td>All</td>
<td>Deflection</td>
</tr>
<tr>
<td>104</td>
<td>gages</td>
<td>Deflection</td>
</tr>
<tr>
<td>113</td>
<td>at</td>
<td>Strain</td>
</tr>
<tr>
<td>114</td>
<td>mid-span</td>
<td>Strain</td>
</tr>
<tr>
<td>121</td>
<td>span</td>
<td>Strain</td>
</tr>
<tr>
<td>122</td>
<td>of</td>
<td>Strain</td>
</tr>
<tr>
<td>123</td>
<td>west</td>
<td>Strain</td>
</tr>
<tr>
<td>124</td>
<td>span</td>
<td>Strain</td>
</tr>
<tr>
<td>125</td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>127</td>
<td></td>
<td>Strain</td>
</tr>
<tr>
<td>1502</td>
<td></td>
<td>Rebar Strain</td>
</tr>
<tr>
<td>1505</td>
<td></td>
<td>Rebar Strain</td>
</tr>
</tbody>
</table>

All gages recorded on U. of I. 14 channel FM Analog tape recorder.
### TABLE 2.7
**STRAIN GAGE LOCATIONS CB&Q BRIDGE - 1969**

<table>
<thead>
<tr>
<th>Gage Number (See Fig. 2.5)</th>
<th>Location (see Fig. 2.5)</th>
<th>Remarks (Strains Recorded only)</th>
</tr>
</thead>
<tbody>
<tr>
<td>122</td>
<td>B</td>
<td>Bottom flange</td>
</tr>
<tr>
<td>123</td>
<td>B</td>
<td>Bottom flange</td>
</tr>
<tr>
<td>124</td>
<td>B</td>
<td>Bottom flange</td>
</tr>
<tr>
<td>212</td>
<td>A</td>
<td>Top flange</td>
</tr>
<tr>
<td>213</td>
<td>A</td>
<td>Top flange</td>
</tr>
<tr>
<td>221</td>
<td>A</td>
<td>Bottom flange</td>
</tr>
<tr>
<td>222</td>
<td>A</td>
<td>Bottom flange</td>
</tr>
<tr>
<td>223</td>
<td>A</td>
<td>Bottom flange</td>
</tr>
<tr>
<td>224</td>
<td>A</td>
<td>Bottom flange</td>
</tr>
<tr>
<td>225</td>
<td>A</td>
<td>Bottom flange</td>
</tr>
<tr>
<td>002</td>
<td>C</td>
<td>Cover plate cut-off</td>
</tr>
<tr>
<td>003</td>
<td>C</td>
<td>Cover plate cut-off</td>
</tr>
</tbody>
</table>

All gages recorded on U. if I. 14-channel FM analog tape recorder.
<table>
<thead>
<tr>
<th></th>
<th>Deflections, inches</th>
<th>Top Flange Strains, ( \mu \text{in./in.} )</th>
<th>Bottom Flange Strains, ( \mu \text{in./in.} )</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lane</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(25.4 \text{ in.} = \text{mm})
<table>
<thead>
<tr>
<th>Gage Lane</th>
<th>101</th>
<th>102</th>
<th>103</th>
<th>104</th>
<th>105</th>
<th>121</th>
<th>122</th>
<th>123</th>
<th>124</th>
<th>125</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.07</td>
<td>0.13</td>
<td>0.18</td>
<td>0.15</td>
<td>0.12</td>
<td>69</td>
<td>62</td>
<td>81</td>
<td>75</td>
<td>47</td>
</tr>
<tr>
<td>2</td>
<td>0.01</td>
<td>0.07</td>
<td>0.16</td>
<td>0.22</td>
<td>0.30</td>
<td>11</td>
<td>27</td>
<td>56</td>
<td>123</td>
<td>132</td>
</tr>
<tr>
<td>3</td>
<td>0.0</td>
<td>0.01</td>
<td>0.06</td>
<td>0.13</td>
<td>0.26</td>
<td>-13</td>
<td>5</td>
<td>25</td>
<td>60</td>
<td>129</td>
</tr>
</tbody>
</table>
Section A

Section B

Section C

Section D

Section E

Section F

\[ \bullet = 1 \text{ Active Strain Gage} \quad \bullet = 2 \text{ Active Strain Gage} \quad \downarrow = \text{Deflection Measurement} \]

**FIG. 2.2 Instrumentation For Salt Fork River Bridge**
FIG. 2.3 Shaffer Creek Bridge
These Gages Spaced at
4'-6-3/4" / 10-3/4" / 10-3/4" From West Bearing

Section A

From West Bearing

Section B

From West Bearing

Section C

From West Bearing

Section D

1) 4'-5-3/8" West of A
2) 2'-1-3/4" West of A
3) 2'-1-3/4" East of A
4) 4'-3-5/8" East of A

Section E

These Gages Spaced at
4'-7" / 9-3/4" / 9-3/4" From West Bearing

FIG. 2.4 Instrumentation For Shaffer Creek Bridge
FIG. 2.5 CB&Q Bridge
FIG. 2.9 Photograph of FHWA Test Vehicle
(a) Wheel Spacing

(b) Axle Spacing

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Loading</th>
<th>P₁</th>
<th>P₂</th>
<th>P₃</th>
<th>G.V.W.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Salt Fork</td>
<td>No. 1</td>
<td>9.40 k</td>
<td>34.69 k</td>
<td>31.59 k</td>
<td>75.68 k</td>
</tr>
<tr>
<td></td>
<td>No. 2</td>
<td>9.02 k</td>
<td>20.43 k</td>
<td>21.76 k</td>
<td>51.21 k</td>
</tr>
<tr>
<td></td>
<td>No. 3</td>
<td>9.52 k</td>
<td>33.97 k</td>
<td>33.36 k</td>
<td>75.95 k</td>
</tr>
<tr>
<td>Shaffer Creek</td>
<td>All</td>
<td>9.88 k</td>
<td>32.10 k</td>
<td>31.90 k</td>
<td>73.80 k</td>
</tr>
</tbody>
</table>

(c) Gross Weight and Axle Loads, kips

FIG. 2.10 Test Vehicle Configuration and Loads
Vehicle Data
Axle Weights
Gross Vehicle Wt.
Axle Spacing
Time of Arrival
Axle Configuration
Description

Data Punched
On Computer Cards

Tape or Disk Storage of all Data

1. Check Data
2. Supplementary Data Calculations
3. Store

1. Data Calibration
2. Calculation of Speed
3. Generate Time Scale
4. Store

High Speed Digital Computer Processing

Processing of Data
Curve Smoothing

Output of Time Histories
Printed Output
Punch-Card Output
Plotted Output

FIG. 3.1 Flow Diagram For Data Acquisition and Reduction
FIG. 3.2 Sequence of Events as Recorded on Analog Tape in The Field For The Passage
of One Truck
FIG. 4.1 Crawl Curves for Deflection, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck, Lane 2, Eastbound
FIG. 4.2 Crawl Curves for Bottom Flange Strain, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck, Lane 2, Eastbound
FIG. 4.3 Crawl Curves for Top Flange Strain, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck, Lane 2, Eastbound
FIG. 4.4 Crawl Curves for Bottom Flange Strain, All Beams, Section A, Salt Fork River Bridge -- FHWA Test Truck, Lane 3, Eastbound.
FIG. 4.5 Crawl Curves for Bottom Flange Strain, Beams 3, 4, and 5, Section D, Salt Fork River Bridge -- FHWA Test Truck, Lane 2, Eastbound
FIG. 4.6 Crawl Curves For Bottom Flange Strain, All Beams, Section A, Shaffer Creek Bridge, 1968 -- FHWA Test Truck, Lane 1
FIG. 4.7 Crawl Curves For Bottom Flange Strain, All Beams, Section A, Shaffer Creek Bridge, 1968 -- FHWA Test Truck, Lane 2
FIG. 4.8 Crawl Curves For Deflection, All Beams, Section A, Shaffer Creek Bridge, 1968 -- FHWA Test Truck, Lane 2
FIG. 4.9  Crawl Curves For Bottom Flange Strain, All Beams, Section A, Shaffer Creek Bridge, 1968 -- FHWA Test Truck, Lane 3
FIG. 4.10 Transverse Distributions of Maximum Crawl Strains and Deflections, Salt Fork River Bridge, FHWA Test Truck
FIG. 4.11 Transverse Distributions of Maximum Crawl Strains and Deflections, Section A, Shaffer Creek Bridge, 1968, FHWA Test Truck
FIG. 4.12 Crawl, Total Dynamic and Dynamic Increment Histories For Deflection, Salt Fork River Bridge, Center of Center Span, Beam 4, 59.7 mph, Lane 2, East Bound, Alpha = .15
FIG. 4.13 Crawl, Total Dynamic and Dynamic Increment Histories, Salt Fork River Bridge, For Strain, Center of Center Span, Beam 4, 37.5 mph, Lane 3, East Bound, Alpha = .09
FIG. 4.14  Time-Histories for Deflection, Beam 4, Section A. Shaffer Creek Bridge,
-- FHWA Test Truck, Lane 3
FIG. 4.15 Deflection Histories, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck, 21.2 mph ($\alpha = 0.05$), Lane 2, Eastbound
FIG. 4.16  Rottom Flange Strain Histories, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck, 21.2 mph ($\gamma = 0.05$), Lane 2, Eastbound
FIG. 4.17 Deflection Histories, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck, 39.0 mph ($\alpha = 0.10$), Lane 2, Eastbound
FIG. 4.18 Bottom Flange Strain Histories, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Vehicle, 39.0 mph ($\sigma = 0.10$), Lane 2, Eastbound
FIG. 4.19 Deflection Histories, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck 59.7 mph ($\alpha = 0.15$), Lane 2, Eastbound
FIG. 4.20  Bottom Flange Strain Histories, All Beams, Section B, Salt Fork River Bridge -- FHWA Test Truck, 59.7 mph ($\alpha = 0.15$), Lane 2, Eastbound
FIG. 4.21 Deflection Histories, Beam 3, Section A, Salt Fork River Bridge -- FHWA Test Truck, Lane 2, Eastbound
FIG. 4.22  Deflection Histories, All Beams, Section A, Shaffer Creek Bridge, 1968 -- FHWA Test Truck, 32.1 mph
($\alpha = 0.07$), Lane 2
FIG. 4.23 Bottom Flange Strain Histories, All Beams, Section A, Shaffer Creek Bridge, 1968 -- FHWA Test Truck, 32.1 mph ($\alpha = 0.07$), Lane 2
FIG. 4.24  Bottom Flange Strain Histories, All Beams, Section A, Shaffer Creek Bridge, 1968 -- FHWA Test Truck 54.0 mph ($\alpha = 0.12$), Lane 2
FIG. 4.25  Bottom Flange Strain Histories, All Beams, Section A, Shaffer Creek Bridge, 1968 -- Typical 35-2 Tractor, Semi-Trailer, 72.5 kips, 45.7 mph ($\alpha = 0.10$), Lane 2
FIG. 4.26 Deflection Histories, All Beams, Section A, Shaffer Creek Bridge, 1968 -- Typical 3S-2 Tractor, Semi-Trailer, 72.5 kips, 45.7 mph ($\alpha = 0.10$), Lane 2
FIG. 4.27 Deflection Histories, All Beams, Section A, Shaffer Creek Bridge, 1968 -- Typical 35-2 Dump Truck, 70.1 kips, 45.7 mph ($\alpha = 0.10$), Lane 2
FIG. 4.28  Bottom Flange Strain Histories, All Beams, Section A, Shaffer Creek Bridge, 1968 -- Typical 3S-2 Dump Truck, 70.1 kips, 45.7 mph ($\alpha = 0.10$), Lane 2
FIG. 4.29 Top and Bottom Flange Strain Histories and Neutral Axis History, Beam 3, Section A, Shaffer Creek Bridge, 1969 -- Typical 3S-2 Tractor, Semi-Trailer, 79.1 kips, 43.5 mph ($\alpha = 0.09$), Lane 2
FIG. 4.30  Top and Bottom Flange Strain Histories and Neutral Axis History, Beam 4, Section A, Shaffer Creek Bridge, 1969 -- Typical 3S-2 Tractor, Semi-Trailer, 79.1 kips, 43.5 mph ($\alpha = 0.09$), Lane 2
FIG. 4.31 Typical Strain History, Transverse Slab Reinforcing Bar Between Beams 2 and 3, Section E, Salt Fork River Bridge, FHWA Test Truck, 40 mph, Lane 2
FIG. 4.32 Effect of Vehicle Speed on Transverse Slab Reinforcing Bar Strain, Between Beams 2 and 3, Section E, Salt Fork River Bridge, FHWA Test Truck
FIG. 4.33 Coding of Truck Types
FIG. 4.34 Histogram for Truck Axle Types, All Trucks, Shaffer Creek Bridge, 1969

Total Number = 861

Number of Trucks

Truck Axle Types

59.3 %
FIG. 4.35 Histogram for Truck Axle Type, All Trucks, CB&Q Bridge, 1969

Total Number = 1489

Truck Axle Types

Number of Trucks

3S-2 2S-2 2D 2S-1-2 2S-1 3D 3S-1 2S-1-1 3S-1-1 3S 2S 2D-2 2S-3 3D-2 2D-1 2D-3
FIG. 4.36 Histogram for Gross Vehicle Weight, All Trucks, Shaffer Creek Bridge, 1968
FIG. 4.37 Histogram For Gross Vehicle Weight, All Trucks, Shaffer Creek Bridge, 1969
FIG. 4.38 Histogram For Gross Vehicle Weight, All Trucks, CB&Q Bridge, 1969
FIG. 4.39 Histogram For Gross Vehicle Weight, 3S-2 Trucks, Shaffer Creek Bridge, 1969
FIG. 4.10 Histogram For Gross Vehicle Weight, 3S-2 Trucks, CB&Q Bridge, 1969
FIG. 4.41 Histogram For Axle Weight, All Trucks, All Axles, Shaffer Creek Bridge, 1969
FIG. 4.42 Histogram For Axle Weight, All Trucks, All Axles, CB&Q Bridge, 1969
FIG. 4.43 Histogram of Axle Weight, 3S-2 Trucks, All Axles, Shaffer Creek Bridge, 1969
FIG. 4.44 Histogram For Axle Weight, 35-2 Trucks, All Axles, CB&Q Bridge, 1969
FIG. 4.45 Histogram For Axle Weight, All Trucks, Axle A, Shaffer Creek Bridge, 1968
FIG. 4.46 Histogram For Axle Weight, All Trucks, Axle A, Shaffer Creek Bridge, 1969
FIG. 4.47 Histogram for Axle Weight, All Trucks, Axle A, CB&Q Bridge, 1969
FIG. 4.42 Histogram For Axle Weight, All Trucks, Axle B, Shaffer Creek Bridge, 1965.
FIG. 4.49  Histogram For Axle Weight, All Trucks, Axle B, Shaffer Creek Bridge, 1969
FIG. 4.50 Histogram For Axle Weight, All Trucks, Axle B, CB&Q Bridge, 1969
FIG. 4.51 Histogram for Axle Weight, All Trucks, Axle C, Shaffer Creek Bridge, 1968
FIG. 4.52 Histogram For Axle Weight, All Trucks, Axle C, Shaffer Creek Bridge, 1969
FIG. 4.53 Histogram For Axle Weight, All Trucks, Axle C, CB&Q Bridge, 1969
FIG. 4.54 Histogram For Axle Weight All Trucks, Axle 3, Shaffer Creek Bridge, 1958
FIG. 4.55
Histogram For Axle Weight, All Trucks, Axle D or E, Shaffer Creek Bridge, 1969
FIG. 4.56 Histogram For Axle Weight, All Trucks, Axle D, CB&Q Bridge, 1969
FIG. 4.57 Histogram For Axle Weight, All Trucks, Axle E, Shaffer Creek Bridge, 1968
**FIG. 4.58** Histogram For Axle Weight, All Trucks, Axle E, Shaffer Creek Bridge, 1969
FIG. 4.59  Histogram For Axle Weight, All Trucks, Axle E, CB&Q Bridge, 1969
FIG. 4.60 Histogram For Axle Weight, 3S-2 Trucks, Axle A, CP&Q Bridge, 1969
FIG. 4.61 Histogram For Axle Weight, 3S-2 Trucks, Axle B, CB&Q Bridge, 1969
FIG. 4.62 Histogram For Axle Weight. 3S-2 Trucks, Axle C, CS&Q Bridge, 1969
FIG. 4.63 Histogram For Axle Weight, 3S-2 Trucks, Axles D and E, CB&Q Bridge, 1969
FIG. 4.54 Histogram For Axle Spacing, All Trucks, Spacing A-B, CS&Q Bridge, 1969
FIG. 4.66 Histogram For Axle Spacing, All Trucks, Spacing C-D, CB&Q Bridge, 1969
FIG. 4.67 Histogram For Axle Spacing, All Trucks, Spacing D-E, CB&Q Bridge, 1969
FIG. 4.68 Histogram For Time Interval, All Trucks, Shaffer Creek Bridge, 1969
FIG. 4.69  Histogram For Time Interval, ALL Tunnels - CSX Bridge, 1969
FIG. 4.70 Maximum Strain Range in Beam 4 Plotted versus Gross Vehicle Weight For Selected Traffic and Test Truck Runs at Shaffer Creek Bridge in 1968
FIG. 4.71 Maximum Strain Range in Beam 5 Plotted versus Gross Vehicle Weight for Selected Traffic and Test Truck Runs at Shaffer Creek Bridge in 1968.
FIG. 4.72 Histogram For Truck Type, 148 Selected Runs, Shaffer Creek Bridge, 1968
FIG. 4.73  Histogram For Gross Vehicle Weight, 148 Selected Runs, Shaffer Creek Bridge, 1968
FIG. 4.74 Histogram of Transverse Position, Selected Runs, Shaffer Creek Bridge, 1968

Deviation From Traffic Lane Centerline

Percent of Total

NORTH

feet

SOUTH

0 1/2 1 1 1-1/2 2 2-1/2 3 3-1/2 4

0 10 20 30 40 50
FIG. 4.75 Length versus Strain Range in Beams, 35-2 Dump Trucks, Shaffer Creek Bridge, 1968
FIG. 4.76 Histogram For the Percent of The Total Strain at Section A in Beam 5, Shaffer Creek Bridge, 1968 -- Heavy Truck Traffic
FIG. 4.77 Variation of Strain Range vs. GVW, Beam 4, Section A
Shaffer Creek Bridge, 1968 -- Heavy Truck Traffic,
S_2 = 5 to 40 ft., Speed = 40 to 65 mph

\[ \varepsilon_R = 2.50 \times \text{GVW} + 11.5 \]

Dev. = 23.7
\[ \rho = 0.91 \]
FIG. 4.78 Variation of Strain Range vs. GVW, Beam 5, Section A, Shaffer Creek Bridge, 1968 -- Heavy Truck Traffic, $S_2 = 5$ to 40 ft., Speed = 40 to 65 mph
FIG. 4.79 Histogram For Bottom Flange Strain Range, Beam 1, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic

236 Counts
Totals
236 Trucks

\[ \mu = 26.4 \]
\[ \sigma = 12.3 \]
\[ \text{c.o.v.} = 0.46 \]
230 Counts
Totals
230 Trucks

\[ \mu = 41.8 \]
\[ \sigma = 21.6 \]
\[ \text{c.o.v.} = 0.52 \]

FIG. 4.80 Histogram For Bottom Flange Strain Range, Beam 2, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic
FIG. 4.81 Histogram for Bottom Flange Strain Range, Beam 3, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic
FIG. 4.82 Histogram For Bottom Flange Strain Range, Beam 4, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic

241 Counts
Totals
241 Trucks

$\mu = 110.6$
$\sigma = 56.6$
$c.o.v. = 0.51$
FIG. 4.83 Histogram For Bottom Flange Strain Range, Beam 5, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic

236 Counts
Total
236 Trucks

$\mu = 129.0$
$\sigma = 66.4$
$c.o.v. = 0.51$
FIG. 4.84 Histogram For Bottom Flange Partial Strain Range, Beam 2, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic.
FIG. 4.85 Histogram For Bottom Flange Partial Strain Range, Beam 3, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic

- 1341 Counts
- 250 Trucks

\[
\mu = 54.6 \\
\sigma = 33.0 \\
c.o.v. = 0.60
\]
FIG. 4.86 Histogram For Bottom Flange Partial Strain Range, Beam 4, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic
FIG. 4.87 Histogram For Bottom Flange Partial Strain Range, Beam 5, Section A, Shaffer Creek Bridge, 1968, Selected Truck Traffic

 Counts
 Totals
 249 Trucks

\[ \mu = 65.4 \]
\[ \sigma = 51.4 \]
\[ \text{c.o.v.} = 0.79 \]
APPENDIX A

NATURAL FREQUENCIES AND MODES OF THE SHAFFER CREEK BRIDGE

During the course of the investigation of the finite element representation of highway bridges Eberhardt (9) determined natural frequencies and modes for an example problem on the Shaffer Creek structure. The natural frequencies and modes so determined were used to establish values of the parameters for the numerical analysis.

In making these computations a stiffness matrix for the beams and slab corresponding to a partial degree of composite action was used. This also yielded deflection values which were consistent with measured data. Thus the use of a modified stiffness matrix implies that the natural frequencies and modes so computed are for the loaded condition. Thus in considering the results which will be presented in the following, one must not expect agreement with the measured unloaded natural frequencies of the bridge. These frequencies might then be more comparable to those based on a forced vibration test in which substantial breakdown and composite action developed.

The finite element modeling of the bridge is shown in Fig. A.1. A total of 41 node points were considered, but of these 15 are fixed points over the supports; thus a 30-degree-of-freedom system for vertical oscillation was considered. Using this model, the interpretation of stiffness noted above, and a consistent mass matrix, the first four natural frequencies and modes are illustrated in Figs. A.2 and A.3. The first and second modes, Fig. A.2, correspond to a single sine wave along the total length of the bridge that is one node in each span of the two span continuous bridge. The lowest mode is nearly symmetric;
and the second is a true antisymmetric mode. The third and fourth modes for the structure correspond to a longitudinal distribution corresponding to the second longitudinal mode of a beam analysis, again one symmetric and one antisymmetric.

In Fig. A.4 the values for all natural frequencies of the 30-degree-of-freedom model of the Shaffer Creek bridge are shown. It should be remembered that only the first few natural modes are significant in response to the bridge. Only the fundamental and the first torsional mode can be detected at all in the response histories. The higher natural frequencies (mode numbers) are significant only in the numerical analysis.
FIG. A.1 Shaffer Creek Five-Beam Bridge Model
First Mode
\[ f_1 = 6.60 \text{cps} \]

Second Mode
\[ f_2 = 7.87 \text{cps} \]

FIG. A.2 First and Second Mode Shapes For Shaffer Creek Bridge
Third Mode
\[ f_3 = 10.35 \text{ cps} \]

Fourth Mode
\[ f_4 = 11.77 \text{ cps} \]

FIG. A.3 Third and Fourth Mode Shapes For Shaffer Creek
FIG. A.4 Natural Frequencies For The 30 Degree-of-Freedom Shaffer Bridge Model